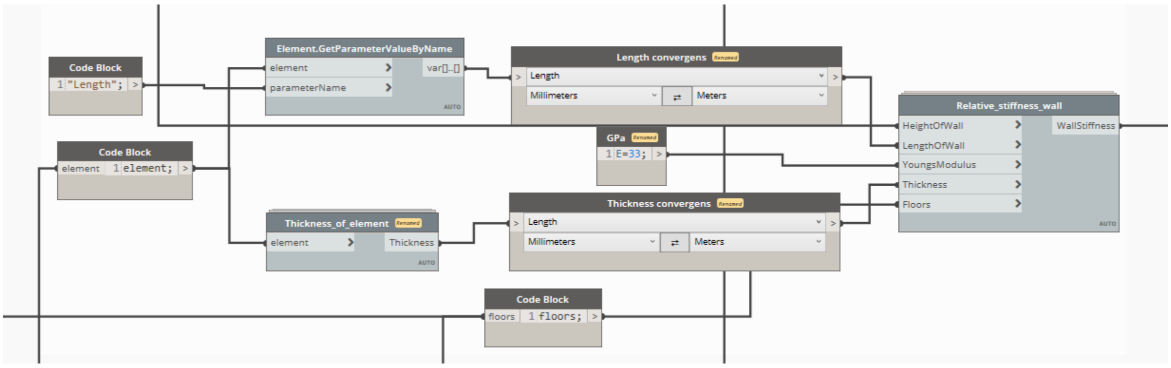
Semi-Automatic calculation of the stability of shear walls in multi-storey buildings



Semi-automatisk beräkning av stabiliteten för skjuvväggar i flervåningsbyggnader

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Abstract

Performing horizontal stability analysis in the conceptual design phase is time consuming and adjustments in the geometry of the building require new time consuming calculations. Often buildings are modelled in 3D modelling software such as Revit and Tekla but the calculations are often performed in a separate software. A script in Dynamo (add-in to Revit) has been designed that manages to gather geometric data and material data from the Revit model and automatically performs horizontal stability calculations for the stabilising system of a concrete building. The purpose of the script is to easily design the stabilising system based on the results of the overturning capacity of the shear walls based on analytical equations. The Dynamo script performs calculations for buildings that are stabilised with shear walls in ULS with load case EQU 6.10[5] (SS-EN 1990). EQU includes both symmetric and asymmetric wind load (over the width of the building) and varying wind load over the height of the building according to Eurocode and EKS11. Imperfection loads are also included. The Dynamo script presents the result of the overturning moment and stabilising moment in a bar chart for the different shear walls. The user can adjust the shear walls (thickness, location, length) in the Revit model and then run the Dynamo script again to see if the overturning moment exceeds the stabilising moment.

Sammanfattning

Stabilitetsberäkningar under tidigt projektstadium är tidskrävande och små ändringar i byggnadens geometri kräver nya tidskrävande beräkningar. Vanligtvis är byggnader modellerade i 3D program såsom Revit och Tekla men beräkningar utförs i andra program. Ett beräkningsprogram i Dynamo (tillägg till Revit) har skapats som kan hämta geometriska data och material data från en Revit modell och automatiskt utföra stabilitetsberäkningar i en betongbyggnad för det stabiliserande systemet baserat på analytiska formler. Syftet med beräkningsprogrammet är att enkelt kunna designa det stabiliserande systemet med hänsyn till stjälpning av skjuvväggar. Beräkningsprogrammet utför beräkningar för byggnader som är stabiliserade med skjuvväggar och använder sig av ULS lastkombination EQU 6.10[5] (SS-EN 1990). EQU inkluderar både symmetrisk och osymmetrisk vindlast (över bredden av byggnaden) och varierande vindlast över höjden av byggnaden enligt Eurokod och EKS11. Imperfektionslaster har även inkluderats i beräkningsprogrammet. Beräkningsprogrammet presenterar resultatet för de olika skjuvväggarna och användaren kan snabbt och enkelt se stabiliserande moment och stjälpande moment i ett stapeldiagram. Användaren kan modifiera skjuvväggarna (tjocklek, plats, längd) i Revit modellen och sen köra beräkningsprogrammet igen och se om det stjälpande momentet överskrider det stabiliserande momentet.

Acknowledgements

This master thesis marks the final part of our studies at LTH. We have had very fun during our years and want to thank all of our classmates and teachers.

We would like to thank our supervisors Nils Dahlman and Eliott Sandefeld at ELU Konsult AB for giving great inputs on the Dynamo script and always helping us. We would also express our gratitude to our supervisor Eva Fruhwald Hansson at LTH for being supportive and guiding us through this work. Also we would like to thank Jonas Niklewski at LTH for being quick to answer when we had questions.

Finally we would like to thank ELU Anläggning Malmö for providing a workplace where the master thesis could be carried out. They were very welcoming and provided great breakfasts.

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Emil Sjöstedt & Erik Bolin

Notations, Symbols and Translations

Translate *Swedish* to *English*

1. Plattbärlag - Permanent formwork
2. Stel skiva - Diaphragm
3. Betongplatta/betongbjälklag - Slab
4. Lovartsida - Windward side
5. Läsida - Leeward side
6. Program som utför uppgifter - Script
7. Sheet - Ark
8. Vy - View
9. CALFEM - Finite element package for MATLAB

Explanations

1. Veddesta - Case study object planing to be built i Stockholm, Veddesta
2. Python - Computer programming language
3. API - Stands for *Application Programming Interface*. It is a software intermediary that allows two applications to talk to each other.
4. Revit - 3D modelling software. Visit webpage for more information: `Https://www.autodesk.se/products/revit/overview?panel=buy&term=1-YEAR&tab=subscription&plc=RVT`
5. Dynamo - Visual programming using Python and pre-programmed functions. Visit webpage for more information: `https://dynamobim.org/`

Contents

1 Introduction

When it comes to designing the structural load bearing system for a structure, the stabilising system must be handled with great respect. The stabilising system is responsible for taking care of the horizontal forces affecting the structure and transferring them to the foundation. The stabilising system must also make the structure stable for vertical loads.

The stabilising system can be designed in many different configurations but one common configuration is to use shear walls.

Structures nowadays are usually 3D modelled in different kind of software. Common software for 3D modelling are Revit and Tekla. 3D models of structures are used to visualize the structure and render drawings. The calculations on the structural system are often done using hand calculations and using a finite element method (FEM) software. Both hand calculations and the FEM calculations can be quite cumbersome and time consuming, depending on the complexity. Small changes in the geometry and structural layout need to be manually adjusted in the hand calculations and FEM model and the calculations have to be reiterated.

ELU Konsult AB want to optimize the workflow process of doing stability calculations of structures by using Dynamo. It is beneficial to be able to use the 3D model in Revit directly to carry out calculations for the stabilising system. All the geometrical data and material data are already in the 3D model. Dynamo is able to gather information such as geometry, material data from the Revit model. This information can then be used in Dynamo to write scrips that can perform calculations based on analytical equations, adding elements to 3D models by accessing the Revit *API*, visualize data and so on.

Neither Erik Bolin, Emil Sjöstedt or Faculty of Engineering, (LTH) take responsibility for this program. Use the program with caution and always make control calculations.

1.1 Research and purpose

The purpose of this master thesis is to construct a script in Dynamo (add-in to Revit) that can perform design verifications for multi-storey buildings that are stabilised with shear walls. The user can easily optimize the stabilising element's positioning, geometry in the 3D model and then run the Dynamo script to quickly see how the changes affect the stabilising capacity of the structure. This can be reiterated by the user to find an optimized configuration of the geometry and positioning of the shear walls.

The script in Dynamo is based on a case study for a multi-storey building in Veddesta, Stockholm. The case study object Veddesta is denoted "Veddesta" for the remaining thesis.

The goal with the script is to automate it as much as possible so the user does not have to manually enter data concerning the structure. The less instructions that are required to perform the Dynamo script - the better. The more instructions, the risk is increased that the script won' be utilized. This is a request from our supervisors at ELU Konsult AB that was taken into account when deciding on limitations and assumptions.

For the script in Dynamo, some questions have to be investigated so the script is created in the best way. The following questions are investigated:

1. How do openings in shear walls affect the load distribution.
2. What horizontal load distribution model should be implemented in the Dynamo script?
3. How can the vertical load distribution be modelled based on a 3D model where there is no data on the boundary conditions and span direction for the slabs and roof?
4. How can the Dynamo script be used for case study object Veddesta to achieve sufficient capacity with only shear walls?

1.2 Limitations

Limitations have been set to generalize the script to work for different building types and geometries. Limitations have also to be set in order to complete the master thesis in reasonable time. The following limitations were set:

1. The Dynamo script and the thesis only covers concrete slabs and walls.
2. The Dynamo script only works for buildings that are stabilised with shear walls.
3. The Dynamo script only works for shear walls that are parallel or perpendicular to all the other shear walls
4. Horizontal load distribution to stabilising elements on all floors is assumed to be according to relative stiffness.
5. The vertical load distribution follows the shortest path (stiffness not taken into account).
6. Constant floor geometry throughout the height of the building is needed to get accurate results.
7. Dynamic effect of wind load is not considered.

8. The Dynamo script is intended for buildings with flat roof.

9. The Dynamo script only makes controls with EQU load combination considering wind load, imperfection load and dead load.

10. The shape of the buildings corners and roof angle/shape are not taken into account when calculating the design wind load.

1.3 Method

First a literature study is conducted to get acquainted with how stabilisation of buildings is achieved with shear walls. The literature study is conducted by reading books, reading scientific papers and old master theses that have covered similar topics.

From discussions with our supervisors at ELU Konsult AB we will get insight in how they envision the script in Dynamo for both what the Dynamo script will calculate (e.g. overturning moment, influence area etc) and how they want the results to be presented.

Time will be spent on learning both how Revit and Dynamo works. Dynamo and Revit are not software we have worked with previously. Simultaneously, as the script is created, verifications will be done using hand calculations to validate the calculations in Dynamo. A 3D model in Revit of Veddesta will be the basis of the creation of the script.

The study of how the load distribution in the shear walls is affected by openings will be made with different analytical equations and then compared with finite element method in MATLAB using CALFEM.

The process of the master thesis will somewhat follow the flow chart seen in figure 1.1. Writing and documentation of the work will be added while the work goes on.

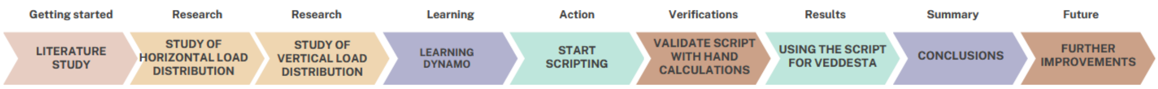


Figure 1.1: Flowchart of how the work on the master thesis will proceed.

1.4 Case study object

The case study object is a high-rise building in Veddesta, Stockholm. The building will be a residential building with 33 floors and a total height of 99 m. The building is, if possible, to be stabilised with only shear walls. Currently it is supposed to have the shear walls complemented with trusses in the exterior (the slender direction) to stiffen the building. However, it is preferred to not have trusses in the exterior of the building as this is complicated to construct.

A challenge with Veddesta is that it's tall and slender building.

The bottom floor does not have the same height as the rest of the floors and the top two floors have a different structural layout for the shear walls compared to rest of the floors. The geometry of the shear walls are not consistent in terms of length from foundation to top floor and some shear walls do not extend from foundation to the top floor. However, as mentioned in the limitations, in this work, the same layout will be used for all floors.

The high-rise building is part of a bigger complex and in Figure 1.2 a Revit 3D model of the high-rise building is seen accompanied with surrounding buildings.

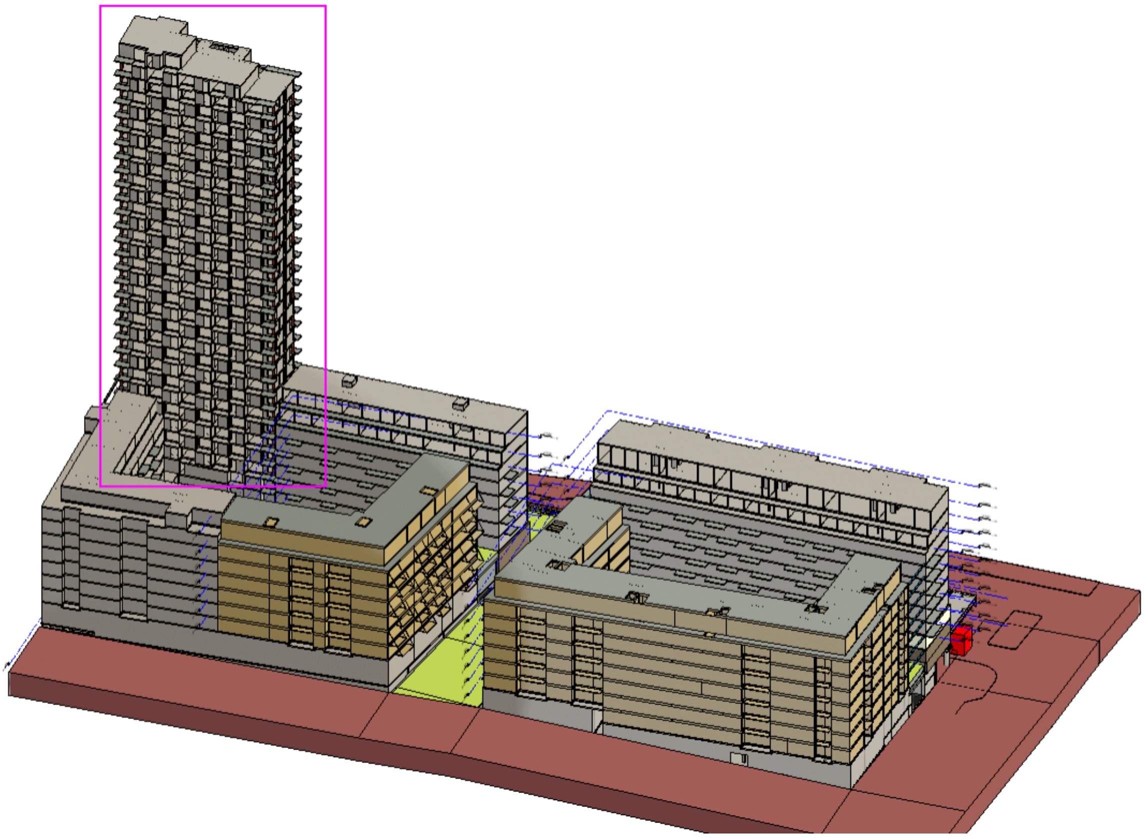


Figure 1.2: Case study object and surrounding area from a Revit model. The high-rise building is marked with a pink rectangle (Source: ELU Konsult AB, edited).

2 Background

2.1 Structural design

In the initial phases of a project, a structural engineer is consulted to present a well functioning structural concept. This structural concept is not only supposed to be structurally safe, but also to be a viable economically and practical to build. In order for the engineer to work on the structural concept, the system needs to be idealized and be analytically manageable. Often this structural concept has to be delivered under pressure as both time and cost is limited. This requires that the engineer has good tools to figure out the structural concept. When the structural concept design is done, the more in depth analysis takes place [1].

In the early design stage, approximations and simplifications are needed and presents the grounds to choosing structural member sizes.

It is in the initial phases of a project the Dynamo script can be utilized. The Dynamo script presents a simplified method that can be used in early stages to determine if the building geometry is reasonable designed in terms of horizontal stability. Before the structural engineer goes on to the stage where detailed calculations are performed, they know the building geometry is within reasonable bounds, and large changes in geometry won't be required at a later stage.

It is always good that an analysis of a structure is done by different methods in order to avoid errors that can be done in the structural design stage or the final design stage [1]. Different methods can be analytical equations for calculations or finite element analysis.

2.2 Stabilisation of structures

A building needs to have a system that takes care of the horizontal loads acting on the building. This is done by the stabilising elements of the building that transfer the loads to the ground. The horizontal loads mainly consist of wind loads, earthquake loads and imperfection loads. Earthquakes generate very large horizontal forces which puts big requirements on the stabilising structure. For multi-storey buildings, the imperfection load has a significant effect on the stabilising system [2]. In Sweden, forces generated by earthquakes do not need to be taken into account for regular buildings.

There are three principal ways of stabilising a structure, see also Figure 2.1: [2]

- Truss action
- Frame action

- Diaphragm action

These stabilising actions can be combined. The principal ways of achieving a stable structure will be explained in more detail later on.

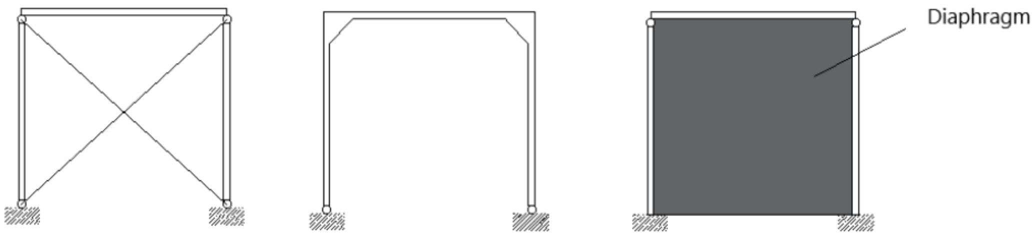


Figure 2.1: Left figure: Truss action, Middle figure: Frame action, Right figure: Diaphragm action

Many different factors come into play when choosing the kind of stabilising system. Some factors to be mentioned are [2, 3]:

- Construction method of building
- Height of building
- Presence of large horizontal forces e.g. wind or seismic forces
- What is the purpose of the building
- Costs
- Design of the foundation
- Need for certain connections

2.2.1 Truss-action

With truss-action, the purpose is to create a stable structure by using triangles. These triangles are created by having elements positioned with an angle relative to the external load. These angled elements can be rods for example. These inclined elements can be placed diagonally as in figure 2.2. The angle of the inclined elements dictates how the forces from the external load is spread in the structure. It is often assumed that the connections between the joints are hinged and thus the inclined elements only resist the load in compression or in tension [2].

Depending on the configuration, the elements can take up load in tension or compression. If the inclined elements are to resist in compression, the elements have to be designed for instability phenomena, i.e. buckling. To reduce the risk of buckling for the compressed elements, they are often braced [2].

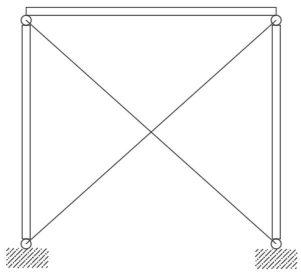


Figure 2.2: Truss-system.

Truss action is commonly used in large industrial facilities. There, the trusses can be placed in the walls of the structure and also in the roof. The trusses have to be placed in both x and y direction in the walls and the roof in order to withstand horizontal load from all directions. The trusses in the roof transfer the wind load to the trusses in the walls, which then transfer the load to the ground [2]. Figure 2.3 shows how an industrial facility being stabilised by truss system in roof and walls can look like [2].

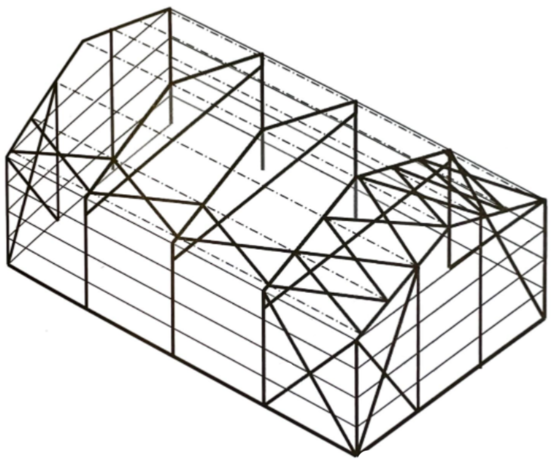


Figure 2.3: Industrial facility stabilised by truss system in both roof and wall (Source: [2], edited).

2.2.2 Frame-action

Another way of achieving stabilisation is to use frame action. This means that horizontal forces are resisted by moment stiff connections between structural elements or moment stiff connection to the foundation. Achieving moment stiff connections between elements are expensive. There are many configurations of how the connections can be made [2].

The use of frame action requires large dimensions of the elements and the costs increase quickly as the height increases. The use of frame action is also not suitable for prefabricated elements of steel and concrete due to the requirement of joints that can transfer moment [4].

The right configuration in figure 2.4 results in high moment in the foundation whereas the left configuration does not result in any moment in the foundation. A moment stiff connection between vertical element and foundation can be complicated to design [2]

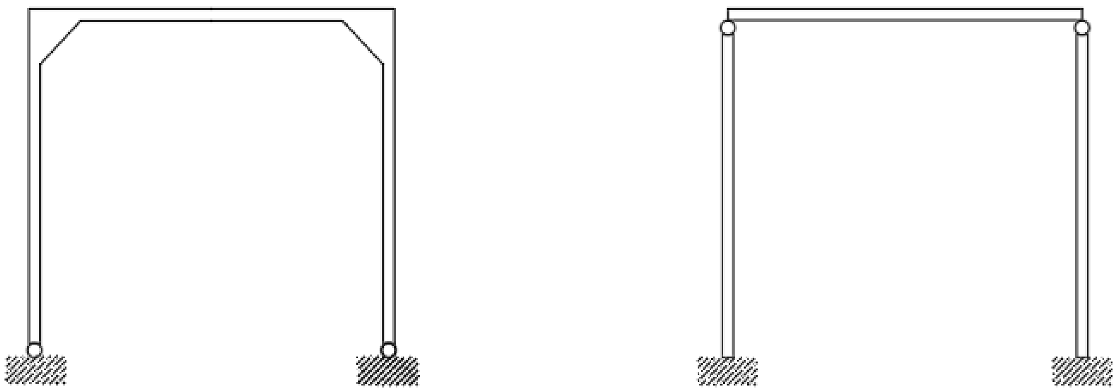


Figure 2.4: Left: Moment stiff connection between vertical and horizontal element, Right: Moment stiff connection between vertical element and foundation.

2.2.3 Diaphragm action

The third of way of achieving stabilisation is to use diaphragm action. Diaphragm action means that a diaphragm is stiff in its own plane and thus the diaphragm can transfer forces in its own plane. For a diaphragm the bending stiffness perpendicular to its plane is weak [2]. This is illustrated in figure 2.5

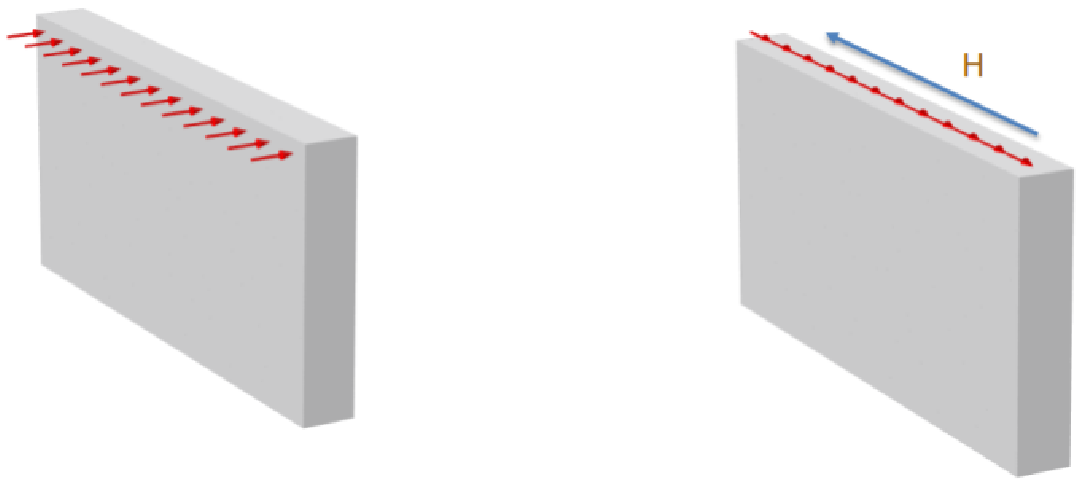


Figure 2.5: Left figure shows diaphragm loaded in its weak direction. Right figure shows diaphragm loaded in its own plane (stiff direction) (source: [5], edited).

Diaphragms can be made of concrete, steel, timber and masonry. Diaphragms can be orientated in vertical direction as well as in horizontal direction [2]. Whether a slab can be considered a diaphragm depends on the thickness of the slab. In general, the thickness requirements for concrete slabs due to vertical load and sound insulation give the slab sufficient thickness to be considered as a diaphragm in calculations [3].

Diaphragm action can be utilized in different configurations to achieve a stable building. It is common to use diaphragm action when designing large industrial facilities. For those cases it is common to let the roof act as a diaphragm. The roof is usually made of steel with a thin profile. The roof acts as a diaphragm in the way that it is able to transfer forces in its own plane. These forces still need to be transferred to the ground and one way is to have wind braces in the gables and the long-sides that transfer the horizontal forces to the foundation [2]. Figure 2.6 illustrates this for a wide one storey building.

Diaphragm action is also commonly utilised through shear walls in office and residential buildings. Limitations exist of how tall a building can be with only shear walls. For taller buildings, they can be combined with trusses and stabilising cores.

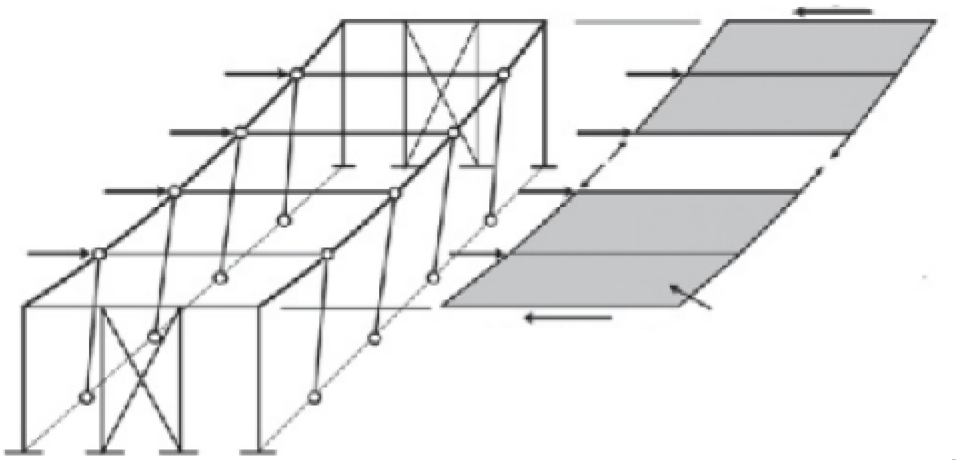


Figure 2.6: Diaphragm action in the roof. The roof is modelled as a beam where the gables act as supports (Source: [6], edited).

2.2.4 Height of building related to stabilising system

There are different ways of achieving a stable building and the high-rise structural engineer Fazlur Kahn developed a guidance for what kind of stabilising system of a building that should be used depending on the amount of floors in a building [3], see Table 2.1. It can be seen as the amount of floors increases, stabilising actions are often combined in order to achieve a stiffer building.

Table 2.1: Stabilising system related number of floors from F.Kahn [3].

Static system	Highest number of floors
Moment stiff connection between columns and beams	15
Hinged connected beams and columns connected to a central tower of concrete or vertical truss-system	25
Fully or partly moment stiff connected beams and columns connected to a tower in concrete or vertical truss-system	40
Same system as for 40 floors supplemented with horizontal trusses in central of the building and top. The horizontal trusses are placed in the facade and also between the central core and the facade	60
Facade columns connected to facade beams to form a structural framework. Together they act as a rectangular cantilever pipe	80
Combination of frame-action and trusses in the facade walls. The facades are connected so that they together act as a rectangular cantilever pipe	100
The building is divided into several "pipes" so that the inner columns work together with the facade. The "pipes" end on different heights so that the wind-load influence on the system is as low as possible.	110
The exterior "pipe" in the previous alternative is combined with large trusses on the exterior.	120

2.3 Revit and Dynamo

Revit is a building information modelling (BIM) software. It is used in several professions, e.g. structural engineering, architecture, plumbing and so on. Revit is used for 3D modelling and render drawings. A layout of the software is shown in Figure 2.7.

Dynamo is a visual programming add-in for Revit. Dynamo makes it able for the user to design custom computational design processes. Dynamo has access to to Revit API. With Dynamo, the user can create scripts that can do multiple things that can improve the efficiency and workflow for the user. Depending on the discipline, one may use Dynamo for different purposes. For structural engineering, it can be used to do repetitive tasks e.g. rotating all columns or placing reinforcement in structural elements in the Revit model. It can also be used to perform calculations, for example, on the stabilisation system based on Revit model. Since all the geometry and elements are already modelled in Revit, it is very efficient to do calculations since the data from Revit can easily can be imported to the Dynamo script.

The layout for Dynamo is shown in figure 2.8.

Dynamo consists of several "building blocks". Figure 2.9 shows the different "building blocks". The box denoted 1 is called a node and a node represents objects or functions.

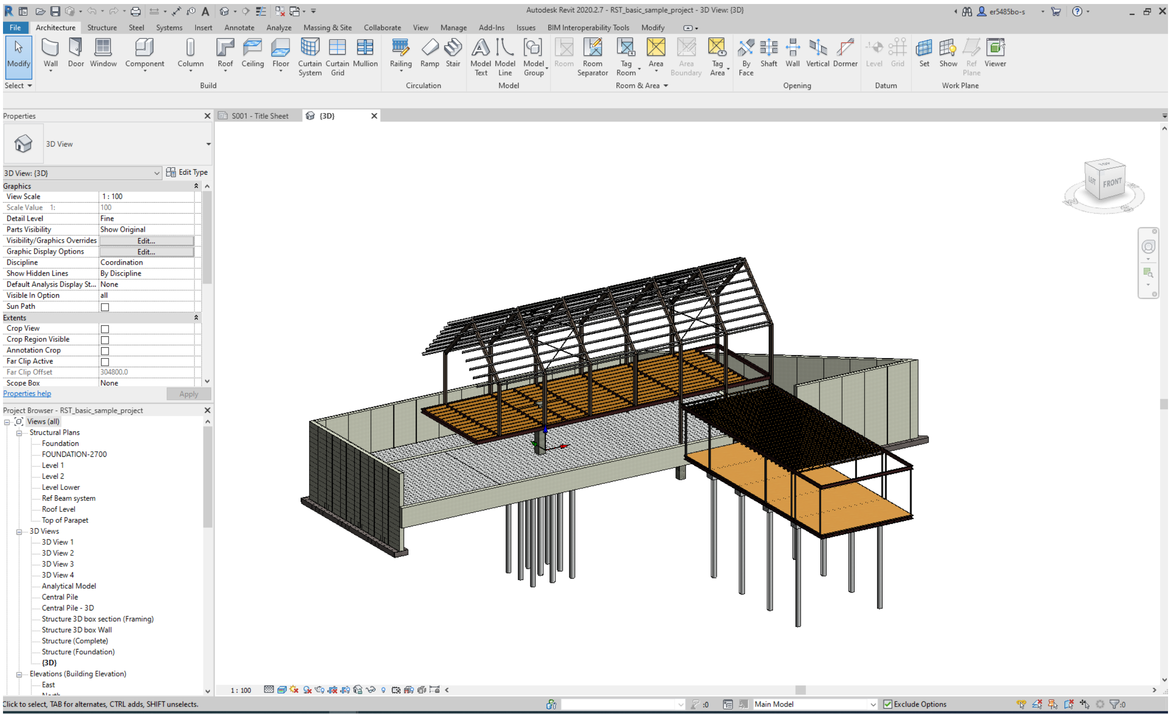


Figure 2.7: Layout of Revit.

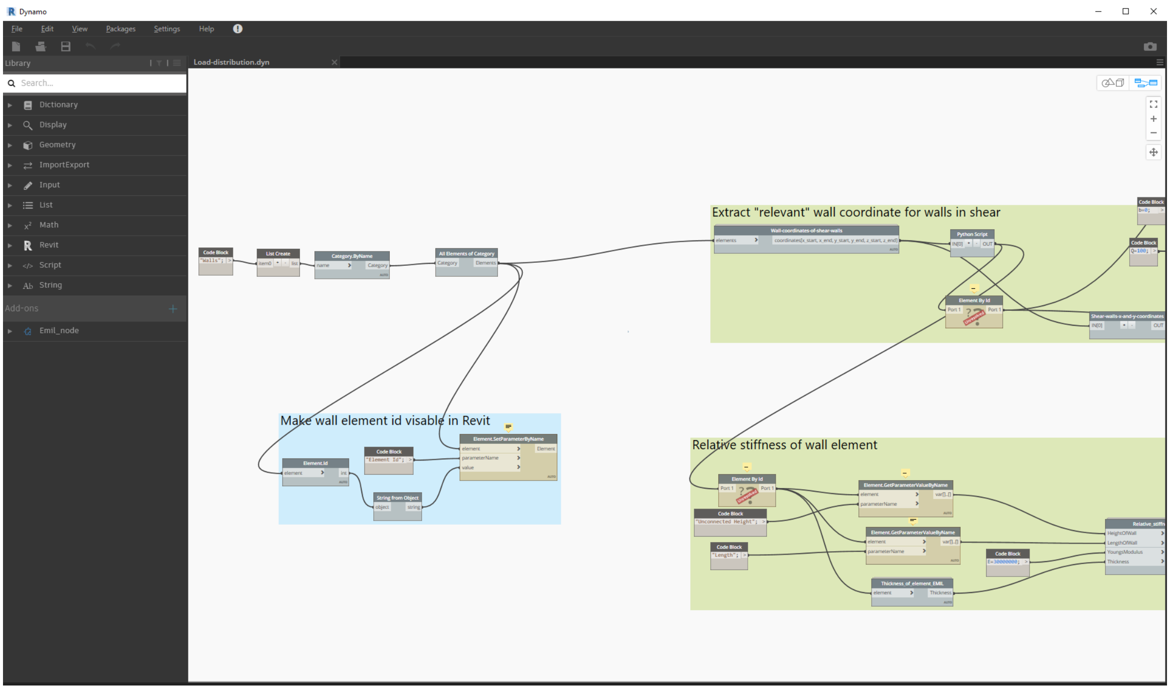


Figure 2.8: Layout of Dynamo.

The line denoted 2 is a wire, determining how the nodes are together connected to form a set of instructions. The arrow denoted 3 shows the direction the data flows, from left to right. How the nodes are wired together determines the order of operations.

In a node, the user can add a python script that manipulates the input data going into the node. The result from the python script can then be extracted from the node as output data. This is showcased in figure 2.10.

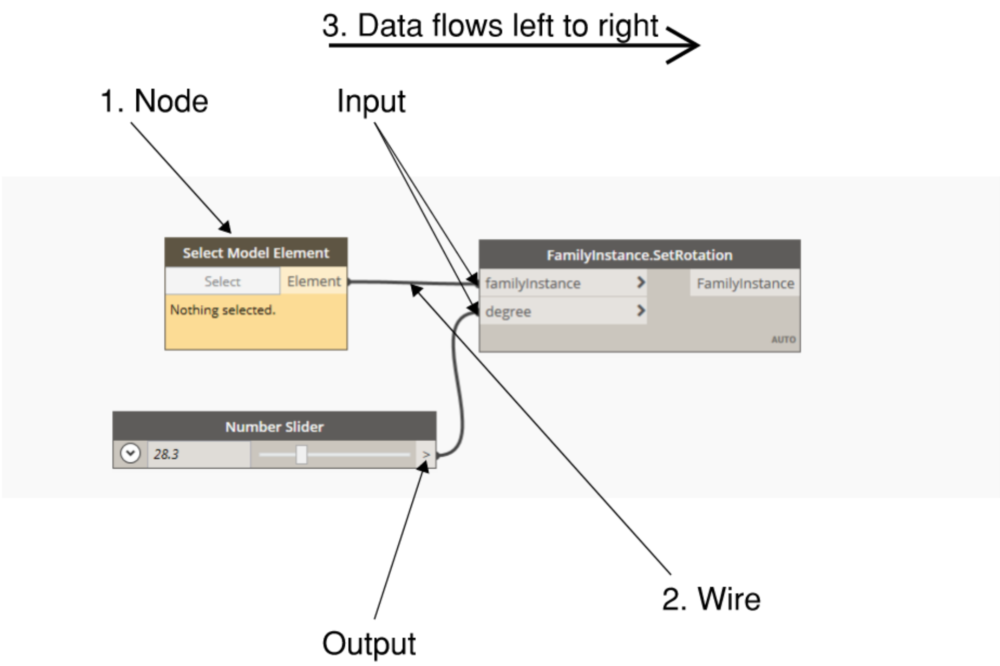


Figure 2.9: The building blocks of Dynamo.

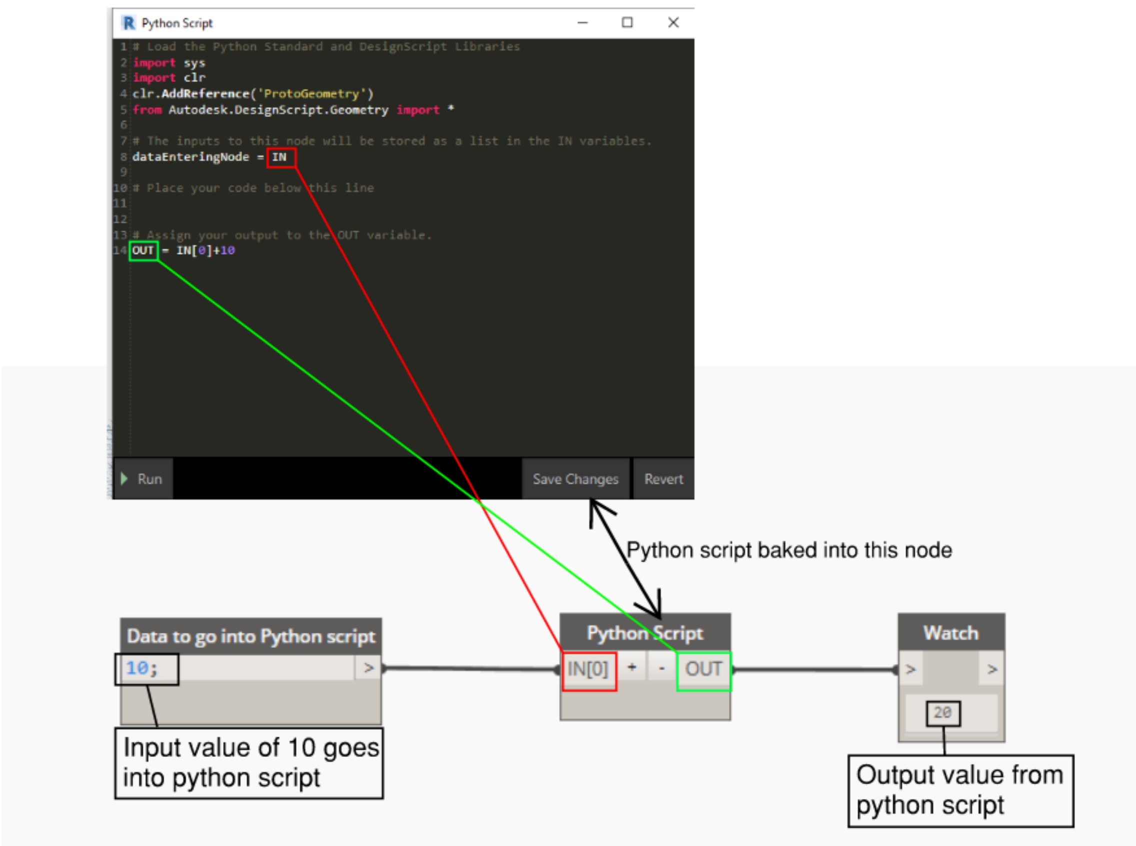


Figure 2.10: Implementation of a python script to a node.

3 Theory

The following chapter covers the subjects surrounding horizontal stabilisation, which is later utilised to build the Dynamo script based on analytical equations.

3.1 Horizontal stabilisation with shear walls

3.1.1 General information

Stabilisation using only shear walls, shear walls together with stabilising core or a single stabilising core are common ways of achieving a stable building. Vertical load bearing walls in buildings can often be used for stabilisation purposes and walls that help to achieve horizontal stabilisation are called shear walls [2].

In figure 3.1 one can clearly see the shear walls. Note the openings in the shear walls in both photos.

Figure 3.1: Two buildings that are stabilised with shear walls. Photos taken by Erik Bolin in Lund.

It is convenient to use shear walls for stabilisation, as the vertical load bearing columns only need to be designed to take vertical loads. If a shear wall is to function as a stabilising element, it is a prerequisite that the slab/roof can act as a diaphragm and thus transfer loads to the stabilising elements.

In tall buildings, shear walls are commonly utilised to carry both horizontal and vertical forces acting on the building by transferring them to the ground [7]. Concrete shear walls are fully fixed to the foundation and must be reinforced according to the building standards. All tension forces must be resisted by the reinforcement [2].

For shear walls, the horizontal forces are transferred through cantilever action to the foundation. In a multi-storey building, the shear walls need to either be continuous or have rigid connections strong enough to transfer the moment.

For a low building that resists horizontal loads by use of shear walls, the shear walls are more seen as cantilever walls. The key difference between a cantilever beam and cantilever wall is that for a wall, the shear deformation has to be considered, meanwhile for cantilever beam only bending deformations need to be considered [2]. For a higher building, stabilised by shear walls, the shear wall is more seen as a cantilever beam. This is illustrated by figure 3.2 where both structures are stabilised horizontally by a shear wall. Both shear walls are fully fixed to the ground. The shear walls have the same width but are of different heights. [7]. For figure 3.2 the left figure's deformation against horizontal load is mainly through shear meanwhile the right figure's deformation is mainly through bending.

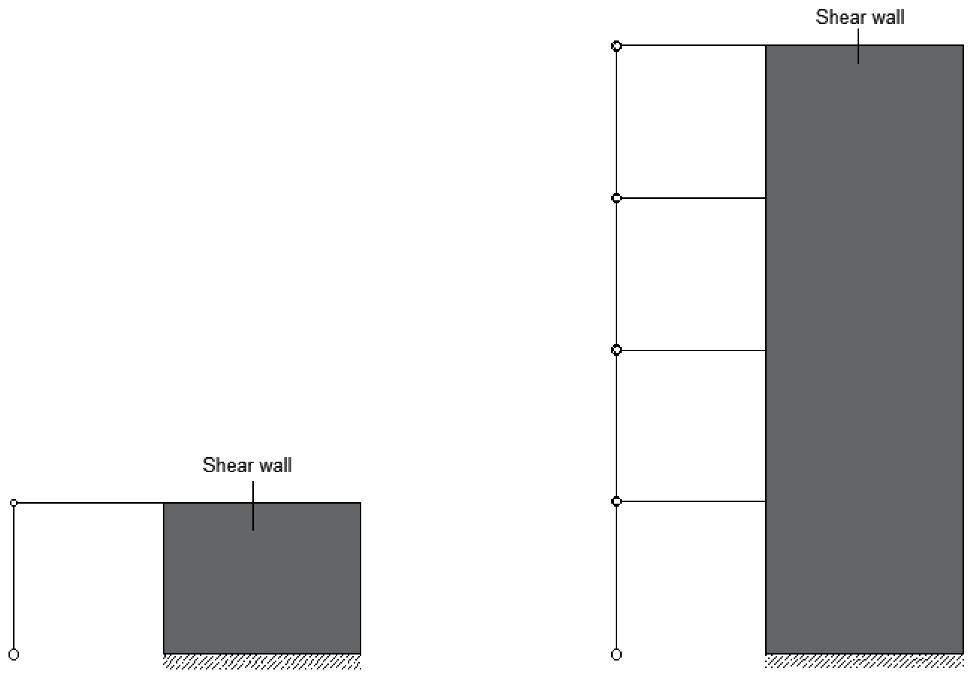


Figure 3.2: Left: Shear wall considered as cantilever wall, Right: Shear wall considered as cantilever beam.

While shear walls have to be able to resist vertical loads and horizontal forces in their own plane, they also have to be verified for horizontal load transversal to its own plane. This is both relevant for shear walls that are placed in the facade and also inner shear walls, as they need to be able to resist inner suction from wind load. This means that the shear walls both need to act as both diaphragms and plates, see figure 3.3 [7].

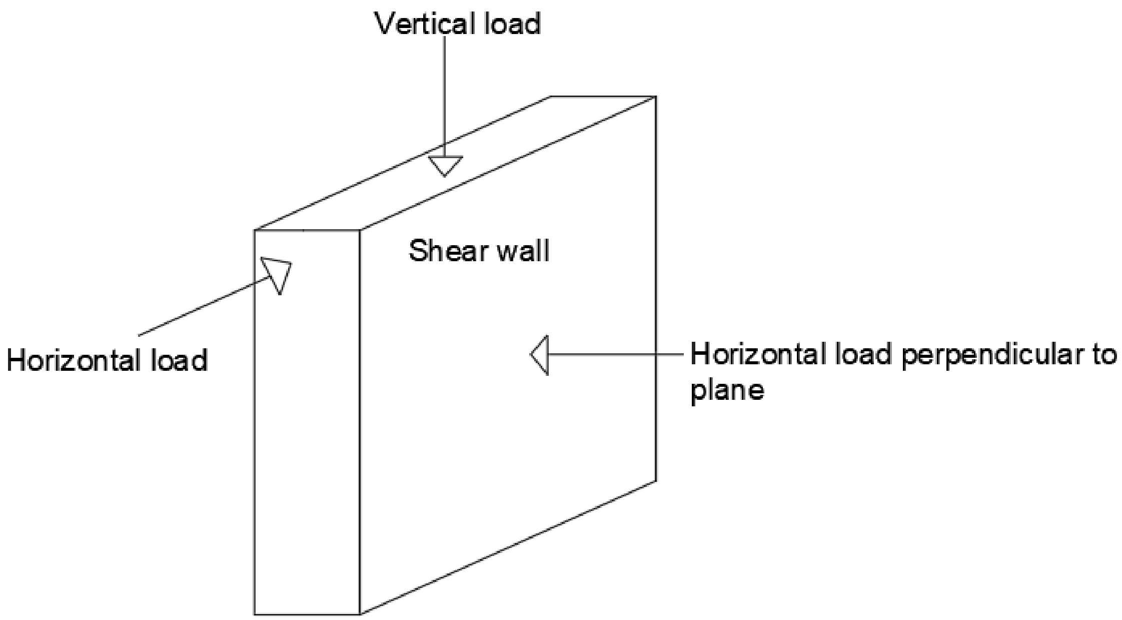


Figure 3.3: Shear wall subjected to horizontal load, vertical load and horizontal load perpendicular to its plane.

3.1.2 Different types of shear walls

Shear walls can have openings with different configurations and depending on the configurations, fractures in the wall occurs in different locations. The size of the openings in shear walls has large impact on the structural behavior of the shear walls [7].

Consider a shear wall that provides lateral support for a one floor building. The shear wall has a big opening in the middle. With the big opening, the shear wall has become a frame with a totally different structural behavior [7]. This is illustrated in figure 3.4. Shear walls or stabilising cores (consisting of shear walls) with openings can be considered as frames if the building has 4 or less floors. Otherwise, they are to be considered as shear walls [4].

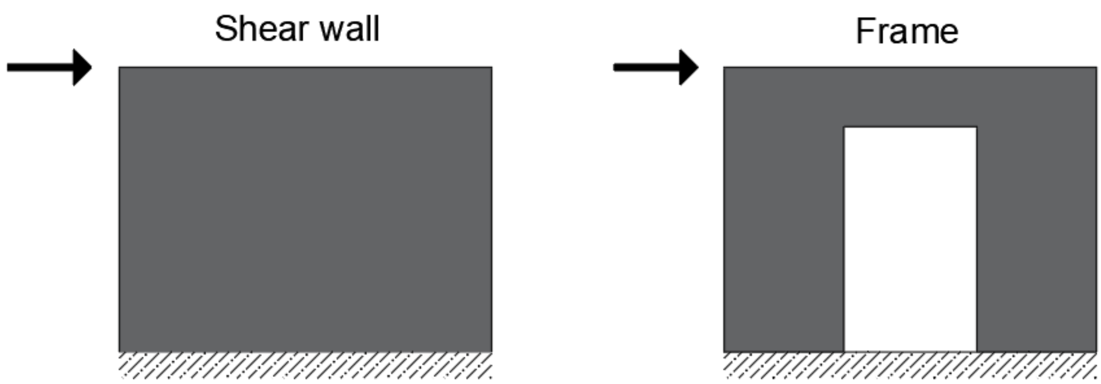


Figure 3.4: Example of a shear wall with hole can be considered as a frame.

The influence of openings in a shear wall will be investigated for some standard cases in chapter 4.6. The opening affects the stiffness and the horizontal load distribution can often be considered distributed according to stiffness. Thus, openings affect the distribution of the horizontal load to the different shear walls.

3.1.3 Shear centre

For a building, the shear centre is a result of the placement and geometry of the shear walls and if shear centre and load centre do not coincide, shear walls will be subjected to additional forces due to torsion. This will be described in more detail in section 3.2.2.1 [7].

3.1.4 Connection between diaphragm and shear wall

The connection between the diaphragm (slab) and shear wall must be able to transfer the forces between the diaphragm and the shear wall. The purpose of stabilising a building with shear walls relies on that this detail is executed well [7]. To connect shear walls and slabs for in situ cast structures, the elements are joined together by the cast-in reinforcement [2]. If a floor is made up by several prefabricated concrete elements, these concrete elements must be able to act as a single diaphragm, i.e. connecting the elements through reinforcement [5].

Prefabricated shear walls and slabs must be connected in order to resist shear forces and bending effects. The connection between the walls and slabs must be able to transfer loads to the foundation. The entire building needs to be anchored in the ground to resist the overturning forces [5].

3.1.5 Positioning of shear walls

In order to achieve a stable configuration by only using shear walls, there are certain criteria that need to be fulfilled. The first criterion is that shear walls must be arranged in a way that they can resist horizontal force in both y and x-direction, i.e. shear walls extend in both x and y-direction. The second criterion is that there must at least be three shear walls. The walls also must be able to build up a moment from the horizontal force component of the walls (from an arbitrary point in the plan of the structure). The previous sentence can be summarized in a way that the line of forces from the different walls have to meet in at least two points.

Figure 3.5 displays examples of stable configurations where the shear walls centre-line meet in at least two points.

Figure 3.6 displays examples of unstable configurations where the shear walls centre-lines meet in only one point.

If three shear walls are used, two of the shear walls should be placed perpendicular to the length direction of the building, i.e. on the gable side [4] (left configuration figure 3.5

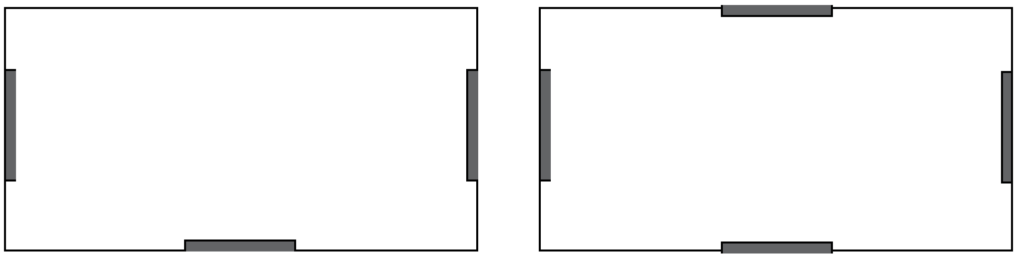


Figure 3.5: Stable configurations of shear walls.

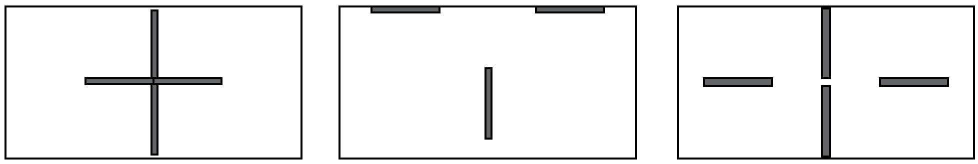


Figure 3.6: Unstable configurations of shear walls.

3.1.6 Loads

Loads can be divided into permanent loads G and variable loads Q (with respect to time). Self-weight and earth pressure count as permanent loads. Variable loads can be snow load, imposed load, wind load and traffic load. Imposed loads are different for different types of structures and for buildings, the imposed load can be viewed as the load from the people and furniture [2].

Wind load is an example of a dynamic load. However, wind load is most of the time considered a static load [2]. In general, the wind load that the biggest impact on a buildings stabilising system [4].

3.1.6.1 Load combinations

In general, a structure is often loaded by one more than one load. As variable loads are time dependent, it is unlikely for multiple variable loads to occur with their maximum value at the very same time. In order to deal with this, load combinations and partial coefficients are introduced [2]. Load combinations can in turn be divided into two different parts. Ultimate limit state and serviceabilility limit state [2].

Ultimate limit state consists of different limit state scenarios: EQU, STR, GEO and FAT. The limit state STR is the most common and is used to design structural elements [2]. EQU stands for equilibrium and that limit state is used when controls are made that the structure is not experiencing loss of static equilibrium. The ultimate limit state EQU is used when horizontal stability analysis is done for a structure. GEO is used for checking excessive deformations or failure in the foundation (underground). FAT is used for failure due to fatigue [2].

3.1.6.2 STR and GEO

The requirement for validating the limit state consisting of failure or large deformation, it is to be verified that [2]: $E_d \leq R_d$

Where:
E_d is the design value of the load effect
R_d is the design value of the load bearing capacity

3.1.6.3 EQU

Horizontal loads might cause overturning and sliding of a building and this has to be controlled. For these controls, usually, the building and its foundation are considered to form a rigid body. Verification that the building is stable for overturning loads is done by controlling that the self-weight of the building plus foundation is enough to counteract the overturning loads. If the case is that the self-weight from building plus foundation is not enough to counteract the overturning loads, this has to be solved by either increasing the self-weight or to anchor the foundation to the bedrock. Sliding is usually not a problem and this is controlled by verifying that the shear stress between foundation and soil does not exceed the soil resistance (shear strength or friction angle depending on what kind of soil) [7].

Validating the static equilibrium (overturning) with load combination EQU, a shear wall is considered as a rigid body. This gives the point the body is rotating around is positioned in the lower corner on the opposite side of the subjecting horizontal load. This is done with the assumption that the soil and rocks in the foundation are sufficient to provide capacity for resistance [8].

In figure 3.7 a plane shear wall with a concrete foundation is showcased being subjected to a horizontal load. The horizontal load causes overturning of the structure and sliding.

The requirement for validating the static equilibrium [8]:

$$E_{d,dst} \leq E_{d,stb} \tag{3.1}$$

Where:
$E_{d,dst}$ is the design value of the overturning moment
$E_{d,stb}$ is the design value of the stabilising moment

When it comes to stabilisation, there are both favourable and un-favourable loads that are considered. A load can be both favorable and unfavorable and this is the case for vertical loads. The vertical loads increase the stabilising moment (favorable) and also give rise to imperfection loads which causes overturning moment (unfavorable).

When doing the stabilisation calculations, the permanent vertical loads acting on the stabilising elements, i.e. shear walls shall be multiplied with 0.9 as these permanent

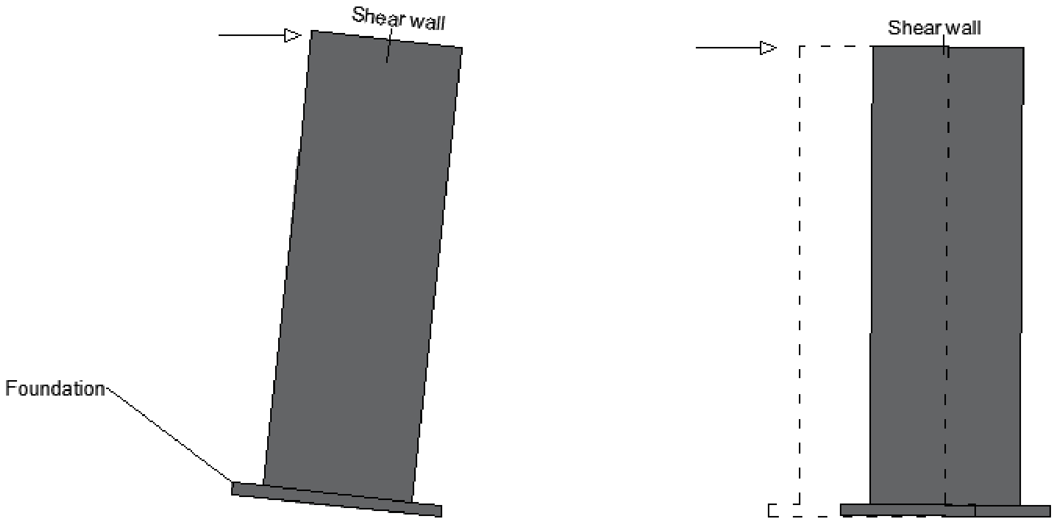


Figure 3.7: Structure has to be verified for overturning and sliding.

loads are favorable loads.

$$V_{d,EQU} = \gamma_d \cdot 0.9 \cdot G_k \tag{3.2}$$

γ_d is the safety class coefficient. γ_d is set to 1.0 for structural components where failure poses risk of severe consequences [2].

The characteristic wind load is multiplied with safety class coefficient γ_d and design load coefficient 1.5 to get the design value of wind load. The design wind load is then:

$$Q_d = \gamma_d \cdot 1.5 \cdot Q_k \tag{3.3}$$

3.1.6.4 Imperfection load

The imperfection load is a function of the vertical load. To get the design value of the imperfection load on each floor (only considering dead load), the vertical load is multiplied with 1.1 (vertical load is an unfavourable load in this case). This design value of the imperfection load is simply added together with the design wind load to get the design value of horizontal load.

When a building is constructed, it is impossible to construct it as perfect as the blueprints. For example, when walls and columns are erected/casted, it's impossible to obtain perfectly vertical elements. Inclined vertical elements give rise to a horizontal force component. This horizontal force component gives a contribution of the horizontal load that the stabilising system has to be able to withstand in addition the wind load. The imperfection load is applied in design in ultimate limit state for both persistent and accidental load scenarios [9]. In comparison to the horizontal load from wind load, the imperfection load is relative small.

Imperfections can be represented by an inclination θ_i. This inclination θ_i (rad) can be calculated using equation 3.4 [9].

$$\theta_i = \theta_0 \cdot \alpha_h \cdot \alpha_m \tag{3.4}$$

Where:
θ_0 has the recommended value 1/200
α_h is the reduction factor for length or height and is calculated with by equation $\alpha_h = 2/\sqrt{l}$ and $2/3 \leq \alpha_h \leq 1$
α_m is the reduction factor for number of members and is calculated by equation $\alpha_m = \sqrt{0.5(1 + 1/m)}$
l is the length of the member or height of the structure and m is the number of vertical members contributing to the total effect

Eurocode illustrates imperfection load as seen in Figure 3.8.

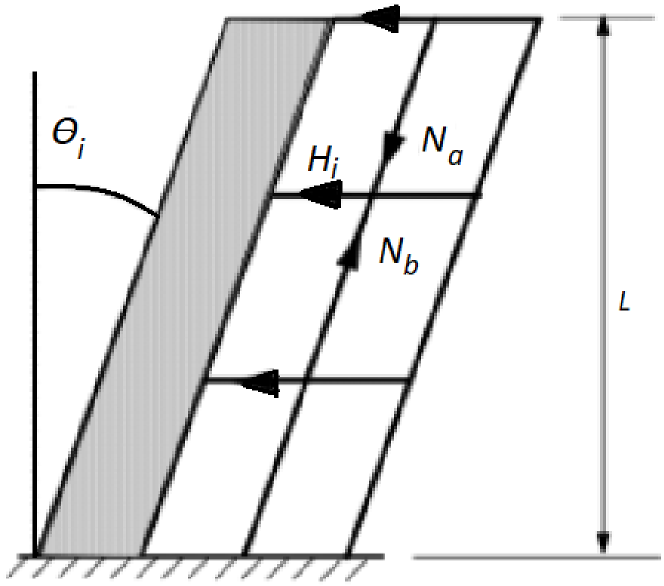


Figure 3.8: Imperfection load on the stabilising system (Source: [9], edited).

For low values of θ_i, $tan(\theta_i) \approx \theta_i$. θ_i is used in equation 3.5 to calculate the horizontal force H_i on each floor

$$H_i = \theta(N_b - N_a) \tag{3.5}$$

As it is of interest to see the effect of imperfection load when looking at the stabilising system, it is stated in the Eurocodes that l is the height of the building and m is the number of vertical members (columns) contributing to the horizontal force on the bracing system [9]. That means that if the horizontal force from imperfection load is considered on a slab that has n columns above and n columns below, the total amount of columns that contribute to the imperfection load is thus $2n$. This gives that for the top floor, $m = n$ and rest of the floors $m = 2n$.

The loads N_a and N_b in figure 3.8 are the vertical loads. The vertical load N_a and N_b always consist of self-weight but depending on the load combination, the vertical load may also include imposed load and snow-load. H_i is the resulting horizontal force transferred to the bracing system.

The vertical loads that subject the stabilising units (shear walls) have a positive effect on the system in the way that they counteract the rotation from the horizontal load (wind, imperfection load). However, the imperfection load increases with increasing vertical load.

Two different verifications have to be made with the imperfection load. The first is checking the stability of the building and using load combination EQU. The other is an ULS check of the stabilisation system, that is controlling the capacity of the lateral resisting structural elements.

Several load combinations have to be calculated when doing stability calculations for a building. The reason is that the very same load can have positive effect and negative effect on the system at the same time. The different load combinations for the building when doing stability calculations are the following:

1. Load combination 1 EQU: Permanent load + Wind load + Imperfection loads
2. Load combination 2 EQU: Permanent load + Imposed load + Wind load + Imperfection load
3. Load combination 3 EQU: Permanent load + Imposed load + Snow load + Wind load + Imperfection load

Consider an example with a building that has the width W and the depth D. the building has n columns, the building has the height L. The imperfection load H_i is to be calculated on each floor/roof i. For simplicity, the self-weight G_k is the same for the floors and the roof. Only vertical load from self-weight is included.

A plan section and horizontal section can be seen in figure 3.9.

The vertical load that causes imperfection load is multiplied with 1.1 as it is a negative load effect on the stabilising system.

The vertical loads [kN/m] are calculated:
$V_{5ö} = V_{4u} = 1.1 \cdot g_k \cdot D$
$V_{4ö} = V_{3u} = 1.1 \cdot g_k \cdot D + V_{4_u}$
$V_{3ö} = V_{2u} = 1.1 \cdot g_k \cdot D + V_{3_u}$
$V_{2ö} = V_{1u} = 1.1 \cdot g_k \cdot D + V_{2_u}$

For the roof and ground floor, $n = m$. For the rest of the floors, $2n = m$ since the columns above the floor and below contribute to the imperfection load. According to equation 3.4 this gives that that $\theta_{5,1} = \frac{1}{200} \cdot \frac{2}{\sqrt{l}} \cdot \sqrt{0.5(1 + 1/n)}$. For the rest of the floors this gives that $\theta_{2-4} = \frac{1}{200} \cdot \frac{2}{\sqrt{L}} \cdot \sqrt{0.5(1 + 1/2n)}$.

The horizontal forces H_i can now be calculated for the roof and the floors using equation 3.4:

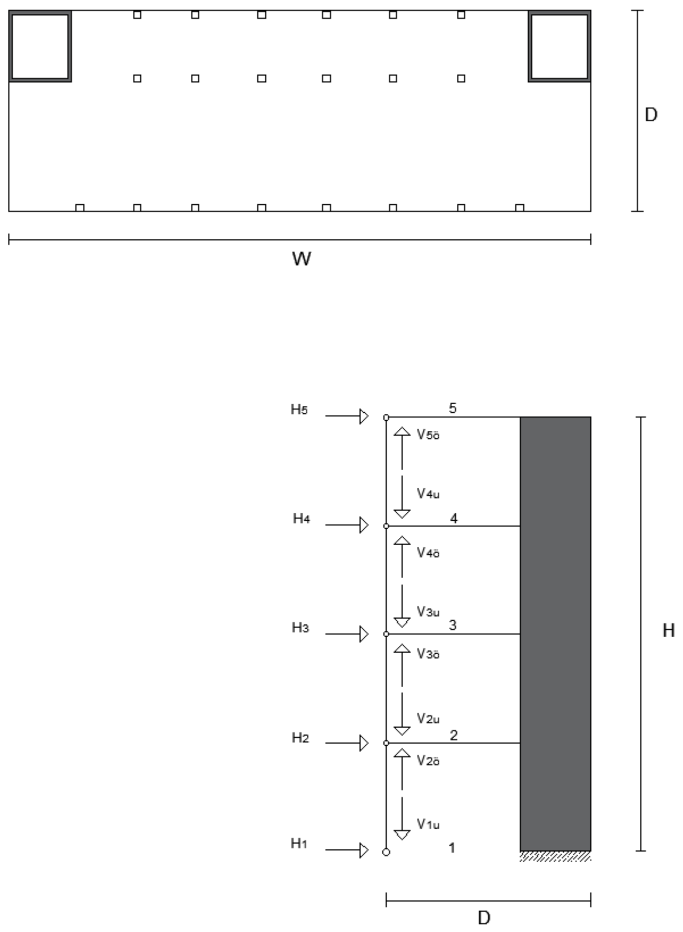


Figure 3.9: Plan view and horizontal section of example building. The two squares are stabilising cores.

$H_5 = V_{5\ddot{o}} \cdot \theta_{5,1}$
$H_4 = \theta_{2-4}(V_{4\ddot{o}} - V_{4u})$
$H_3 = \theta_{2-4}(V_{3\ddot{o}} - V_{3u})$
$H_2 = \theta_{2-4}(V_{2\ddot{o}} - V_{2u})$
$H_1 = \theta_{5,1} \cdot V_{2\ddot{o}}$

These imperfection loads H_i are then added together with the corresponding wind load.

3.2 Distribution of horizontal loads

3.2.1 Wind load distribution to diaphragm

The wind load over the height of the building is transferred to the shear walls through the slabs/roof. With a simplified model, each segment of wall between two diaphragms can be modelled as a simply supported beam with the diaphragms as supports. With a uniform load over the height of the building, the load will be distributed with half load to each diaphragm.

The top floor (floor 5 in figure 3.10) diaphragm is loaded with the upper half of the wind load acting over the wall between said floor and the one below. The second diaphragm from the top is loaded with the lower half the loaded wall above it, and half the upper part of the loaded wall below it. Figure 3.10 presents the load in each diaphragm over a building with 5 floors, where the wind load is split up into influence heights for each floor, where the wind load then can be integrated over the height for the total wind load per diaphragm.

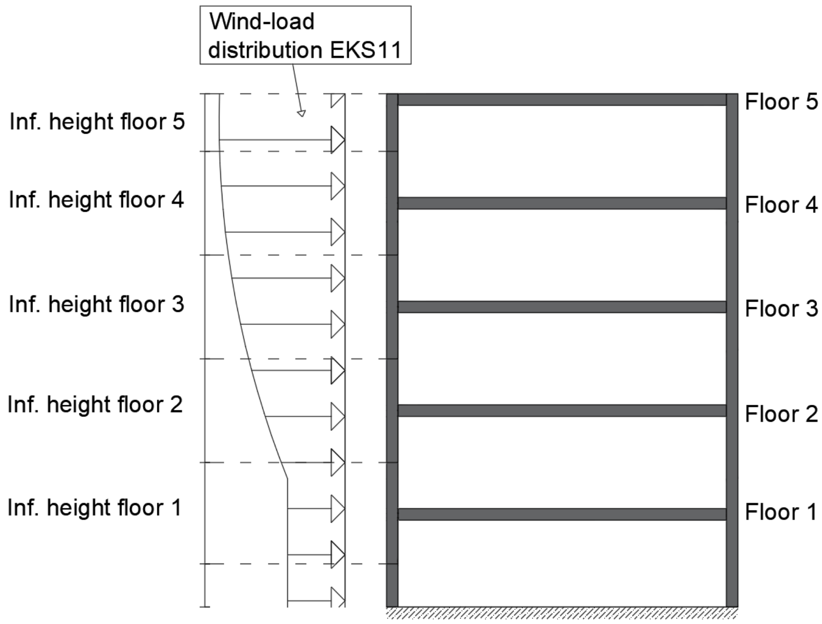


Figure 3.10: Building with influence height of wind load to each diaphragm marked with dotted lines, where the wind load follow the distributions suggested by EKS11 [10].

3.2.2 Distribution of horizontal loads to shear walls

The horizontal load is distributed over the shear walls based on the stiffness of the walls in relation to the concrete slab transferring the load [5]. If there are only two shear walls, the load distribution can be determined using the equilibrium equations (static determinate structure). Equation 3.6 is used for comparing the stiffness between shear walls and the concrete slab.

$$C = \frac{S_{floorslab}}{S_{shearwall}} \tag{3.6}$$

The relative stiffness ratio C is the inverse of the resulting deformation from a unit load of 1 N on the floor slab and shear wall according to figure 3.11.

There are three different load distribution models to use depending on the stiffness ratio between the shear walls and the slab [5].

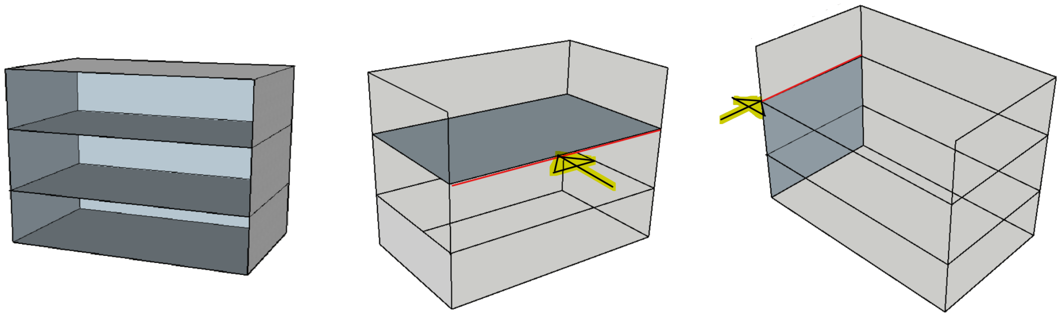


Figure 3.11: Example building explaining calculations for relative stiffness of the second floor. Left: Showing all shear walls and floor slab. Middle: Showing floor slab with unit load. Right: Showing shear wall with unit load.

In general, the shear walls are less stiff than the slabs [7].

The shear wall deformation is calculated by applying a unit load to a wall being fully fixed to the ground. The deformation includes both bending deformation and shear deformation. The deformation is calculated with equation 3.7.

$$\delta = \frac{1 \cdot H}{E_m \cdot t \cdot h} \left(\frac{4 \cdot H^2}{h^2} + 3 \right) \tag{3.7}$$

The stiffness for the shear wall is found in equation 3.8.

$$S_{wall} = \frac{1}{\delta} \tag{3.8}$$

Where:
S_w is the stiffness of shear wall [N/m]
H is height of shear wall
E_m is Young's modulus
t is the thickness of shear wall
h is the cross sectional height for shear wall

An illustration of the model used to calculate the deformation for shear wall can be seen in figure 3.12.

For a diaphragm (slab/roof) with rectangular cross section, the deformation for a unit load is calculated with equation 3.9. To calculate the deformation, a simply supported beam model is used where the deformation is calculated using length L where L is the longest length of a span between two shear walls. The deformation δ involves both bending deformation and shear deformation [7].

$$\delta = \frac{1 \cdot L}{E_b \cdot t_b \cdot h_b} \left(\frac{L^2}{4 \cdot h_b^2} + \frac{3}{4} \right) \tag{3.9}$$

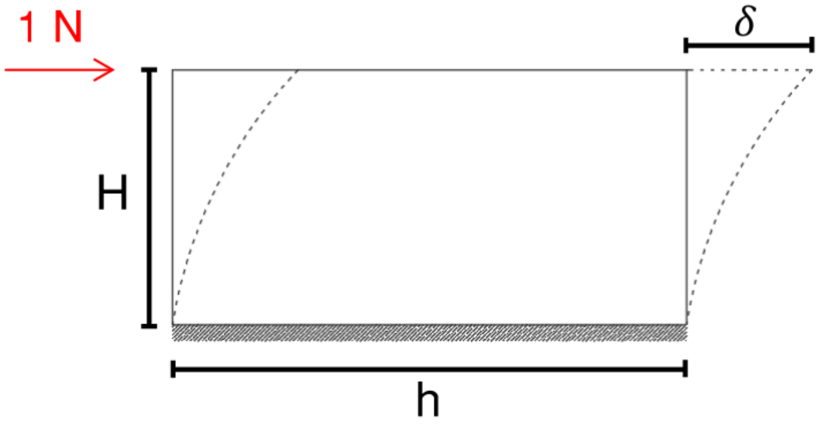


Figure 3.12: Load model used to calculate deformation of shear wall. Unit load of 1 N subjects the wall.

The stiffness for the diaphragm is calculated with equation 3.10

$$S_{diaphragm} = \frac{1}{\delta} \tag{3.10}$$

Where:
$S_diaphragm$ is the stiffness of the diaphragm [N/m]
L is the span length
E_b is Young's modulus of the diaphragm
t_b is the thickness of the diaphragm
h_b is the cross sectional height of the diaphragm

An illustration of the model used to calculate the deformation for the diaphragm can be seen in figure 3.13.

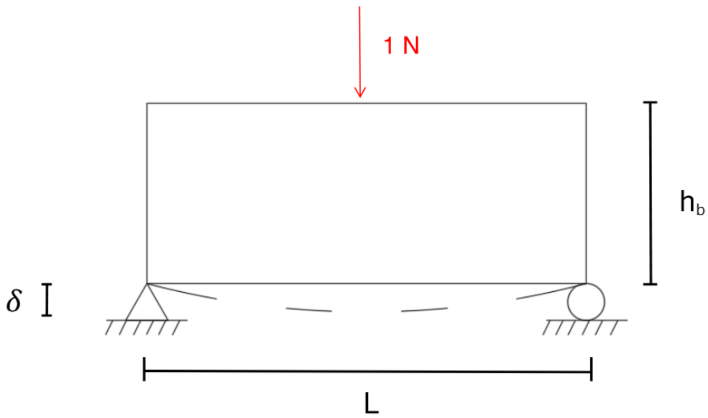


Figure 3.13: Load model used to calculate deformation of diaphragm. Unit load of 1 N.

3.2.2.1 Horizontal load distribution for stiff diaphragm

When the stiffness ratio $C > 10$, that means the slab is stiffer than shear walls. It is therefore presumed that the slab remains undeformed under loading. That results in that the load is divided proportionally to each shear wall according to their respective stiffness. For this load-distribution model, torsion needs to be considered [5]. Figure 3.14 shows how the horizontal load distribution is modelled (elastic springs) when diaphragm is stiffer than shear walls.

In general, for multi-storey buildings, the diaphragm is much stiffer than the shear walls and thus horizontal load distribution based on shear wall relative stiffness (stiff diaphragm) is valid. The lower floor in a building, the stiffer the shear walls are compared to the diaphragm. For that reason, it is enough to verify that the lowest floor in a building has a stiff diaphragm compared to the shear walls ($C \geq 10$) to know that load distribution based on shear wall relative stiffness is valid on all floors in the building [7].

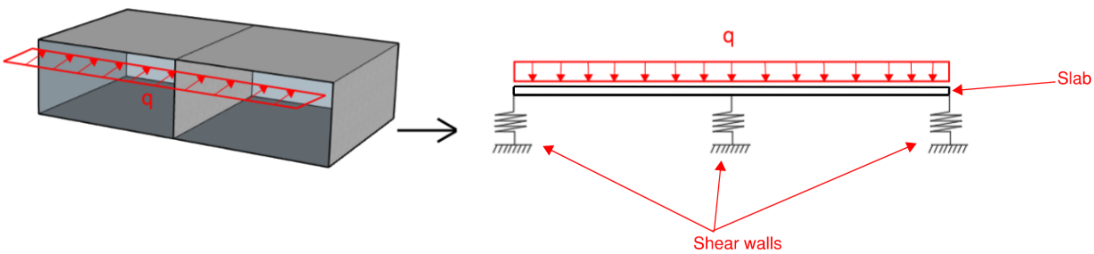


Figure 3.14: For $C > 10$ the load is distributed depending on the stiffness of wall. q denotes a uniformly distributed horizontal load acting on slab.

The positioning of load centre LC relative to the shear center SC determines the torsion the shear walls are subjected to. The torsional moment is calculated with equation 3.11

$$M = Q \cdot e \tag{3.11}$$

Where:
Q is the resulting horizontal force acting on the diaphragm (slab) in question
e is the distance between load centre and shear centre

For a building with double symmetrical placement of shears walls, the building won't experience torsion for symmetric loading (SC and LC coincide). A building must also be verified for asymmetric wind load and this will cause torsion even though the shear walls are placed double symmetric. Torsion results in additional horizontal force which the shear walls have to resist. The additional force due to torsion considers direction so it can both lead to additional loading and unloading of the shear walls [7].

For an uniformly distributed load, the load centre is in the middle of that load.

For an asymmetric load that is linearly increasing from value 0, the shape of the load forms a triangle. For this, the load centre LC is located 2/3 of the distance from where the load starts (value of 0) to its end.

To calculate the distance e_x in x-direction between load centre and shear centre for a load in y-direction, equation 3.12 is used [7].

$$e_x = x_{ref}LC - x_{ref}SC = x_{ref}LC - \frac{\sum S_{yi} \cdot x_{refi}}{\sum S_{yi}} \tag{3.12}$$

Where:
x_{refLC} is the distance in x-direction to the load centre from reference point
x_{refi} is the distance in x-direction to shear wall i from reference point
S_{yi} is the relative stiffness for shear wall i [N/m]

Figure 3.15 shows the plane geometry of a building being subjected by uniformly distributed load in the y-direction. The load is resisted by shear walls and the shear centre SC and load centre LC do not coincide, where distance between SC and LC is e_x.

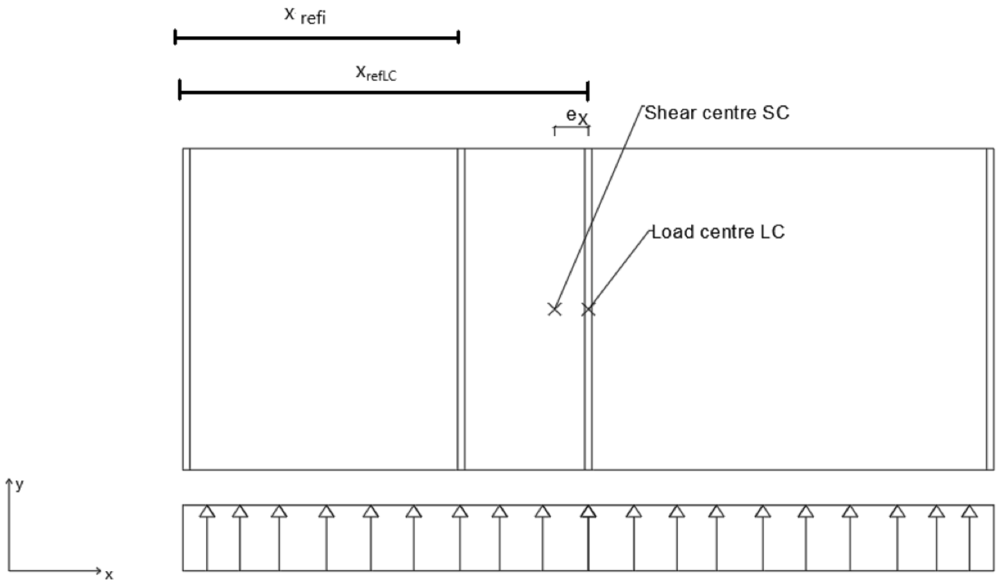


Figure 3.15: Plane geometry with shear centre and load centre marked. Load in y-direction.

For a load in the y-direction the force on shear wall i is calculated according to equation 3.13:

$$F_{yQi} = \frac{Q_y \cdot S_{yi}}{\sum S_{yi}} \tag{3.13}$$

Where:
F_{yQ_i} is the force on shear wall i
Q_y is the resulting force from the wind load
S_{yi} is the relative stiffness of wall i [N/m]

The torsional moment from the wind load, M_y, is divided proportionally to wall i relative stiffness S_{yi} and the sum of the relative stiffness for all shear walls (both x and y-direction). The torsional moment is also divided according to the distance x_i for shear wall i to shear centre SC.

To calculate the force from the torsional moment M_y on each shear wall i, orientated in the y-direction, equation is used 3.14.

$$F_{yMi} = \frac{M_y \cdot x_i \cdot S_{yi}}{\sum(S_{yi}x_i^2 + S_{xi} \cdot y_i^2)} = \frac{Q_y \cdot e_x \cdot x_i \cdot S_{yi}}{\sum(S_{yi}x_i^2 + S_{xi} \cdot y_i^2)} \tag{3.14}$$

When calculating the additional force on shear wall i due to torsion according to equation 3.14, e_x and x_i is with respect to direction where the shear centre is the "new" origo.

Equation 3.13 can now be combined with equation 3.14 to have an equation for the total force on wall i that is subjected due to wind load in y-direction and torsion:

$$F_{yi} = (\frac{Q_y}{\sum S_{yi}} + \frac{Q_y \cdot e_x \cdot x_i}{\sum(S_{yi}x_i^2 + S_{xi} \cdot y_i^2)}) \cdot S_{yi} \tag{3.15}$$

Where:
F_{yi} is the force on wall i from the wind load in y-direction from Q_y and torsional moment M_y
S_{yi} is the relative stiffness for wall i with regard to load in the top of the wall [N/m]
x_i is the distance in x-direction from shear centre to wall with respect to direction x
y_i is the distance in y-direction from shear centre to wall with respect to direction y

This analysis must be done for positive and negative x direction, as well as positive and negative y direction.

3.2.2.2 Horizontal load distribution for equal stiff diaphragm and shear walls

When the stiffness ratio C is between 5 and 10, the load is distributed to the closest support as seen in figure 3.16. No torsional moment needs to be considered for this load model [5].

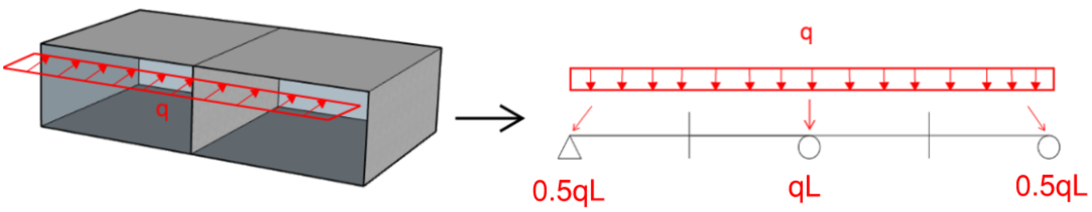


Figure 3.16: For C between 5 and 10, the load is distributed to the closest support.

3.2.2.3 Horizontal load distribution for stiff shear walls

When the stiffness ratio is $C < 5$, that means the slab is not as stiff as the shear walls. For this the load distribution is modelled as for a continuous beam. No torsional moment needs to be considered. The reaction forces are calculated as for an elastic beam. Figure 3.17 shows how the horizontal load distribution is modelled [5].

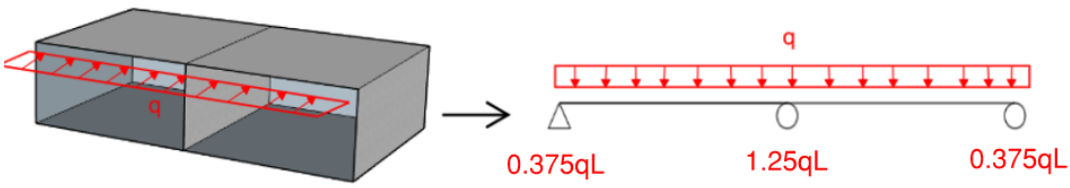


Figure 3.17: For $C < 5$ the load is distributed as a continuous elastic beam.

3.2.2.4 Horizontal load distribution according to EN 1992-1-1-2005

EN 1992-1-1-2005 states the following in regard to shear walls:

1. Shear walls are plain or reinforced concrete walls which contribute to the lateral stability of the structure.
2. Lateral load resisted by each shear wall in a structure should be obtained from a global analysis of the structure, taking into account the applied loads, the eccentricities of the loads with respect to the shear centre of the structure and the interaction between the different structural walls.
3. The effects of asymmetry of wind loading should be considered.
4. The combined effects of axial loading and shear should be considered.
5. In addition to other serviceability critera in this code, the effect of sway of shear walls on the occupants of the structure should also be considered (see EN 1990).
6. If members with and without significant shear deformations are combined in the bracing system, the analysis should take into account both shear and flexural deformation.

Eurocode gives a method of calculating the distribution of horizontal loads to shear walls for a *reasonably symmetrical layout* with less than 25 storeys and shear deformation being negligible. It suggests the loading in each shear wall from horizontal-forces are distributed based on the stiffness EI of single wall n compared to the total stiffness of all walls [9].

A second contribution comes from torsion in cases when the load is not located in the centroid of shear wall group (marked as [A] on figure 3.18). When this is the case, the load is distributed based on the eccentricity e, the distance y_n from wall n to the centroid of stiffness, and again stiffness EI of wall n. The equation is presented in equation 3.16 [9].

$$P_n = \frac{P(EI)_n}{\sum(EI)} \pm \frac{(Pe)y_n(EI)_n}{\sum(EI)y_n^2} \tag{3.16}$$

The difference between equation 3.16 and equation 3.15 is that the latter equation considers both bending and shear stiffness calculation and the equation from Eurocode only considers bending stiffness.

Figure 3.18 explains the variables used in equation 3.16, and also gives an example of how a *reasonably symmetrical layout* can look.

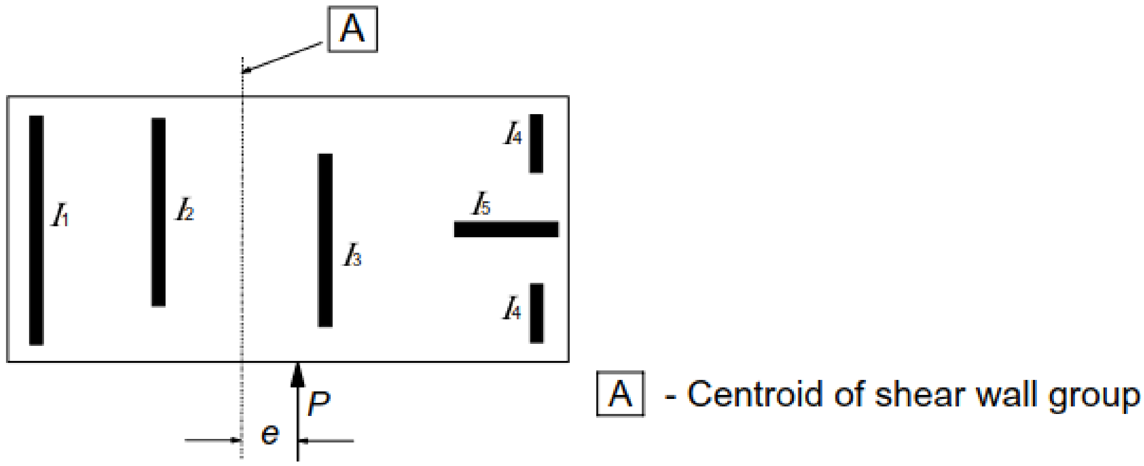


Figure 3.18: Shear walls marked with thick black lines in a building, with load P, centroid of shear wall group *[A]* and eccentricity e displayed (Source: [9]).

3.3 Shear and bending deformations

For a cantilever wall, the H/h ratio plays a significant role if whether shear deformation or bending deformation is the prevalent deformation for a load, with H being the wall height and h being the wall length according to Figure 3.12 [7]:

The deformation for a cantilever wall can be divided into bending and shear deformation according to equation 3.17.

$$\delta = \frac{P \cdot H^3}{3 \cdot E_m \cdot I_m} + \frac{P \cdot 1.2 \cdot H}{0.4 \cdot E_m \cdot A_m} \tag{3.17}$$

Where:
δ is the deformation P is the load

H is height of shear wall
E_m is Young's modulus
I_m is the moment of inertia
A_m is the cross sectional area
P is the load

In equation 3.17, the first term is the bending deformation and the second term is the shear deformation.

The contribution of bending and shear deformation to the total deformation can be seen in figure 3.19

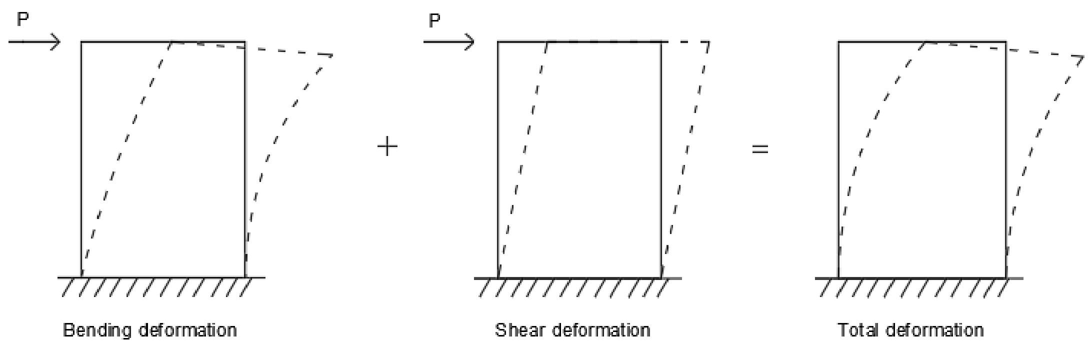


Figure 3.19: Shear and bending deformation contribution to the total deformation of a cantilever wall.

Equation 3.17 is reshuffled to express how large the shear deformation is for a cantilever wall considering the total deformation. The reshuffled expression is shown graphically for increasing H/h values, i.e. cantilever wall becoming more slender, see Figure 3.20. In Figure 3.20 h is the length of the wall.

The shear deformation's part of the total deformation is large for low ratios H/h, i.e. long walls. This means that the shear deformation plays a significant role in the relative stiffness.

This can be translated to that for the lower storeys of a building that is stabilised with shear walls, to get the correct assessment of how the horizontal load is distributed to the shear walls on that floor, the shear deformation needs to be included.

According to *Bärande tegelmurverk* [7], for $H/h \leq 0.2$ the bending deformation can be excluded in the calculation of the relative stiffness. For $H/h \geq 8$, the shear deformation can be excluded in the calculation of the relative stiffness.

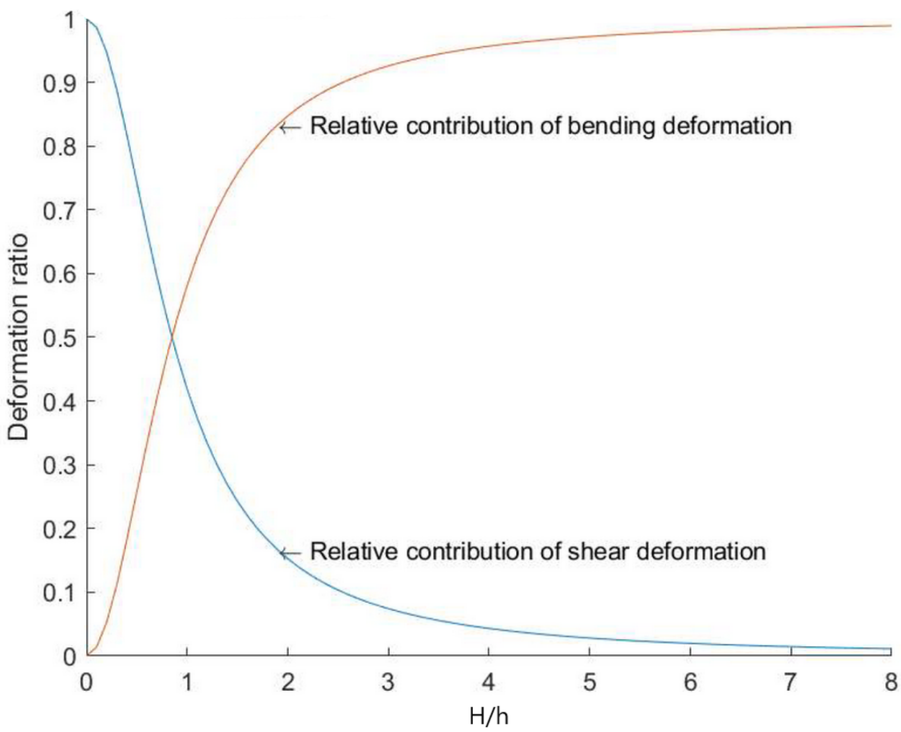


Figure 3.20: Relative contribution of shear and bending deformation to total deformation for a cantilever wall. H/h is the ratio between height H and length h of the cantilever wall.

3.4 Wind load

Wind load is a variable load with the unit kN/m^2. Wind load acts perpendicular to the exterior of the surface for a structure. Inner wind pressure also occurs to the actual surface which has to be considered when designing the exterior facade element. Wind load increases with height. For normal cases, the wind load is often calculated with the reference height of the building z (height of the building) and assuming this load is evenly distributed over the entire height of the building [2].

During storms, wind can take on high velocities during short intervals, but buildings have sufficient natural damping to handle short term dynamic loads in general. For that reason, the wind-pressure used for design is taken as a equivalent static load based on average wind velocities [2].

Among many factors affecting the wind load, the significant factors are the location of the building, topography, height z of the building and the shape of the building (width and height) [2].

For a slender and tall building where the eigenfrequency has a significant effect on the wind pressure, the dynamic effects should be considered [10].

3.4.1 Wind load over the height of the building

With wind load increasing over the height of the building, the wind load transferred to each diaphragm increase with floor height.

The Eurocode suggests a method where wind is split up in zones with constant wind pressure between the intervals. EKS11 suggests an exponential relation of wind load between the top and height z_{min} depending on the terrain category.

For both methods, the wind load is constant up until the height z_{min}. See figure 3.21 for illustration of height z_{min} and how the wind load increases with height according to EKS11 [10].

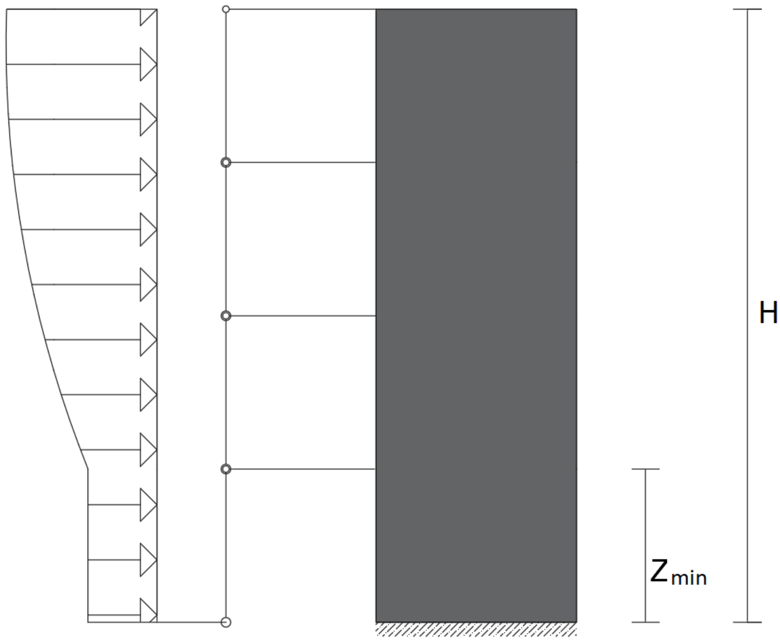


Figure 3.21: EKS11 wind load distribution presented (not to scale).

3.4.2 Wind load over the width of the building

The distribution of wind load over the width of the building can have two different configurations that can result in the worst case scenario. These are symmetric wind load and asymmetric wind load.

For the largest load on the building, the wind load is assumed to be placed evenly over the width (symmetric wind load). This gives that the load centre placed is in the centre of the buildings windward side, see figure 3.22.

For the largest rotational component, the wind load is placed with a linear relation over the width of the building (asymmetric wind load). The rotational component comes from the shear centre and the load centre not coinciding. The further away they are the larger the rotational moment will be. For the asymmetric wind load, the load centre will be one third from the largest end, and asymmetric load is placed for the longest lever-arm, see figure 3.23.

Each wind load must also be calculated from the opposite side to determine the largest load into each wall [7].

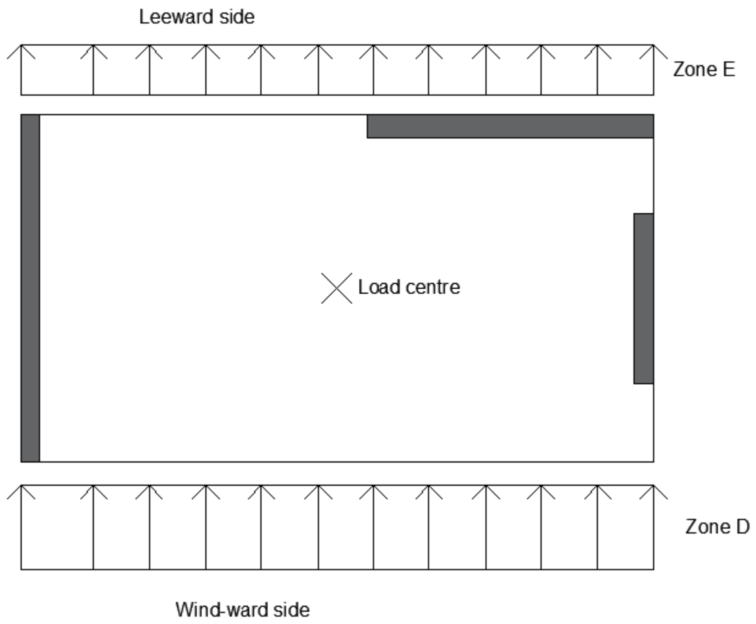


Figure 3.22: Symmetric wind load over the width of a building. Grey marked are shear walls.

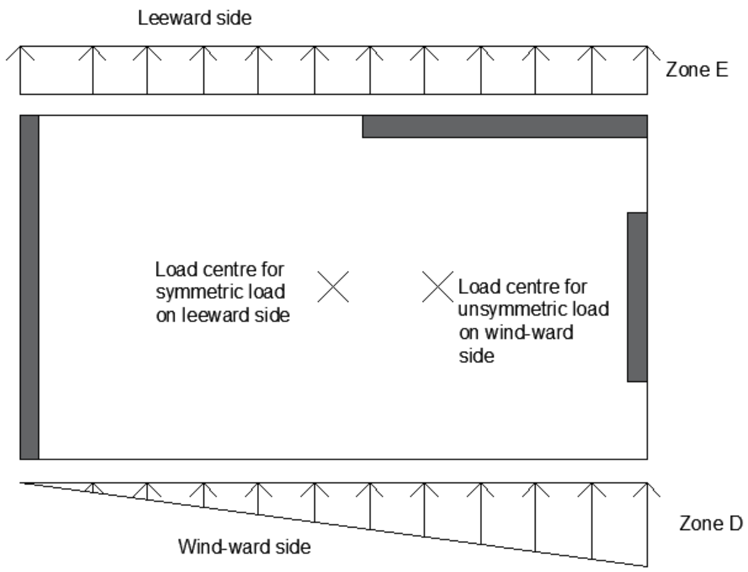


Figure 3.23: Asymmetric wind load on windward side and symmetric wind load on leeward side. Grey marked are shear walls.

3.5 Overturning moment and stabilising moment

For a building to be stable, the stabilising elements, i.e. shear walls, need to have the design value of the stabilising moment to be larger than the design value of the overturning moment [7].

The overturning moment M_0 is calculated for each wall by multiplying the horizontal force on the shear wall for each floor i with the height from foundation up to that floor:

$$M_0 = \sum_{i=1}^{n_{floors}} P_i \cdot z_i \tag{3.18}$$

Where:
P_i is the horizontal force on floor i
z_i is the height from foundation to floor i

The stabilising force is the resultant self-weight force. It is assumed that the vertical force on the wall element is evenly distributed over the wall, resulting in a lever arm of half the wall length. The stabilising moment is calculated with equation 3.19.

$$M_{stab} = \sum_{i=1}^{n_{floors}} N_i \cdot e = 0 \tag{3.19}$$

Where:
N_i is the vertical force on the wall on floor i
e is half the length of the wall

Figure 3.24 further explains how these loads can be placed.

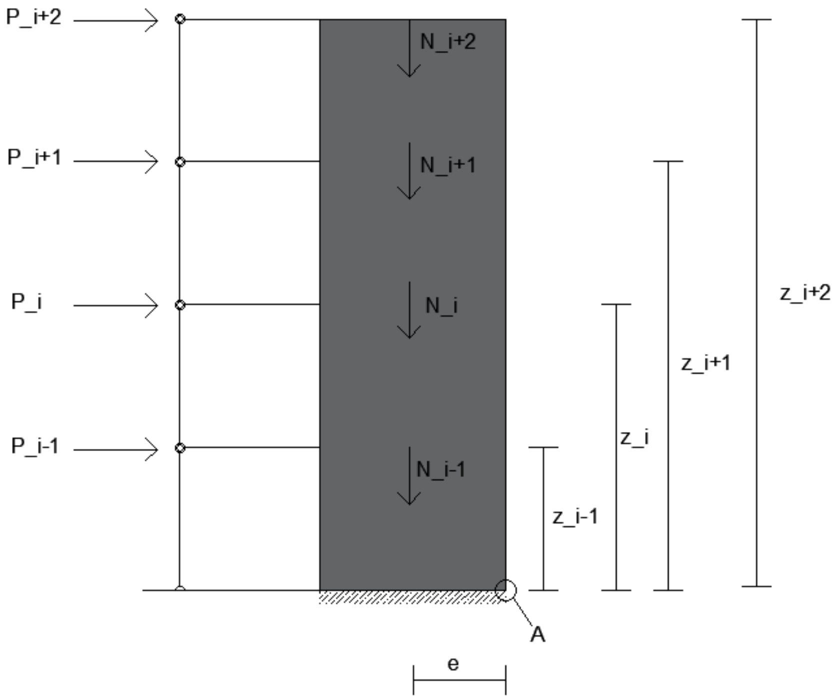


Figure 3.24: Figure of horizontal loads P and vertical loads N acting on shear wall when considering overturning moment.

4 Method

4.1 Horizontal load distribution method

In accordance to what is written under section 3.3, the horizontal load distribution in Dynamo is considering both bending and shear deformation of the shear walls. This is done to get an accurate view of how the horizontal load is distributed among the shear walls on each floor.

It is common to assume that a slab acts as a stiff diaphragm. In the Dynamo script the horizontal load distribution to the shear walls is based on relative stiffness for three or more walls, i.e. considering the slab as a stiff diaphragm. For two shear walls, the horizontal load distribution is based on beam model.

4.2 Modelling of wind load

Wind load is explained in short terms in this chapter. A more detailed explanations is presented in the appendix.

The peak velocity pressure $q_p(z)$ at the height z is decided though equation 4.1 from Swedish national document EKS11 [10].

$$q_p(z) = [1 + 2 \cdot k_p \cdot I_v(z)] \cdot [k_r \cdot \ln{(z/z_o)} \cdot c_o(z)]^2 \cdot q_b \tag{4.1}$$

Equation 4.1 follow a graph given from EKS11 for cases where the building topography and natural frequencies are not taken into account (see Figure 4.1). The calculated values from equation 4.1 will give same values as Figure 4.1, since simplifications have been made regarding topography and dynamic effects.

Equation 4.1 is long and tedious, and will therefore be described in deeper detail in appendix.

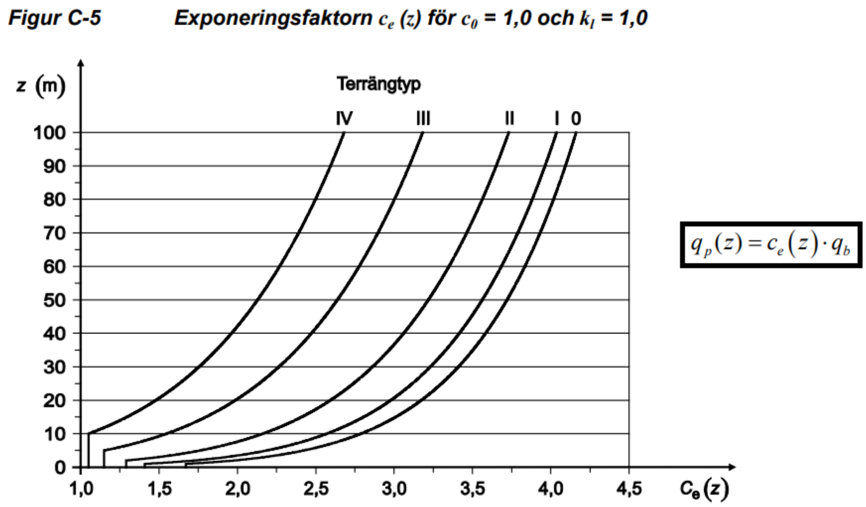


Figure 4.1: EKS11 graph for exposure factor, with added equation for wind pressure $q_p(z)$ (black rectangle) (Source: [10], edited).

The load w_e for the outer wind pressure is described with equation 4.2 [11].

$$w_e = q_p(z_e) \cdot c_{pe} \tag{4.2}$$

Where c_{pe} is the form factor. Table 4.1 shows the table in which coefficients are taken from, based on the height (H) to depth (d) ratio. In case of a $H/d > 5$, Eurocode introduces another method based on shape of corners and edges of the building. As this leads to a large amount of assumptions being made, it is deemed sufficient to apply values found in table 4.1 for the interval $h/d > 5$ by extending the interpolation based the data between $H/d > 1$ and $H/d < 5$. The equations derived from interpolation presented in equation 4.3 to 4.10.

Table 4.1: Values for form-factor c_{pe} based on building zones (see more in appendix figure 7.6). Source: [11]

Zone	A		B		C		D		E	
h/d	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
5	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,7	
1	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,5	
≤ 0,25	-1,2	-1,4	-0,8	-1,1	-0,5		+0,7	+1,0	-0,3	

$$c_{pe,D} = -0.7 \text{ (for } h/d \leq 0.25) \tag{4.3}$$

$$c_{pe,E} = -0.3 \text{ (for } h/d \leq 0.25) \tag{4.4}$$

$$c_{pe,D} = -0.8 \text{ (for } h/d > 0.25 \text{ and } h/d \leq 1) \tag{4.5}$$

$$c_{pe,E} = -h/d \cdot 0.267 - 0.233 \text{ (for } h/d > 0.25 \text{ and } h/d \leq 1) \tag{4.6}$$

$$c_{pe,D} = -0.8 \text{ (for } h/d > 1 \text{ and } h/d \leq 5) \tag{4.7}$$

$$c_{pe,E} = -h/d \cdot 0.05 - 0.45 \text{ (for } h/d > 1 \text{ and } h/d \leq 5) \tag{4.8}$$

$$c_{pe,D} = -0.8 \text{ (for } h/d > 5) \tag{4.9}$$

$$c_{pe,E} = -h/d \cdot 0.05 - 0.45 \text{ (for } h/d > 5) \tag{4.10}$$

z_{min} is a height interval depending on the terrain category. Table 4.2 presents values of z_{min} for the different terrains.

Table 4.2: Minimum height for different terrain categories.

Terrain category		z_0 m	z_{min} m
0	Sea or coastal area exposed to the open sea	0,003	1
I	Lakes or flat and horizontal area with negligible vegetation and without obstacles	0,01	1
II	Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights	0,05	2
III	Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)	0,3	5
IV	Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m	1,0	10
NOTE: The terrain categories are illustrated in A.1.			

Over the height of the building, the wind load is simplified into a evenly distributed load between each floor in the script. Each floor slab takes wind load equal to half the wind load from the floor above, and half the wind load from the floor below. The wind load is simplified into segments with uniform distributed load in each segment (see figure 4.2). These segments are based on the wind calculations from the EKS11.

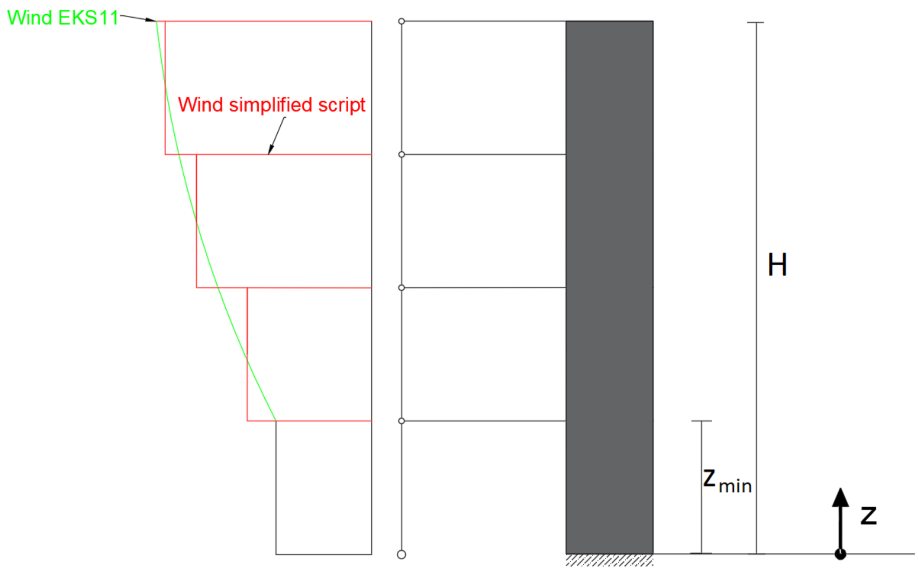


Figure 4.2: EKS11 wind load distribution (green line) compared to the simplified approach in the Dynamo script (red blocks).

4.3 Modelling of asymmetric wind load

As been described in section 3.4.2, a structure must be verified for asymmetric wind load subjecting the structure on the windward side (zone D). Simultaneously, there is a symmetric wind load on the leeward side (zone E).

Previously, when calculating the magnitude of symmetric wind load, the wind load on windward side and on leeward side can be added together and be considered as one "single" load on only one side. This works if the load centre coincides for the wind load on windward and leeward side.

For asymmetric wind load, there is a asymmetric wind load on windward side (zone D) and uniform wind load on leeward side (zone E). These two loads do not have their load centre coinciding and the loads cannot simply be considered as one single load. See figure 4.3 for illustration of the asymmetric and symmetric wind load acting together. In Dynamo, scripting is facilitated if the loads on windward side and leeward side can be considered together and for this, an expression is derived which calculates the load centre for a rectangular building that is subjected to asymmmetric load and symmetric wind load simultaneously.

The idea is that the resulting load from the asymmetrical load and uniform load are in moment equilibrium around the combined load centre from the two loads. See figure 4.4, where LC^{*} is the common load centre, the moment equilibrium is calculated around.

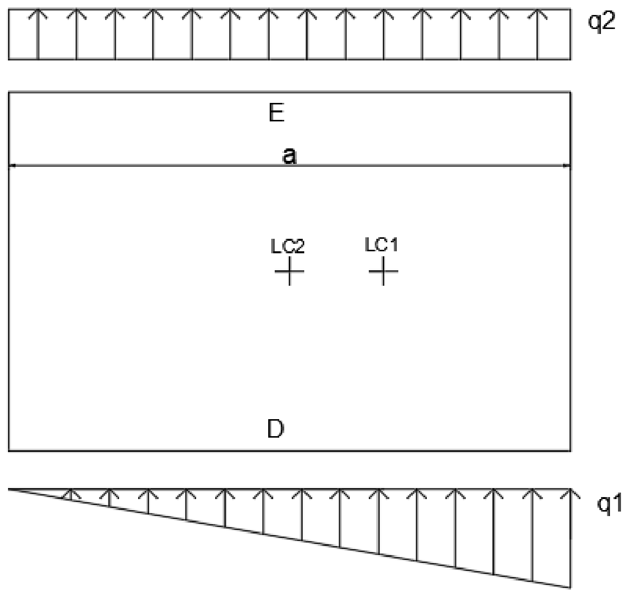


Figure 4.3: Structure subjected to asymmetric wind load and symmetric wind load.

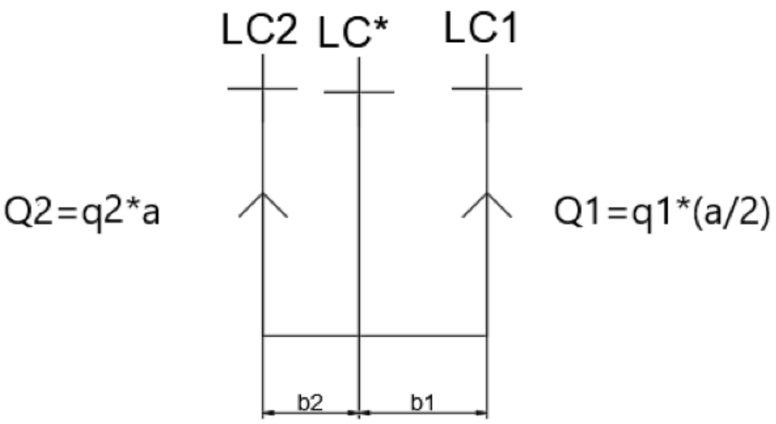


Figure 4.4: Figure used to illustrate the moment equilibrium and the combined load centre LC*.

An expression for the location of the new combined load centre is derived. The moment equilibrium can be expressed as following:

$$Q2 \cdot b2 - Q2 \cdot b1 = 0 \tag{4.11}$$

Where:

$$Q2 = q2 \cdot a \tag{4.12}$$

and

$$Q1 = q1 \cdot \frac{a}{2} \tag{4.13}$$

Where:
a is the width of the structure that is subjected by the loads
$b1$ is the distance from the combined load centre LC^* to load centre from asymmetric

load
$b2$ is the distance from combined load centre LC^* to load centre from symmetric load

The distance to combined load centre can be expressed as following:

$$LC^* = LC2 + b2 \Longleftrightarrow b2 = LC^* - LC2 \tag{4.14}$$

and

$$LC^* = LC1 - b1 \Longleftrightarrow b1 = LC1 - LC^* \tag{4.15}$$

Where $LC2$ is $\frac{a}{2}$. Equation 4.15 and 4.14 are inserted in equation 4.11. Some reshuffling is made and the distance LC* can be expressed:

$$LC^* = \frac{Q1 \cdot LC1 + Q2 \cdot LC2}{Q1 + Q2} \tag{4.16}$$

4.4 Choice of horizontal load distribution model for Veddesta

It has been mentioned earlier that it is often a valid approximation for all floors that the horizontal load distribution is dependent on the stiffness for the shear walls. However, this assumption is based on that the slab is stiffer than the shear walls. It is now checked if this is the case for Veddesta.

A floor plan for a storey for Veddesta is seen in figure 4.5

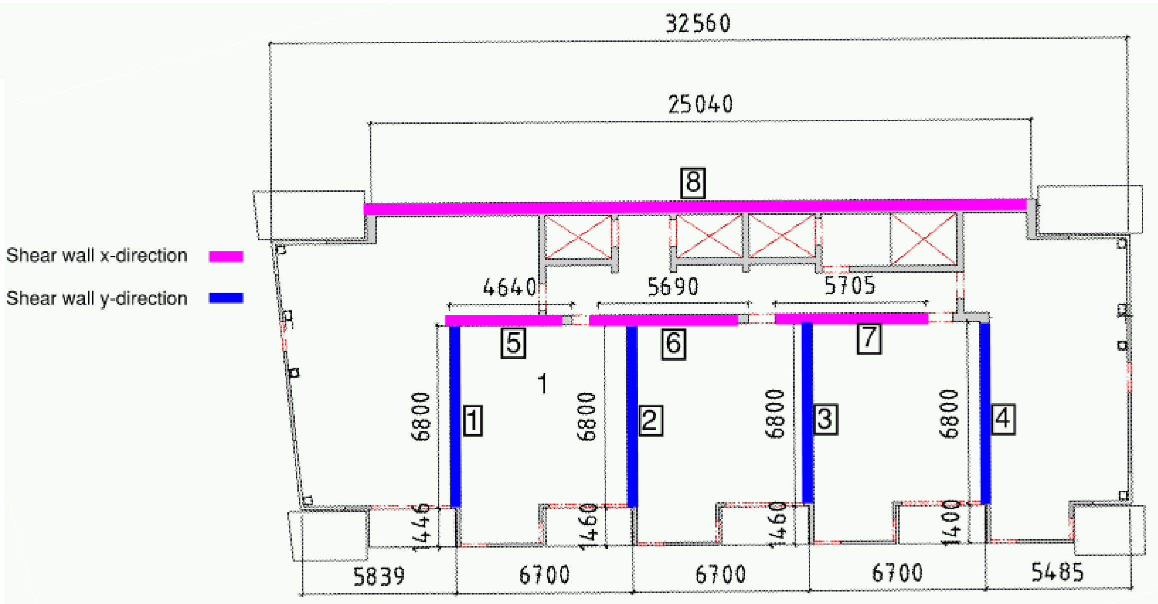


Figure 4.5: Floor plan with marked shear walls in x and y-direction.

The stiffness ratio is calculated using equation 3.6.

For calculating the stiffness of the shear wall(s), equation 3.7 is used. For Veddesta case study, storey height $H = 3$ m. Concrete is C30 and thus $E_b = 33$ GPa. The shear walls thickness (in y-direction) is 0.35 m and thus $t = 0.0.35m$. Length of the shear wall h is 6.8 m.

$$\delta_{wall} = \frac{1 \cdot H}{E_m \cdot t \cdot h}\left(\frac{4 \cdot H^2}{h^2} + 3\right) = \frac{1 \cdot 3}{33 \cdot 0.35 \cdot 6.8}\left(\frac{4 \cdot 3^2}{6.8^2} + 3\right) = 1.44 \cdot 10^{-10}m \quad (4.17)$$

The stiffness of the shear wall is thus:

$$S_{wall} = \frac{1}{\delta_{wall}} = \frac{1}{1.44 \cdot 10^{-10}} \text{ [N/m]}$$

(4.18)

For calculating the stiffness of the slab (diaphragm), equation 3.9 is used. $L = 6.7$ m. Concrete is C30 and thus $E_b = 33$ GPa. h_b is set to $8.52 + 4.215 = 12.735$ m. Thickness of the slab t_b is 0.25 m. The largest span L is seen in figure 4.5 for Veddesta.

The values are now inserted into equation 3.9 and thus:

$$\delta_{diaphragm} = \frac{L}{E_b \cdot t_b \cdot h_b}\left(\frac{L^2}{4 \cdot h_b^2} + \frac{3}{4}\right) = \frac{6.7}{33 \cdot 10^9 \cdot 0.25 \cdot 12.735}\left(\frac{6.7^2}{4 \cdot 12.735^2} + \frac{3}{4}\right) \tag{4.19}$$

$= 5.22 \cdot 10^{-11}$m

The stiffness for the diaphragm is thus:

$$S_{diaphragm} = \frac{1}{\delta_{diaphragm}} = \frac{1}{5.22 \cdot 10^{-11}} \text{ [N/m]} \tag{4.20}$$

The stiffness ratio on first floor can now be calculated:

$$C = \frac{S_{diaphragm}}{S_{wall}} = \frac{\frac{1}{5.22 \cdot 10^{-11}}}{\frac{1}{1.44 \cdot 10^{-10}}} = 2.7 \tag{4.21}$$

The stiffness ratio is also calculated for floor two and floor three and it is seen that C=8.6 for shear wall height of 6 m (floor two) and C=20.9 for a shear wall height of 9 m (floor three). This gives that for the slab on the first floor (3 m up) the load distribution shall be according to a continuous beam. For the second floor (6 m up), the load distribution shall be according to subjected area. For third floor and upper the load distribution is based on stiffness. However, since the whole building consists of 33 floors, it is reasonable to let Dynamo for simplification reasons distribute the horizontal load according to stiffness on all floors. This is further motivated by the fact that the wind load increases with height over the building and the lever-arm for the horizontal load is small on the first lower floors.

4.5 Vertical load distribution modelling

For horizontal stabilisation, the stabilising moment from vertical loads on the stabilising elements must be larger than the overturning moment from horizontal loads (wind load and imperfection load). It's of importance to know how much of the vertical load that is transferred down through the stabilising element as this governs the stabilising moment.

How much of the vertical load that is transferred down through a vertical element depends on the boundary conditions for the slab that is connected to the vertical element and if the slab/roof is continuous. For simply supported slabs, it is simple to calculate the vertical load distribution. In Figure 4.6 the influence area for two columns is marked with green and red. The slab marked with span-direction is simply supported.

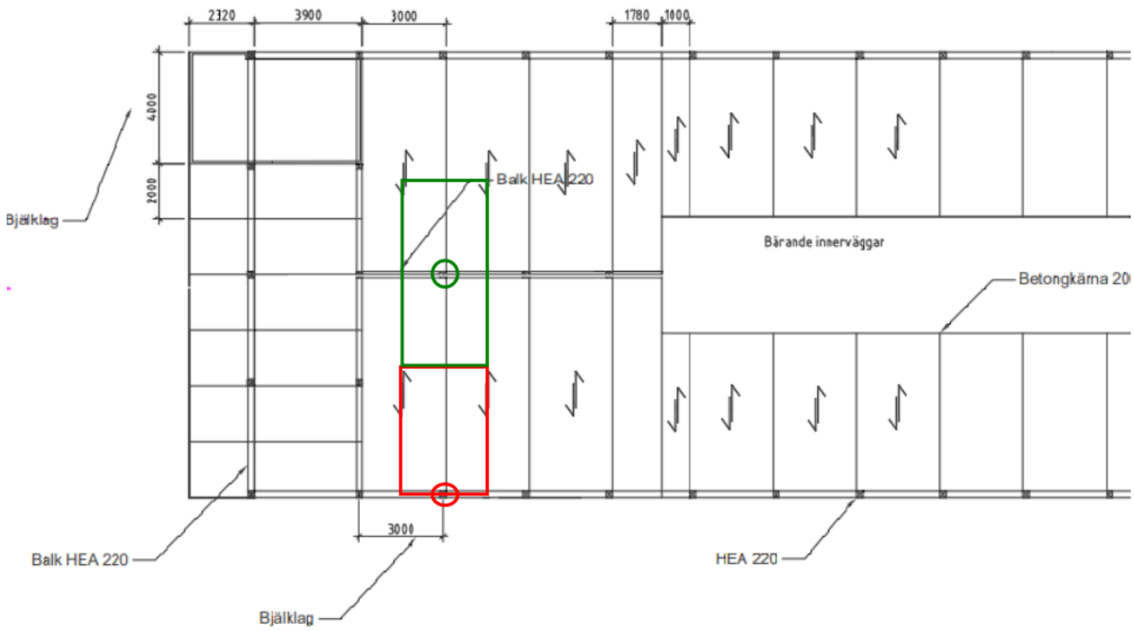


Figure 4.6: Vertical load distribution for column marked with circle in green with corresponding influence area in green rectangle. Same goes for red marking(s).

In early stages of the design process, fast and easy hand calculations are required to get an approximate value of the vertical load distribution. For the hand calculations, the influence area for a vertical element is approximated by the structural engineer.

For the Dynamo script, the less manual input the user has to do - the better. It is sought after to automate the calculation of the vertical load to the vertical load bearing elements.

For our case study Veddesta, the building is planned with a permanent formwork with concrete cast on top spanning in several directions. Due to the concrete poured on top, the slab on a storey will act as one single "unit" and all columns, load bearing walls and shear walls will act as supports.

Permanent formwork is a prefabricated concrete slab with precast bottom reinforcement. At the building site, installations and reinforcement is added to the permanent formwork and then concrete is poured to get the sought-after height of the slab [12].

The problem of how to tackle automatic implementation of vertical load distribution was discussed with the industrial supervisors of the thesis at ELU. The recommendation was to implement a so-called *Voronoi diagram* script in Dynamo.

The Voronoi diagram performs a geometric partitioning of the floor slab. Each partitioned area (region) is connected to a vertical support (column, load bearing wall, shear wall). The area is partitioned to the closest object,[13] which means for load distribution that the load is transferred to its closest support. To get the vertical load down for the region connected to the support, the area of the region is multiplied with the load on the region.

Figure 4.7 shows a Voronoi partitioning of the slab into several regions, where each region is connected to a column or wall.

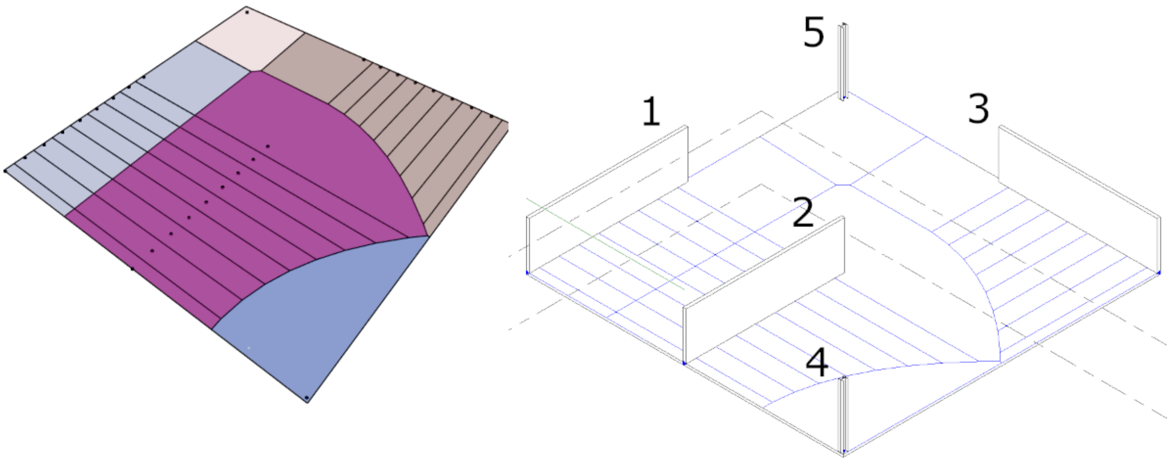


Figure 4.7: Influence area for element 1-5 with voronoi diagram where each color highlights influence area in one element.

The downside of using only Voronoi diagram partitioning, is that the stiffness of the walls and columns are not taken into account. In some load cases, this might yield inaccurate vertical load distribution.

4.5.1 Test of implemented Voronoi script in Dynamo

A small test is done to see if the Voronoi script in Dynamo gives a reasonable approximation for the vertical load distribution for a floor that is supported by columns and walls. The sample structure is a 20 x 20 m structure with a height of 3 m. The sample structure is supported by two IPE 80 columns in two corners and three load bearing walls with all of them having a length of 10 m and a thickness of 0.35 m. A slab is resting on the said vertical load bearing elements and is subjected with a load of 1 kN/m^2.

The comparison are done using a FEM-model in FEM-Design. In the FEM-model, the supports for the walls and columns for the ground are set to fully fixed.

For the vertical load distribution the governing parameter is the stiffness of the columns and walls. Figure 4.8 shows the sample building used for comparison test between FEM and Voronoi script in Dynamo.

The resulting vertical reactions on the vertical load bearing element can be seen in figure 4.9.

The supports in the FEM model are changed to hinged support to see if it results in any difference in the vertical load distribution. The load distribution on the hinged support can be shown in figure 4.10. It is shown that whether the supports are modelled as

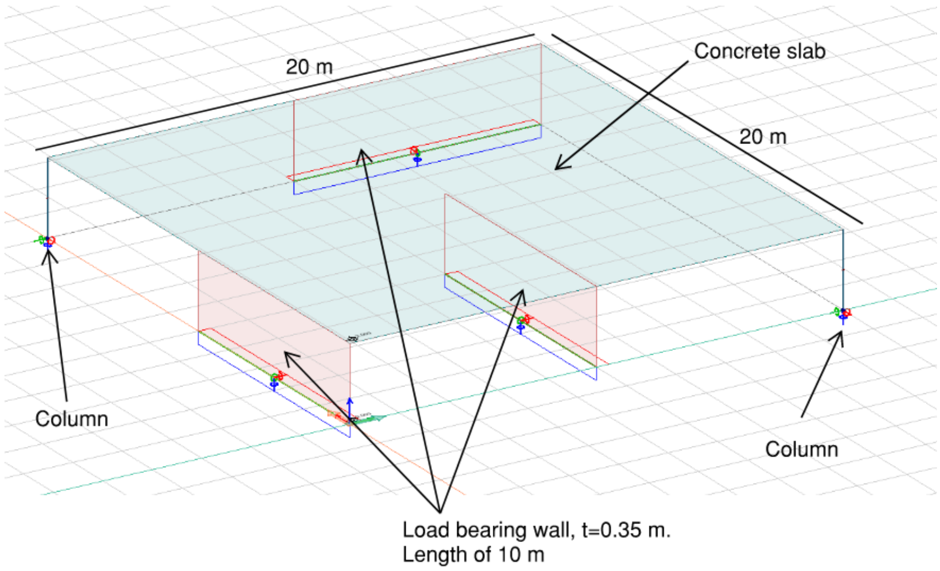


Figure 4.8: Sample building for voronoi comparison.

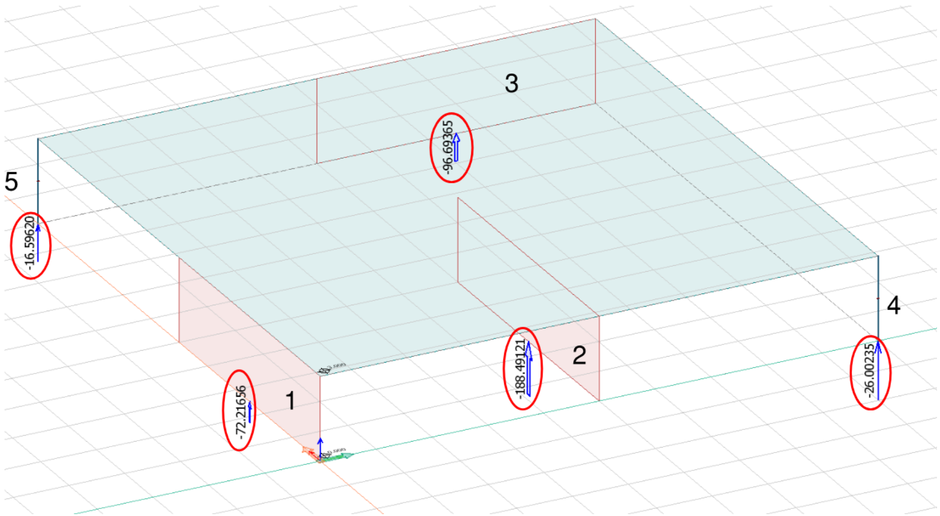


Figure 4.9: Reaction forces on vertical load bearing elements. Vertical load bearing elements are numbered 1-5. Supports are set to fully fixed.

hinged or fully rigid, the difference in vertical support reaction is negligible.

The same analysis is computed but now increasing the stiffness of the columns (changing cross section). Instead of cross section IPE 80, concrete cross section 1000x1000 mm^2 and thus much stiffer columns are assumed. From figure 4.11 it can be seen when comparing to figure 4.10 that the difference in vertical load for columns IPE 80 vs concrete 1000x1000 mm^2 is about 20 percent. For walls, the difference is a few percent.

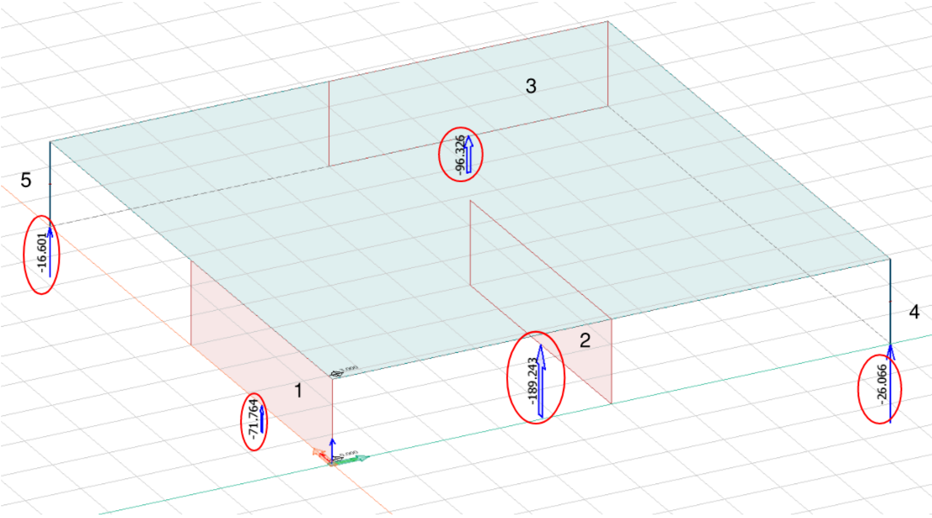


Figure 4.10: Reaction forces on vertical load bearing elements . Vertical load bearing elements are numbered 1-5. Support conditions are set to hinged.

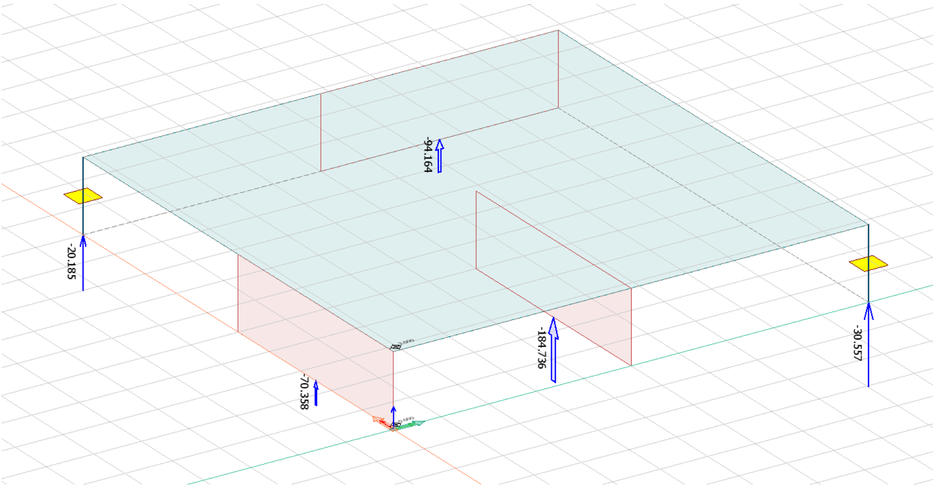


Figure 4.11: Reaction forces on vertical load bearing elements . Vertical load bearing elements are numbered 1-5.

The same structure is modelled in Revit to see what the output from the Voronoi script is, see Figure 4.12.

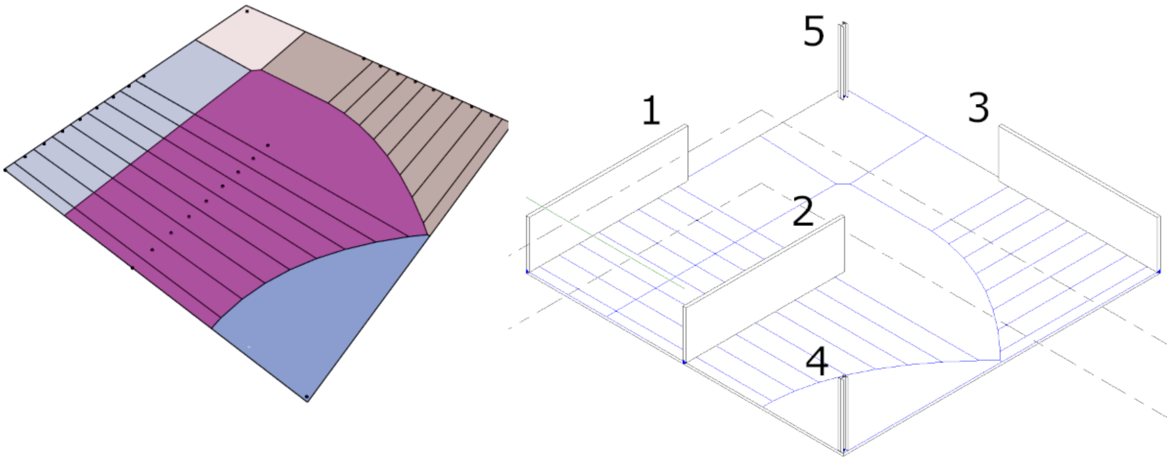


Figure 4.12: Influence area for element 1-5 with voronoi diagram where each color highlights the influence area in one element.

Table 4.3 shows the vertical load distribution in FEM-Design (analysis with 1000x1000 mm^2 columns) and vertical load distribution using Voronoi diagram on same structure.

Table 4.3: FEM vertical load distribution and Voronoi vertical load distribution where element 1, 2, 3 are shear walls and element 4, 5 are columns.

Element	FEM Vertical Load [kN]	Voronoi Vertical Load [kN]	Similarity [%]
Element 1	70.4	73.9	95%
Element 2	184.7	171.9	107%
Element 3	94.2	90.6	104%
Element 4	30.6	34.7	88%
Element 5	20.2	28.9	70%

It can be seen that the vertical load distribution in the Voronoi script in Dynamo gives a good approximation of the actual load distribution (FEM). The relative difference in vertical support reaction for the walls in FEM compared to Voronoi in Dynamo is small. For the columns, a relative difference is notable. For the stabilisation controls of the shear walls, it is important that the vertical load in said elements are somewhat correct.

4.5.2 Test of implemented Voronoi script in Dynamo for Veddesta

A test is done on Veddesta case study to compare the vertical load distribution using Dynamo (Voronoi) and FEM-Design. Only the resulting support reaction on the shear walls have been compared as those are the walls of interest in this analysis. The concrete used in the FEM-Design is C25/C30 with Young's modulus of 31 GPa. Vertical load bearing wall and shear wall thickness according to drawings in the Revit model. Figure 4.13 presents the model in FEM-design based on Veddesta the Revit model.

A vertical surface load of 1 kN/m^2 is put on the slab in the FEM-model.

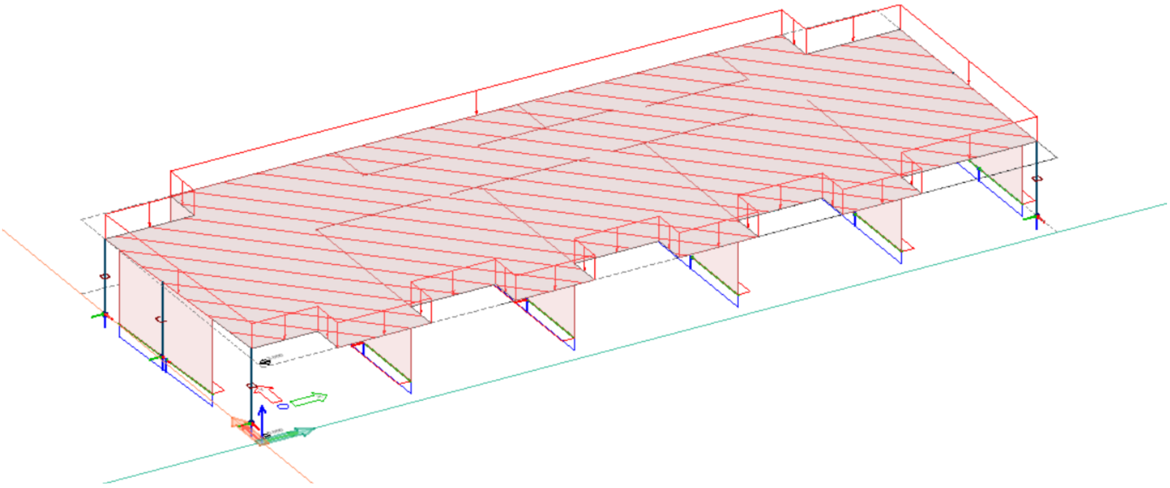


Figure 4.13: FEM model for comparing support reaction forces with Dynamo, vertical load is applied.

Dynamo split the slab surface according to Figure 4.14.

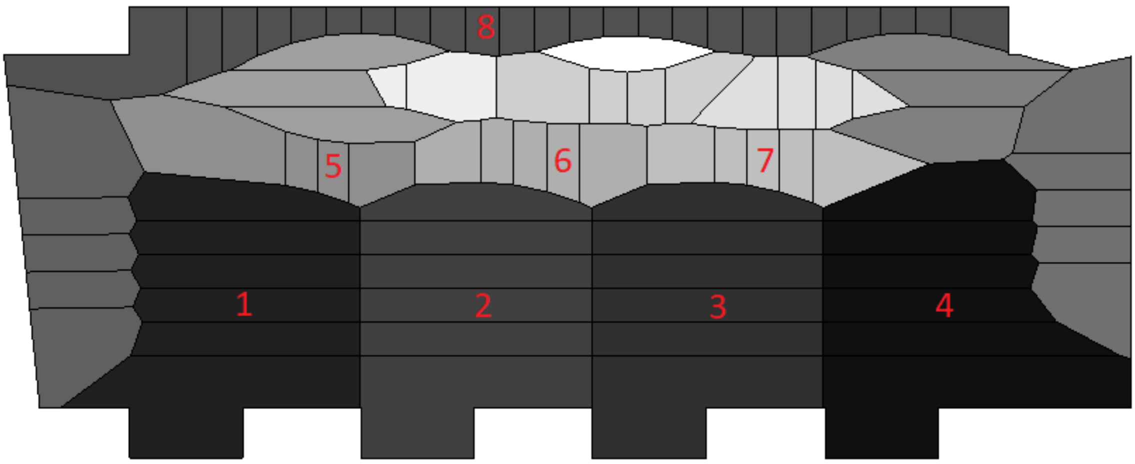


Figure 4.14: Voronoi surface regions. Numbered regions in figure represent corresponding element in table 4.4.

Table 4.4 presents a comparison between vertical load distribution for the shear wall elements. The other load bearing walls and columns are not presented in table 4.3 since they are not relevant for the script. By looking at table 4.4 it can be seen that a majority of the shear walls support reaction according to Voronoi partioning by Dynamo is rather similar to the FEM-model. Thus, in this thesis, the distribution of vertical loads to shear walls will be determined by Voronoi diagrams in Dynamo.

Table 4.4: FEM load distribution and Dynamo Voronoi vertical load distribution for Veddesta.

Shear wall	FEM Vertical Load [kN]	Voronoi Vertical Load [kN]	Similarity [%]
Element 1	56.9	48	84.4%
Element 2	40.6	47	115.6%
Element 3	41	47	114.6%
Element 4	53.2	52	97.8%
Element 5	19.6	17	86.8%
Element 6	19.5	12	61.5%
Element 7	18.3	13	71.2%
Element 8	34.7	32	94.3%

4.6 Evaluation of openings in shear walls

Openings are often occurring in shear walls and reduces the stiffness of the wall. This results in a different horizontal load distribution for a structure with three or more shear walls orientated in same direction. In tall buildings, the openings (from windows and doors) commonly occur at the same position on each floor [7]. The impact of openings in shear wall is mainly on the deformations caused by the shear force [3].

The connection between two parts of the same shear wall where an opening divides them is called a coupling beam. The strength of the coupling depends on the size of the holes, where small holes result in the same stiffness as a massive wall without openings, and large holes result in the stiffness of the wall being completely separated into two parts (see figure 4.15).

In the case where the hole size is somewhere in between the models presented in figure 4.15, the wall can be modeled as two walls with synergy. The synergy effect result in a greater stiffness than two totally separated walls (further explained in chapter 4.6.2).

Different depth to span ratio of the coupling beam results in different failures. Slender beams (high span/depth ratio) result in failure in flexure, and deeper beams result in diagonal tension failure (preventable with stirrups) [14].

Different analytical methods are studied to determine which method is most suitable for the script being used.

To compare the methods, length parameters are determined for one floor and are the same for all the methods being compared (see figure 4.16 for notations). The height is multiplied with the number of floors to determine the total height of the building. As discussed in earlier chapters, the stiffness of a wall can be derived through a horizontal unit load at the top of the structure. This method is used where the displacement (which is inverted stiffness) at the top is compared.

In the comparisons, material values are assumed to have normal values for concrete E=34 GPa, G=12.5 GPa). For the sake of comparison, the chosen value of the thickness does not matter, as long as it is constant throughout the tests. For this reason, the thickness is given a value of t = 1 m, meaning it can be removed from the analytical

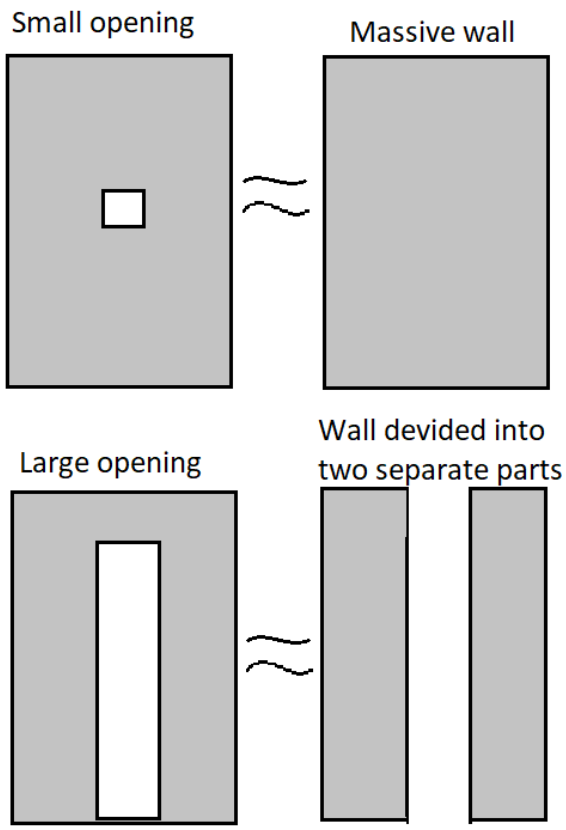


Figure 4.15: Impact of different size of openings in modeling of shear walls.

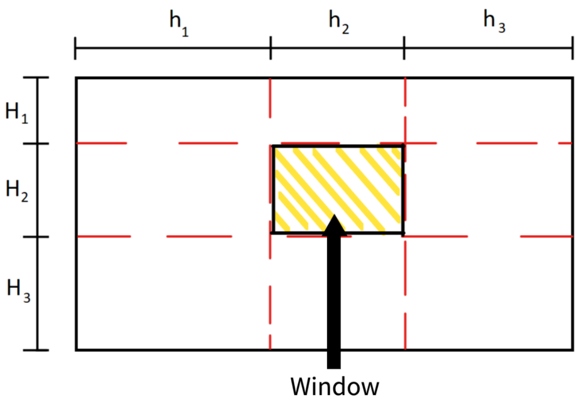


Figure 4.16: Notations of length parameters used in the evaluation of openings in shear wall. Inner rectangle is an opening in a wall.

equations in MATLAB. The length of the wall is the sum of the length parameters $h = h_1 + h_2 + h_3$ and the height of one floor is the sum of the height parameters $H_{floor} = H_1 + H_2 + H_3$. The total height of the building is calculated by multiplying floor height with the number of stories $H = H_{floor} \cdot n_{floors}$. The comparison is made up too 13 floors as this gives a sufficient representation of the relations between the methods.

4.6.1 Massive wall model

The first method, modelling the wall as a massive wall with no openings is represented by figure 4.17 below giving deformation $\delta_{massive}$ from equation 4.22. This equation considers both shear and bending deformation.

$$\delta_{massive} = \frac{4 \cdot H^3}{E_m \cdot t \cdot h^3} + \frac{3 \cdot H}{E_m \cdot t \cdot h} \tag{4.22}$$

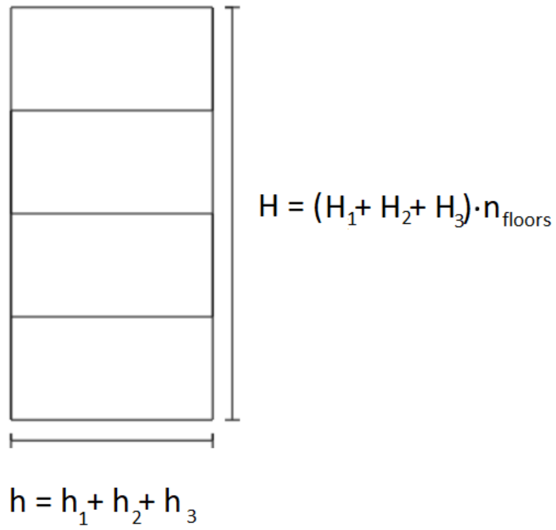


Figure 4.17: Modeling of massive shear wall used for comparison.

4.6.2 Separated wall with synergy model

The second method removes the part of the wall with an opening (width L_2 and height H_2, see figure 4.16). The shear walls are modeled with full synergy between the separated parts. This is done by modelling it as one shear wall with one length as seen in figure 4.18. The displacement from the unit load is calculated with equation 4.23.

$$\delta_{Synergy} = 4 \cdot \frac{H^3}{E_m \cdot t \cdot (h_1 + h_3)^3} + 3 \cdot \frac{H}{E_m \cdot t \cdot (h_1 + h_3)} \tag{4.23}$$

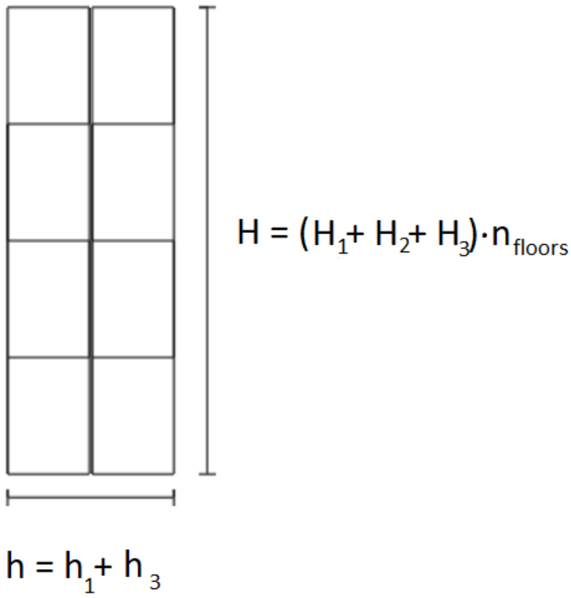


Figure 4.18: Example of removing the part of the wall with opening to take into account reduced stiffness.

4.6.3 Finite element method (MATLAB)

The third method uses finite element method modeling, and introducing a point load in the top left corner of the building with a force of 1 N. In this method, the Poisson's ratio v is needed, which is given the value for concrete of 0.2. The building is modelled with quadrilateral plane stress elements. It is modelled linearly (elastic stage) with no consideration of reinforcement.

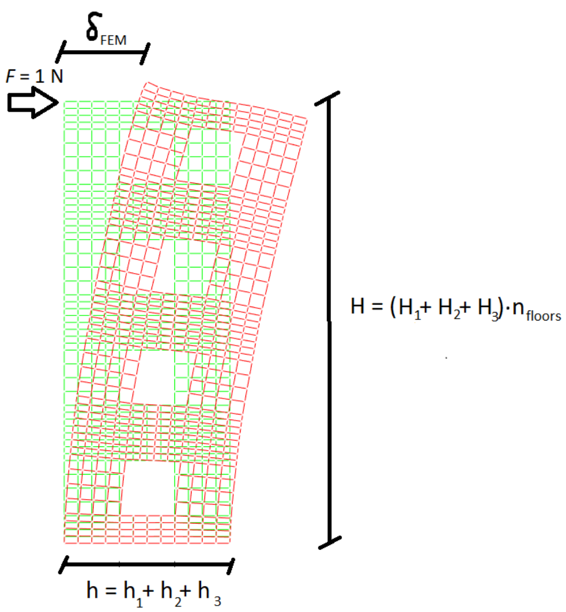


Figure 4.19: Modeling of shear wall used for comparison.

4.6.4 Continuous medium method (CMM)

The fourth model is the continuous medium method, where the wall is modelled as two beams with an imaginary continuous lamina with an equivalent stiffness of I_b/h for a story height h, giving stiffness $\frac{I_b}{h}dx$ for a height of dx. [14].

The model of the shear wall with a continuous lamina is presented in figure 4.20.

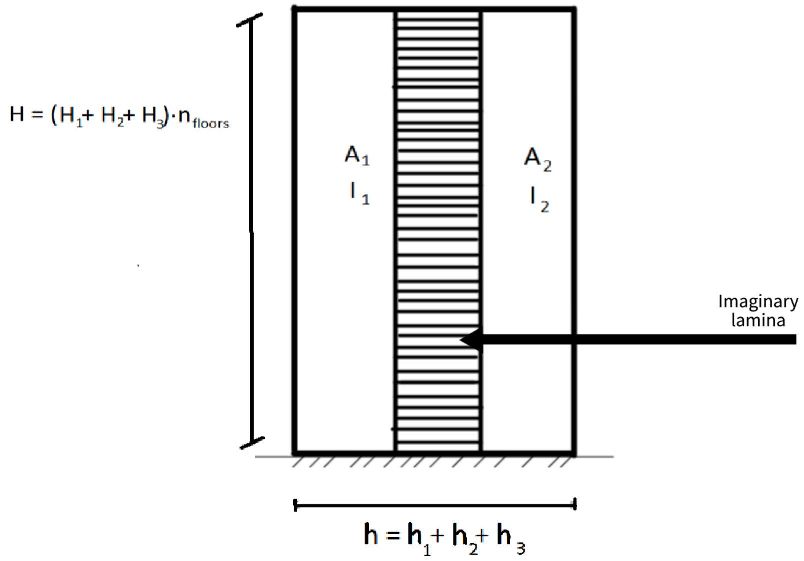


Figure 4.20: Continuous medium method with a continuous lamina where instead of a row if windows.

Sosena Eshetu [15] introduces a variable F_3 that is multiplied with the equation for bending deflection for a load P. The continuous medium method does not take shear stiffness into account, resulting in larger errors for deeper and less slender buildings.

Applying the same conditions as methods above, the variables are as following:

$E = 34$ GPa
$h = h_1/2 + h_2 + h_3/2$
$I = \frac{t \cdot h^3}{12}$
$H =$ Height of the entire building
$b = h_2$
$A_1 = h_1 \cdot t$
$A_2 = h_3 \cdot t$
$A = A_1 + A_2$
$d = (h_1 + h_3)/2$
$I_b' = \frac{t \cdot d}{12}$
$I_b = \frac{I_b'}{1+2.4(\frac{d}{h_2})^2 \cdot (1+v)}$

Variable α and k are geometric parameters, and a measure of the relative stiffness with respect of the wall. The variables are calculated in equation 4.24 and 4.25.

$$\alpha^2 = \frac{12 \cdot E \cdot I_b}{h \cdot b^3} \tag{4.24}$$

$$k^2 = \Big[\frac{l^2}{I} + \frac{A}{A_1 \cdot A_2}\Big] \tag{4.25}$$

With these, the deformation δ at height H from load P is calculated with equation 4.26 and 4.27.

$$\delta(H) = \frac{P \cdot H^3}{3 \cdot E \cdot I} \cdot F_3 \tag{4.26}$$

$$F_3 = \frac{1}{k^2}\Bigg[1 - 3\Big(\ \frac{1}{3} + \frac{sinh(k \cdot \alpha \cdot H\sqrt{n_b})}{k^3 \cdot \alpha^3 H^3 \cdot n_b^{3/2} \cdot cosh(k \cdot \alpha \cdot H\sqrt{n_b})} - \frac{1}{k^2 \cdot \alpha^2 \cdot H^2 \cdot n_b}\Big)\Bigg] \tag{4.27}$$

where n_b is number of beams per floor which is 2 for a wall with only one opening over the same height. To determine the stiffness of the wall, the load P is the unit load of 1 N, and the deflection δ_{CMM} is calculated and compared with the other methods

4.6.5 Results of the comparison

The comparison is made for four different building geometries. The height of one floor is set to 3 meters and the number of floors vary with the y axis (presented in total height of the building in m). The displacement of is seen on the x axis. The total building height starts with 3 meters (1 floor) and adds one floor on top and compares until it reaches 39 meters (13 floors). The comparison was tried up too 33 floors as well and lead to the same conclusions. 13 floors was used for figures since taller buildings lead to difficulties reading the results in the graphs.

The wall dimensions are constant over all floors in each individual study. The widths of the walls and windows are varying between the compared configurations as seen on the Figure 4.21. The widths are chosen to try the following geometries:
top left: wide window (2 m) with slim walls on the side (2 m) figure 4.21.
top right: slim window (1 m) with wide walls (3 m) figure 4.21.
bottom left: slim window (1m) with slim walls (0.5 m) figure 4.21.
bottom right wide window (5 m) with slim walls (0.5 m) figure 4.21.

Comparing the results are done with the assumption that the finite element method is correct. Figure 4.21 shows the different displacements from the unit load of 1 N, and shows how the massive modeling method and CMM are both close to the FEM-model, while the synergy model gives a large error margin.

The weakness of the CMM method, is that all openings must be evenly placed in a line with the same area. When this is not the case, an average window size and placement could be implemented for the CMM method to be applicable, but more assumptions and changes can lead to larger error margins.

The conclusion from figure 4.21 and above argument is that the most suitable model for a shear wall with openings is to model is as a massive cross section (with regards to distribution of horizontal load).

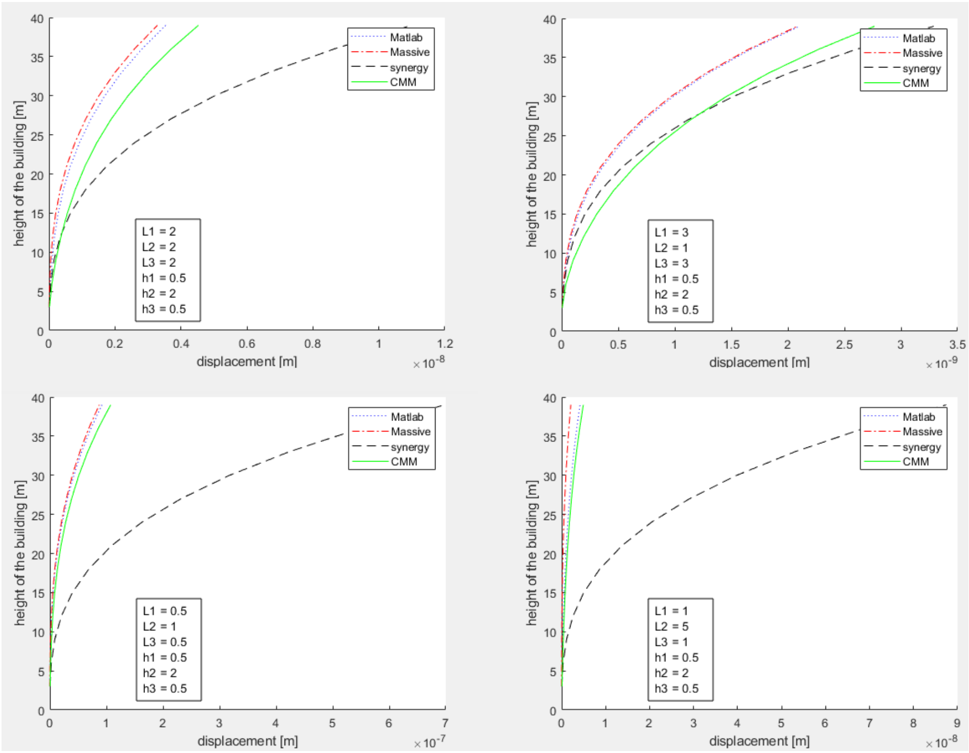


Figure 4.21: Comparison of models of shear wall with regular occurring openings in a line.

The MATLAB code of the evaluation of openings is presented in appendix 8.

4.7 Creation of the script i Dynamo

The script is created by using an example building model from Revit. The script is generic and it is supposed to work on buildings with different geometries and different positioning/geometries of the shear walls.

The logical procedure of the script is presented in figure 4.22. The Revit model data is exported to the Dynamo script. The Dynamo script performs calculations, where the user then can see if the shear wall capacity is enough. If they are not approved, the user can change the Revit model based on the feedback from the Dynamo script, and run the script again.

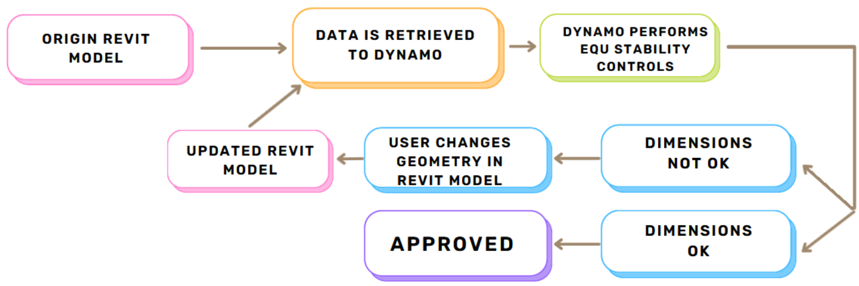


Figure 4.22: Logical procedure of the script.

4.7.1 General

Safety class coefficient γ_d is set to 1.0 when failure poses risk of severe consequences. In the Dynamo script, the safety class coefficient is thus set to 1.0.

According to *KL-trähandbok* [16], the overturning moment in a wood building can be only verified against the stabilising moment from self-weight of the building [16] (load combination 1). This load combinations gives the lowest vertical load and imperfection load while´the another load combinations would give a larger vertical load and imperfection load. For a early stage design, this is deemed okay to do for a concrete building as well since the relations between vertical load and imperfection load is the same for wood and concrete buildings.

Because of this, the imperfection load in the Dynamo script will only be calculated as a product of the self-weight. Snow and imposed load will not be used in the calculation of the imperfection load or stabilising moment.

4.7.2 Modelling of walls and columns in Dynamo for Voronoi

Voronoi function in Dynamo uses points to split the entire surface into small regions closest to each point. Each column is modelled as a point with the centre of the column as the coordinate. Walls are modelled as multiple points, where the areas of each point are summarized for a total influence area for the entire wall. The Voronoi Figure 4.23 shows how an example wall and column are modelled in the Dynamo model.

For controls of the overturning moment on the shear walls, the vertical load on the shear wall is evenly distributed over the cross section of the shear wall.

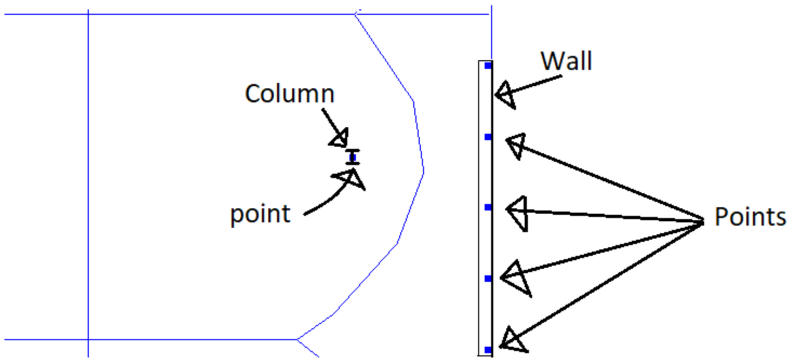


Figure 4.23: Wall and column modeled as points for use in Voronoi diagram, where the blue lines are the Voronoi surface being split into smaller surfaces.

4.7.3 Modelling of the building coordinate system

For many of the equations previously stated in this thesis, the difference in coordinates for x and/or y are used to describe a distance (for example for the lever-arm in equation 3.14) utilising the building being parallel with one of the coordinate axes. In reality however, buildings will not be perfectly aligned with the cardinal directions, which some may show in the Revit model. For the equations earlier described in this thesis to still be valid, the model of the building is rotated in Dynamo with help of coordinate transformation as presented in equation 4.28 where x, y are the original coordinates, ϕ is the angle in which the building is being rotated and x', y' are the new coordinates. To reverse the coordinate system into its original axis, equation 4.29 is utilized.

$$\begin{bmatrix} x' \\ y' \end{bmatrix} = \begin{bmatrix} cos(\phi) & sin(\phi) \\ -sin(\phi) & cos(\phi) \end{bmatrix} \begin{bmatrix} x \\ y \end{bmatrix} \tag{4.28}$$

$$\begin{bmatrix} x \\ y \end{bmatrix} = \begin{bmatrix} cos(\phi) & -sin(\phi) \\ sin(\phi) & cos(\phi) \end{bmatrix} \begin{bmatrix} x' \\ y' \end{bmatrix} \tag{4.29}$$

The coordinate rotation is not seen in Revit, but only occurs in the background of Dynamo for the calculations.

The transformation from original coordinate system to new coordinate system is illustrated by figure 4.24

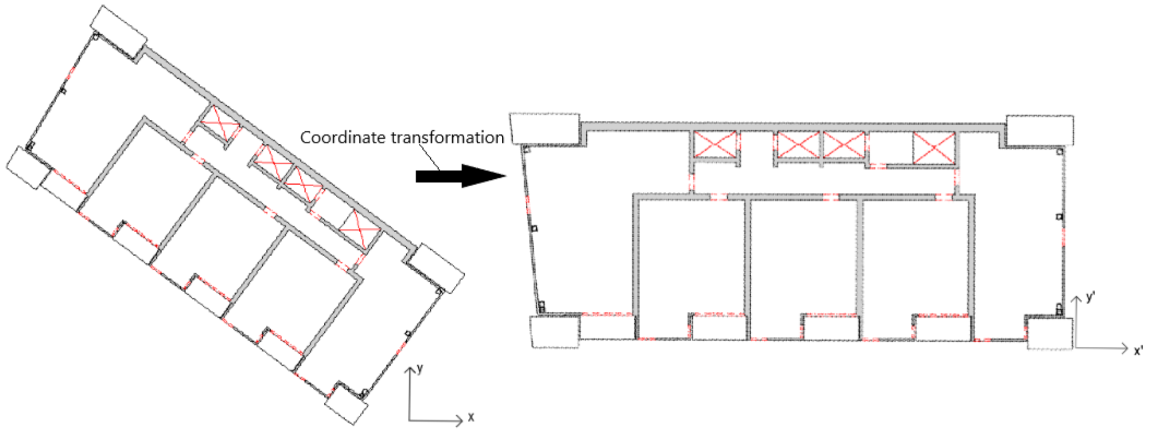


Figure 4.24: Left: Original coordinate system, Right: New transformed coordinate system.

4.7.4 Modelling of height

Experience with different Revit models showed that there are multiple ways to draw the same building. Some users draw walls as continuous elements from base to top, and some users draw multiple stacking walls with the height of one floor each. For this model to work in both scenarios, the height of the shear walls are entered manually in Dynamo instead of being extracted from Revit. This is done with the assumption that every wall, and every floor height is the same. The values are entered with sliders in Dynamo according to figure 4.25.

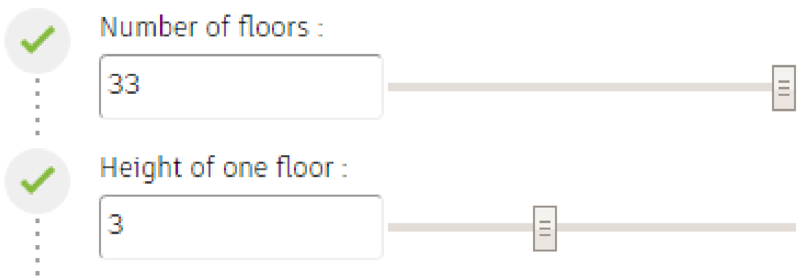


Figure 4.25: Input of floor height and number of floors in Dynamo used to model the building.

4.7.5 Modelling of walls

The coordinates are extracted from the Revit model and are used for calculations in Dynamo. For each wall, only the start and end coordinates are extracted and a linear line is drawn between these to symbolise the wall. This only works for straight walls, which is a prerequisite for the script to work. For stiffness calculations, all wall elements are calculated as massive, with holes not taken into account.

4.7.6 Modelling of building geometry for wind load

For Dynamo to perform calculations, a simplified model of the building geometry is used for the horizontal wind load. The wind load, the width and depth of the building is simplified to a square according to figure 4.26 below, where the outer edges follow the largest x,y coordinate and the smallest x,y coordinate of the real building geometry. This is deemed a good simplification for some what rectangular buildings.

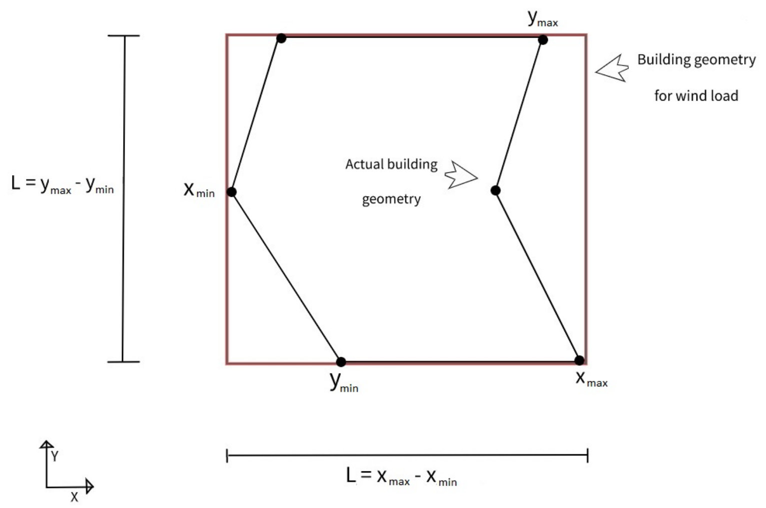


Figure 4.26: Building geometry simplified into a rectangle based on the max and min coordinate values of the floor slab.

4.7.7 Modelling of the slab

The slab is modeled of a floor element in Revit chosen by the user. This floor geometry is modelled on every floor for calculations. A simplification of the floor is made, where the same thickness of the slab is used on the roof. In cases where the roof is not designed to have people on the roof, the slab may be thinner (resulting in less stabilising load). An option for the user is implemented where the user can reduce the roof self-weight. The way this is implemented in the script is that the user has the option to reduce the thickness of the floor slab that acts as the roof.

Consider that the roof load per square meter from the slab is 6 kN/m^2 for a structure. In the Dynamo script, the roof is then by default 6 kN/m^2. For this example, the roof load is supposed to be 3 kN/m^2. In Dynamo player, the user then can then change the thickness of the slab on the roof so that the load from the "modified" slab would match the load for the actual roof. In the example this would mean that the thickness would be reduced by 50%, $0.5 \cdot 6$=3 kN/m^2

In Dynamo player, there is also the option to add installation-load on every storey of the building. By default this value is set to zero. Installation load act favourable as it increases the stabilising load.

A figure of the sliders in Dynamo player where the user can modify the self-weight load from roof and add installation load is shown in figure 4.27

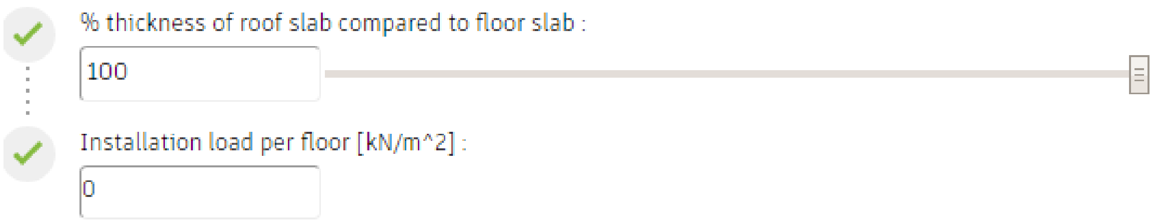


Figure 4.27: Adjusters in Dynamo player to reduce self-weight load from roof and add installation load to slab and roof.

4.8 Hand calculations

In section 7.3, hand calculations are provided for Veddesta case study to validate that the Dynamo script is correct. The hand calculations show the result of the overturning and stabilising moment for shear walls 1, 2, 3, 4 for when the building is subjected to symmetric loading and increasing wind load over the height of building (in y-direction) according to EKS11.

5 Results

The script created in Dynamo is able to calculate the stabilising and overturning moment respectively on shear walls in a building considering design load case EQU. The script considers both symmetric and asymmetric wind load for both positive and negative x- and y direction. It considers wind load over the height of the building according to EKS11. Imperfection load is considered and the imperfection load is only a function of the permanent loads (i.e. self-weight).

Dynamo collects data from the stability calculations and in a bar chart where the worst load for each wall is presented. This means that the user of the script would not need to manually go through the different load scenarios (symmetric wind load positive y-direction, asymmetric wind load negative x-direction) and so on.

Data from calculations are also automatically presented in an Excel sheet where the user can more in depth look at calculation data.

The wind load per floor is also presented in a chart that gives a good overview of the wind-load model and the numerical values for the wind load.

In the Revit file, the user can in the properties tab see the moment capacity of the stabilising element that is highlighted. If the moment capacity is exceeded for highlighted element, the user can try to increase the capacity and re-run the Dynamo script.

5.1 The script, and how to operate it

The Dynamo script is operated through the *Dynamo player* inside Revit. After entering the inputs the user runs the Dynamo script and can draw conclusions based on the outputs presented. The Dynamo player is presented in figure 5.1

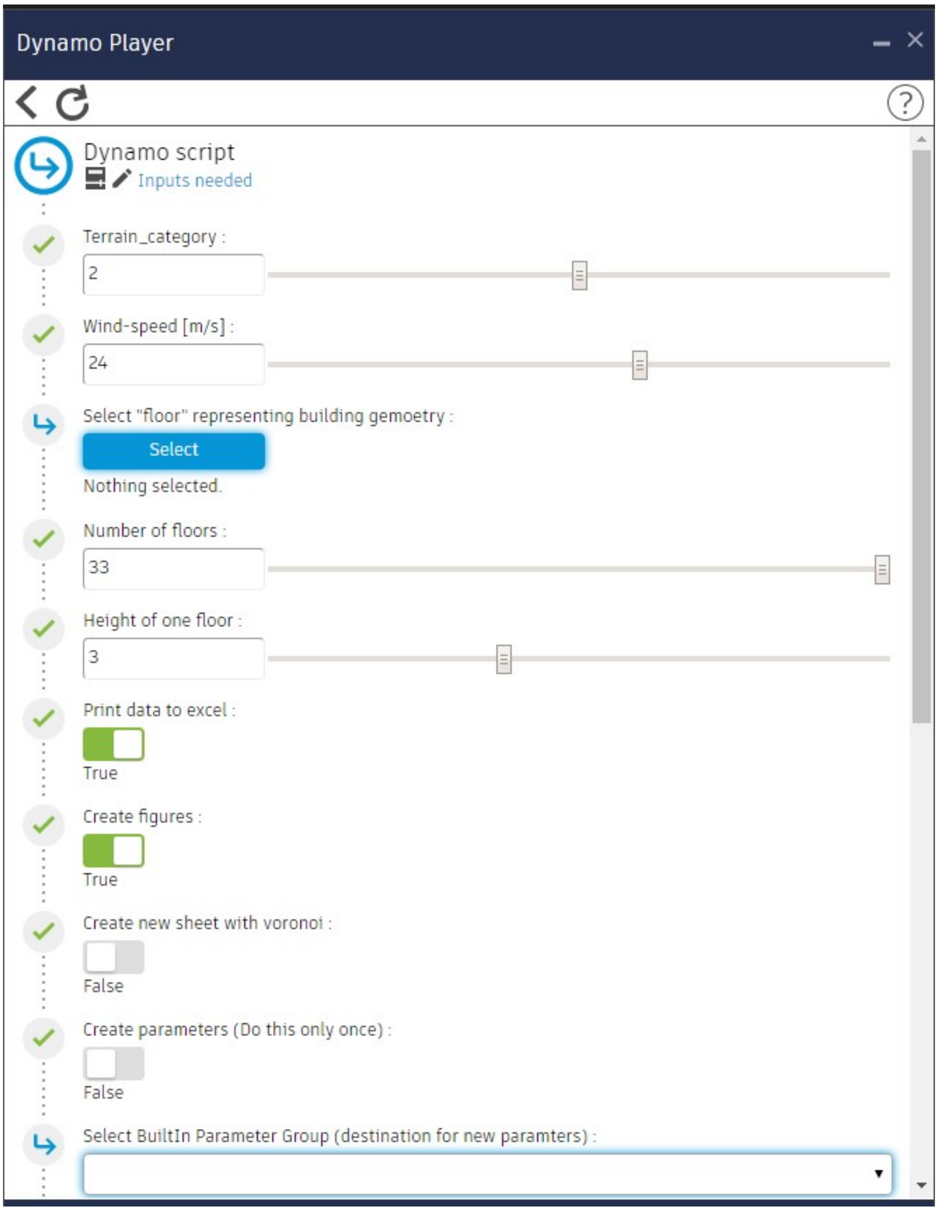


Figure 5.1: Dynamo player.

5.1.1 Inputs with Veddesta case study as example

In *Dynamo player* the inputs are entered by the user. With Veddesta as a calculation example, the inputs are:

Terrain category: 2
Wind speed: 24 [m/s]
Number of floors: 33
Height of one floor: 3 [m]

The user chooses a single floor geometry from the Revit file that is representative of the entire structure. The chosen floor geometry is then used for the entire building, with every floor being the same.

For Veddesta, the floor plan has two different layouts which are changing every other storey. The shear walls, load-bearing wall and load-bearing columns are always placed in the same coordinates, but the floor slab geometry is changing slightly. The shear walls, load-bearing walls, load-bearing columns and floor representing the building are chosen from *plan 34* in the Revit file, see figure 5.2. Figure 5.3, Figure 5.4 and Figure 5.5 show the load-bearing columns, load-bearing walls and shear walls being selected by checking the box at the parameter value in the properties inside the Revit elements. The script assumes all shear walls to be load-bearing as well, even if the box isn't checked in.

All walls for Veddesta are assumed to be load-bearing. Choosing which walls that are shear walls is done by consulting ELU.

With the height inputs, floor plan, walls and columns being selected, the script builds a calculation model of the building which has these placements on all floors from the first floor at 3 meter height to the last floor at a height of 99 meters.

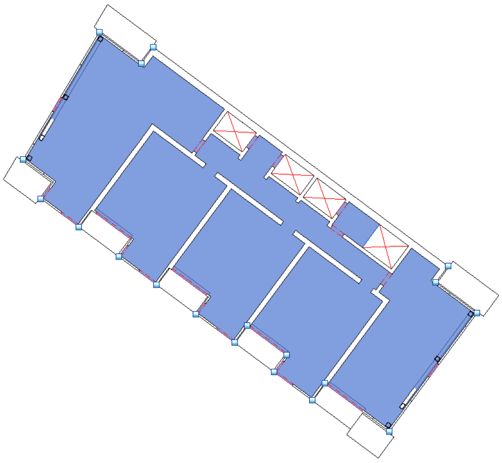


Figure 5.2: Choosing the floor plan which will represent the building geometry in the Dynamo script. The floor slab is marked with blue.

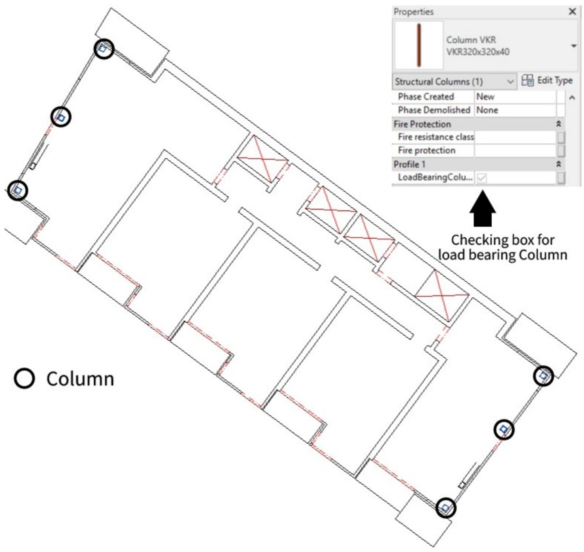


Figure 5.3: Selecting the load-bearing columns and checking the "Loadbearing" box in Revit to export coordinates of the columns to Dynamo.

Figure 5.4: Selecting the load-bearing walls and checking the "LoadBearing" box in Revit to export coordinates and lengths of walls to Dynamo. Selected walls are blue.

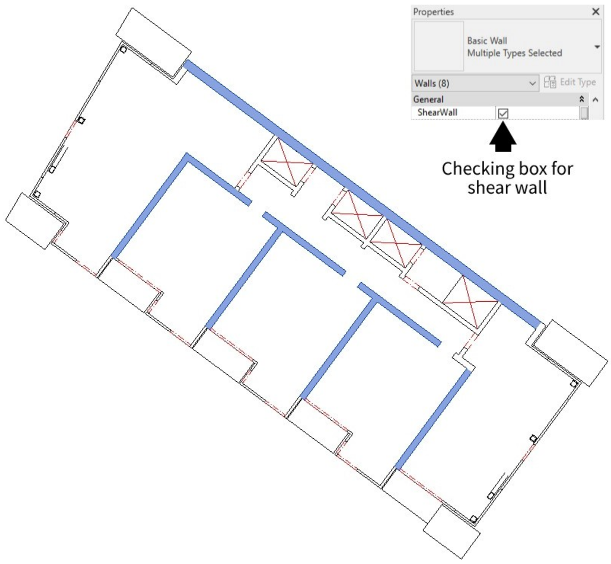


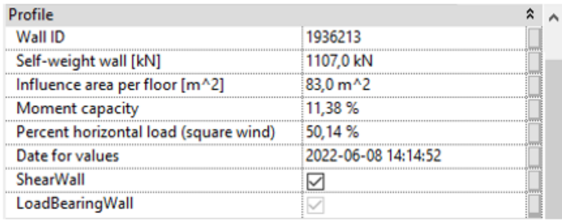
Figure 5.5: Selecting the shear walls and checking the "ShearWall" box in Revit to export coordinates, lengths and thickness of wall to Dynamo. Selected shear walls are blue.

5.1.2 Output with Veddesta case study as example

With building geometry being selected, the Dynamo script can be run. The data is presented in three different ways:

5.1.2.1 Inside Revit

Inside Revit, for the wall elements chosen as "shear walls", the influence area, wall ID, capacity (overturning moment divided by stabilising moment), percentage of horizontal load and vertical load from self-weight are seen in the properties tab of the element, see figure 5.6.



Profile	
Wall ID	1936213
Self-weight wall [kN]	1107,0 kN
Influence area per floor [m^2]	83,0 m^2
Moment capacity	11,38 %
Percent horizontal load (square wind)	50,14 %
Date for values	2022-06-08 14:14:52
ShearWall	☑
LoadBearingWall	☑

Figure 5.6: Output from script presented in the Revit wall element.

The Voronoi diagram is presented in a Revit view, with colors chosen so the element with the largest influence area is black, and the element with the smallest influence area is white. The shear centre on the first floor is presented as a blue circle, and the shear centre for the entire building height is presented as a red circle. Figures 5.7 presents the Voronoi diagram with load and shear centre for Veddesta.

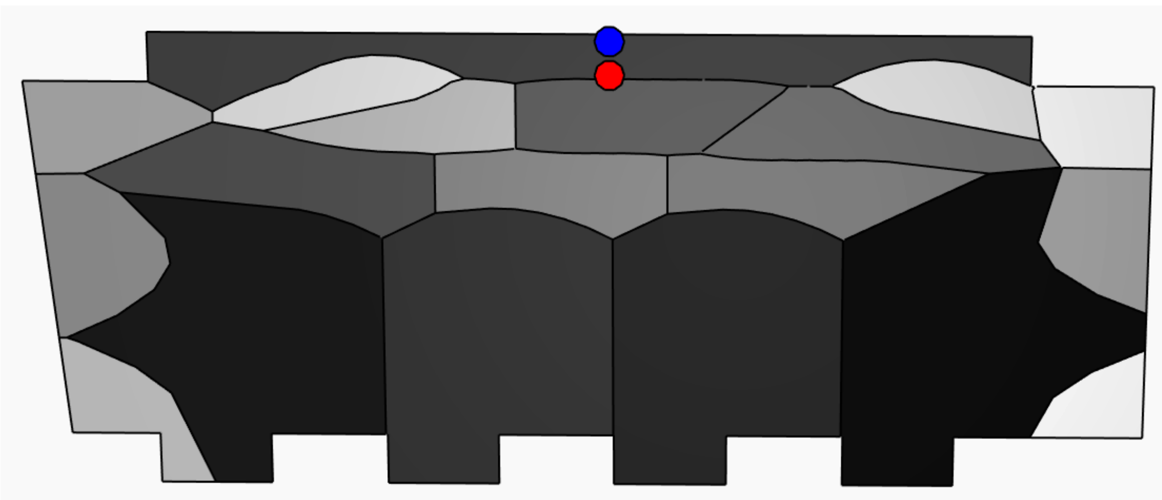


Figure 5.7: Voronoi diagram output in Revit per wall/column with shear centre displayed with circles. Blue circle (furthest up in figure) is for only the first floor and red circle (closer to centre in figure) for entire building. Regions represent influence areas for elements.

If only bending stiffness would be considered in calculation of the shear centre, the shear centre would be located at the same position for all floors. When both bending and shear deformation stiffness is considered there is difference in position of the shear centre for the first floor and top floor. This can be seen in appendix 7.3 in Table 7.9 where the distance e_y is calculated for each floor for Veddesta case study.

5.1.2.2 In Excel

In an Excel file, multiple sheets are created and data is exported from Dynamo. The calculation of wind load in two directions are presented in a sheet with the wind load as force per width (N/m) at each slab. The first excel column is the wind load for a square wind load over the entire width, and the second Excel column is the wind load for an asymmetric wind load over the width. The Excel data is presented in appendix Table 7.19.

In the Excel sheets named "wind load against y" and "wind load against x", a study of wind load distribution into the different walls based on the different sizes, orientation and load centre are presented. In the rightmost Excel column in both sheets, the worst-case characteristic load for each wall are presented per floor. The Excel data is presented for "wind load against y" in appendix Table 7.17.

The overturning moment and the stabilising moment are calculated in Excel sheet "Moment control against y" and "Moment control against x". For overturning moment controls, design values are used. The wind load has been multiplied with 1.5 and self-weight multiplied with 0.9. Self-weight multiplied with 1.1 for imperfection load. The load and the corresponding lever-arm are presented next to each other to make it easy for the user to follow the calculations. At the right part of the sheet, the overturning moment, stabilising moment, and % of capacity reached are presented. If the capacity is larger than 100 %, the overturning moment is larger than the stabilising moment. The Excel data is presented in appendix Table 7.18.

General information used in Dynamo are presented in Excel Table 7.20 in appendix. This gives the user a summary of the inputs going into the Dynamo script.

5.1.2.3 Inside Dynamo

Figures are opened when the Dynamo script has finished its calculations. The first figure shows the wind load distribution over the height of the building in both directions (for symmetric load) in a bar chart as presented in Figure 5.8.

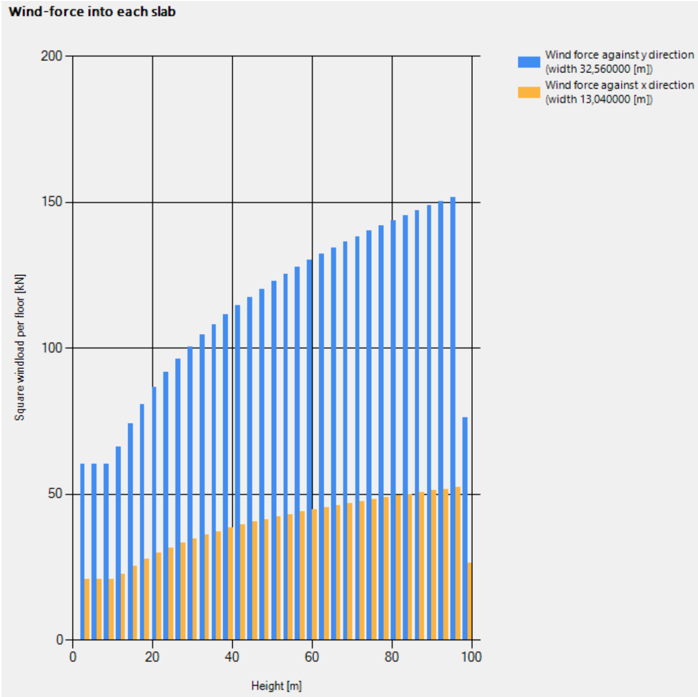


Figure 5.8: Wind load in the building over each floor for symmetric wind load (width) against y and x direction presented in a bar chart.

The second figure shows the overturning moment and stabilising moment for each shear wall given in a bar chart, see figure 5.9. The horizontal loads into all floors subjecting each shear wall multiplied with respective lever arms, and the moments are summarised as an overturning moment per wall. The vertical load from each floor, as well as the vertical load from the wall, are multiplied with respective lever arm and then summarised as a stabilising moment.

In the bar chart, all the wind load cases have been calculated, and the one resulting in the largest overturning moment is presented for each wall.

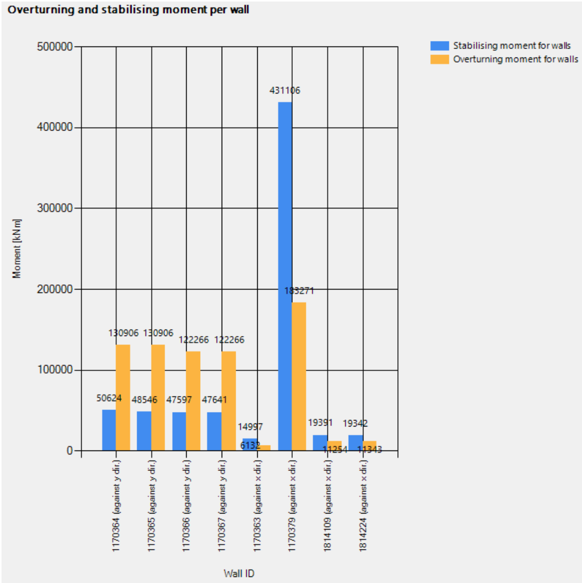


Figure 5.9: Overturning moment and stabilising moment presented per wall in a bar chart (Wall index from left to right: 4, 1, 2, 3, 5, 8, 6 ,7).

The walls are indexed with "Wall Id" which exist for all elements in Revit. For all the bar charts used, the wall index follow the numbers displayed in the bottom of Figure 5.10, with Figure 5.11 linking them to the shear walls in the Vedddesta drawings.

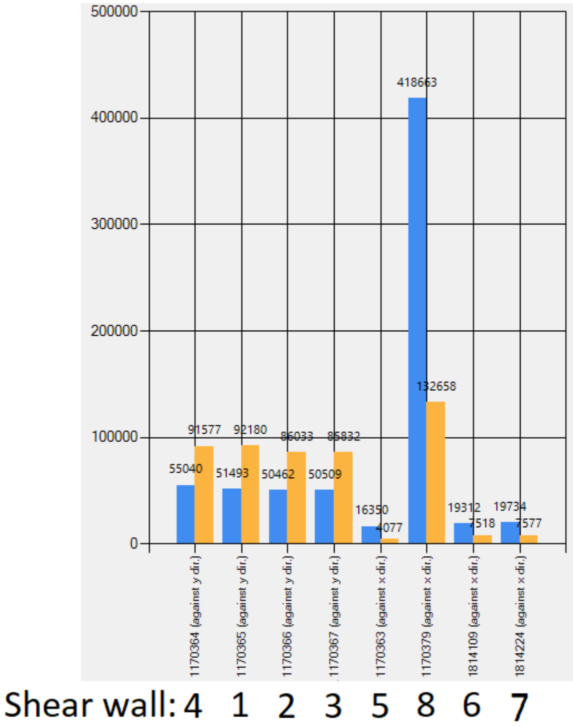


Figure 5.10: Bar chart, where the index of the walls are displayed (from left to right: 4, 1, 2, 3, 5, 8, 6 ,7) refering to the numbering of shear walls displayed in figure 5.11.

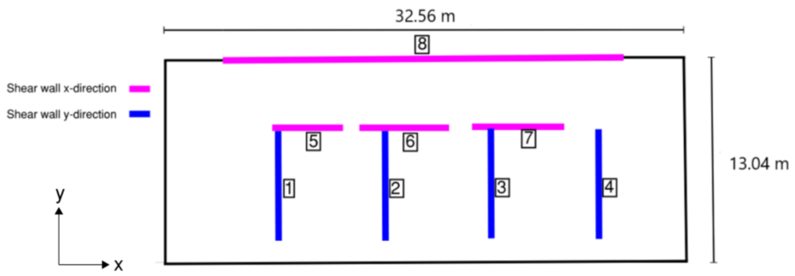


Figure 5.11: Simplified floor layout where wall elements are numbered.

5.2 Stability calculations results from Dynamo on Veddesta

5.2.1 Wind load type giving largest overturning moment.

Stability calculations are performed in Dynamo for both y-direction and x-direction for Veddesta. Calculations are performed for both symmetric wind loading and asymmetric wind loading in both direction. The total number of wind load cases being computed are 12 (6 per wind orientation). The 8 types on asymmetric wind load and 4 types of symmetric wind loads are presented in figures 5.12 and 5.13 respectively. The wind types are numbered (a-h and i-l) in figure 5.12 and figure 5.13 with Veddesta floor plan in the background. The shear walls are also numbered (1-8).

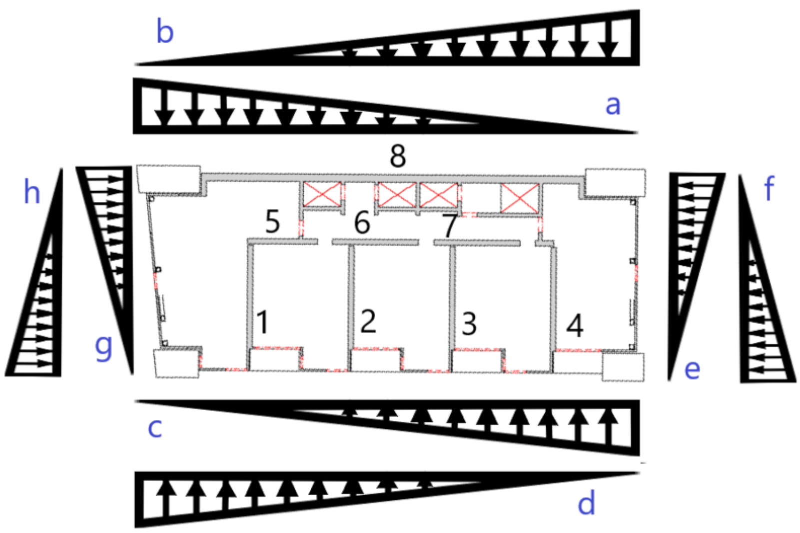


Figure 5.12: Windward placement and direction of asymmetric load in the two different directions.

The type of wind load giving the largest overturning moment per wall is presented in Table 5.1. The numbering of walls and the wind loads refer to the ones presented in figure 5.12 and 5.13.

As table 5.1 shows, the symmetric wind loads result in largest overturning moment for all shear walls in both directions. Since the shear centre and load centre are not coinciding, the load distribution from different sides also give a different load

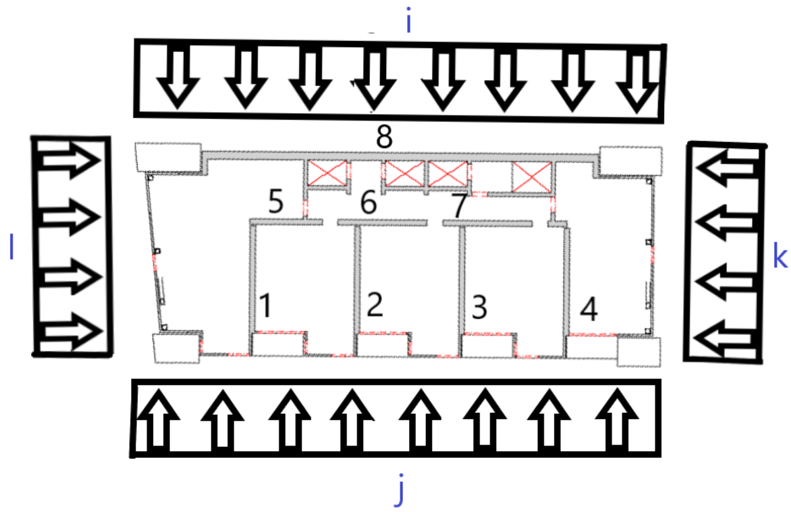


Figure 5.13: Windward placement and direction of symmetric load in the two different directions.

Table 5.1: Largest wind load type for respective wall.

Shear wall number	1	2	3	4	5	6	7	8
Load positioning giving largest overturning moment	i	i	j	j	l	l	l	k

distribution since torsion is considered.

5.2.2 Capacity control for shear walls

By looking at figure 5.14, it can be seen that the overturning moment exceeds the stabilising moment for shear wall 1, 2, 3, 4 (orientated in y-direction). The overturning moment for shear wall 5, 6, 7, 8 (orientated in x-direction) does not exceed capacity. The wind load cases that resulted in highest load per shear wall was stated in table 5.1.

Measures taken to increase the stabilising moment for shear walls 1, 2, 3, 4 are presented in section 5.3.

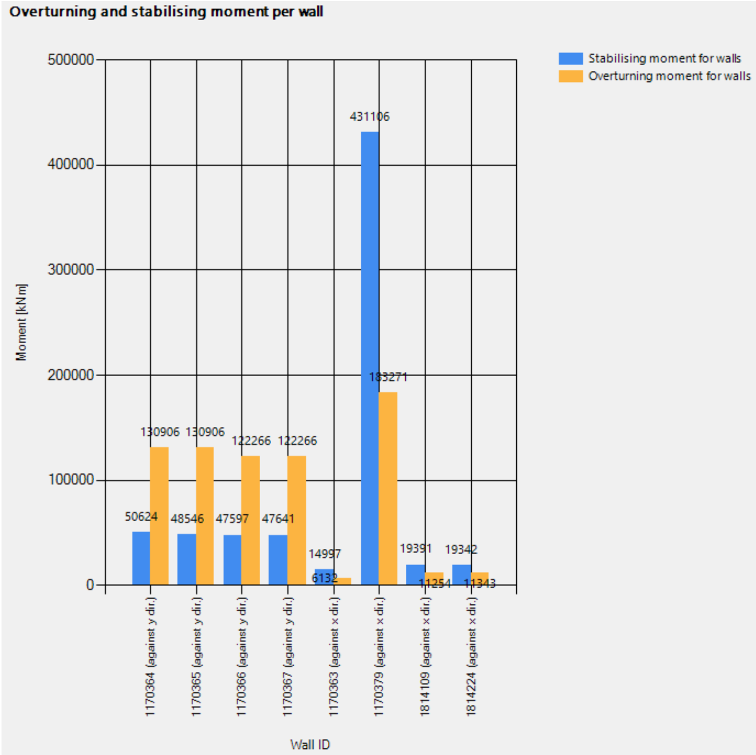


Figure 5.14: Overturning moment (right yellow bars) and stabilising moment (left blue bars) presented per wall in a bar chart (Wall index from left to right: 4, 1, 2, 3, 5, 8, 6 ,7).

5.2.3 Comparison considering only bending deformation or bending+shear deformation for relative stiffness on Veddesta

By default, Dynamo considers both bending and shear deformation when the relative stiffness is calculated for each shear wall. It was earlier written that both bending and shear deformation was included get the most accurate horizontal load distribution as possible. However, it can be seen by looking at figure 5.15 that there is almost no difference in the overturning moment for the different shear walls for Veddesta if bending and shear deformation or only bending deformation is included.

The shear stiffness has greatest impact on the lowest floors for the horizontal load distribution. Wind load has the lowest value there, and the lever-arm is also shortest there. These three factors result in the shear stiffness having low impact on a tall building like Veddesta. Thus, it can be concluded that it is sufficient to consider bending deformation only for Veddesta.

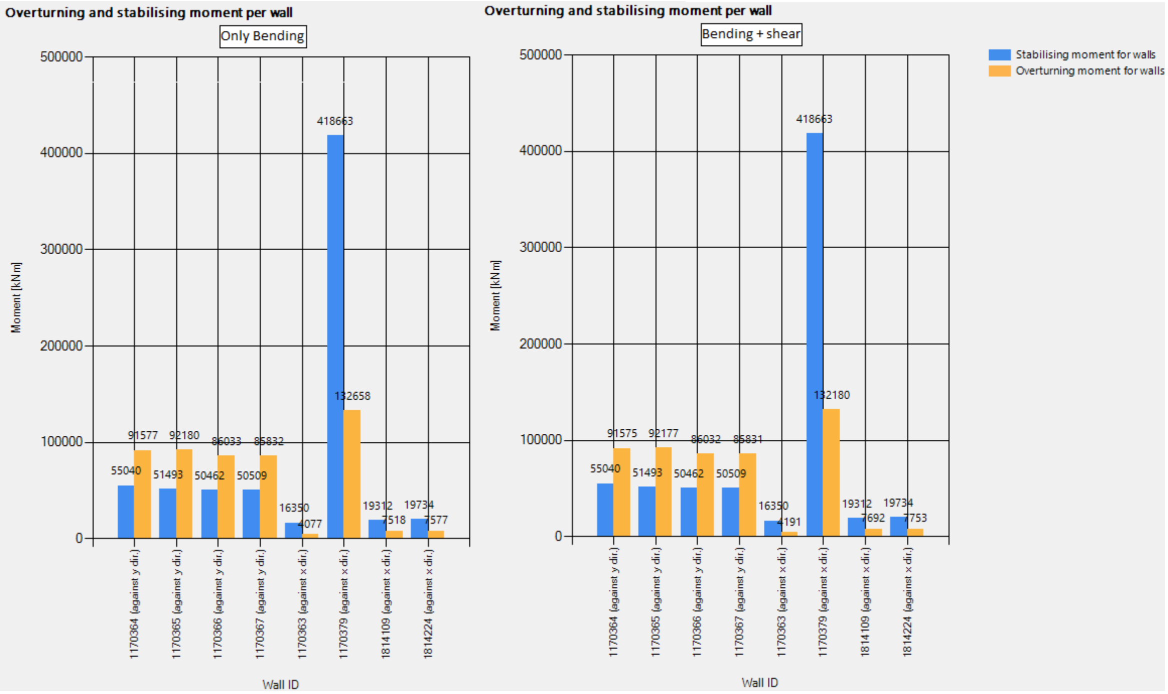


Figure 5.15: Comparison considering bending deformation (left) or bending and shear deformation for Veddesta. The wind load cases that resulted in highest load per shear wall was stated in table 5.1 (Wall index from left to right: 4, 1, 2, 3, 5, 8, 6 ,7).

5.3 Finding sufficient wall configuration for Veddesta's building height

5.3.1 New shear wall configuration for shear walls orientated in y-direction

With the current geometry according to reference floor plan drawing used in Dynamo, the shear walls exceed their capacity for wind load in the y-direction. An attempt is done to change the plan drawing and see if it is possible to find a configuration of the shear walls that can manage the horizontal load.

A new configuration of shear walls is optimised that manages the horizontal load. The changes that were made are:

- Change slab thickness from 250 to 350 mm
- Extend length of shear wall 1, 2, 3, 4 and moving them slightly (to fit the geometry better with the new wall lengths)
- Removing columns in gables and substituting them by shear walls with 300 mm thickness

5.3.2 New shear wall configuration for shear walls orientated in x-direction

With the current geometry according to reference floor plan drawing used in Dynamo, the shear walls are not close to their capacity in the x-direction.

A new configuration of shear walls is found that better optimised against horizontal load. The changes that were made are:

- Reduce thickness of shear wall 5, 6, 7 to 250 mm thickness.
- Reduce thickness of shear wall 8 to 350 mm thickness.
- Reduce length of shear wall 9 to 16.48 m

5.3.3 New floor plan

With changes from section 5.3.1 and 5.3.2, the modified plan drawing is created. The original plan drawing above the modified plan drawing is shown in figure 5.16.

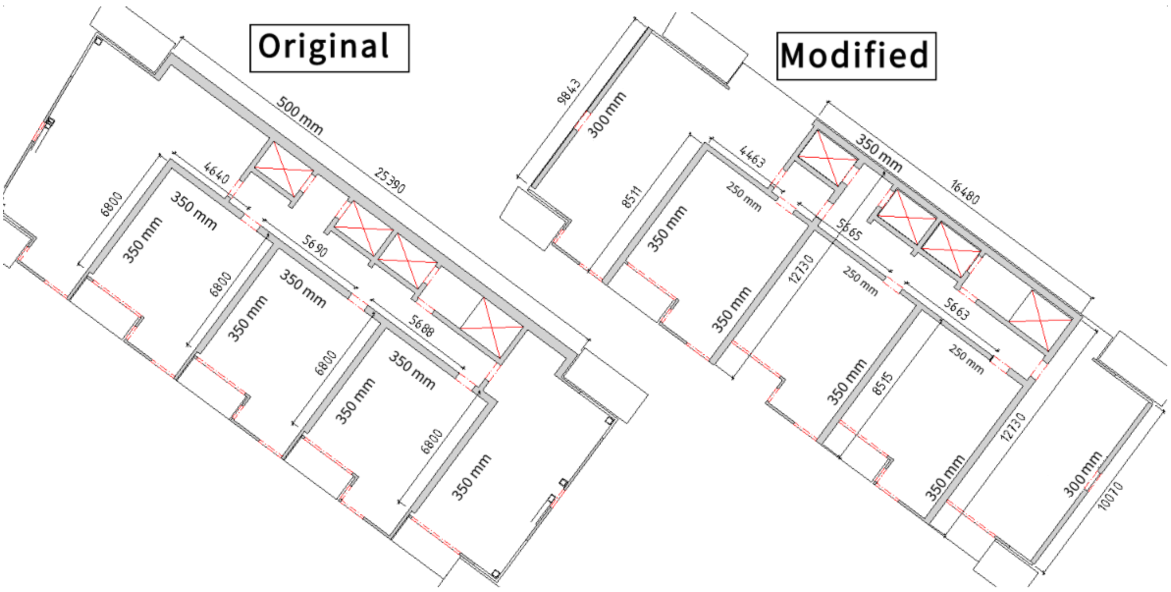


Figure 5.16: Original and modified plan drawing where shear wall dimensions are displayed for both configurations.

The new configuration has 6 shear walls in the y-direction, and 4 shear walls in the x-direction. The capacity of the walls follow Figure 5.17 where the first 6 bars are for shear walls in the y-direction and the last four are shear walls in the x-direction.

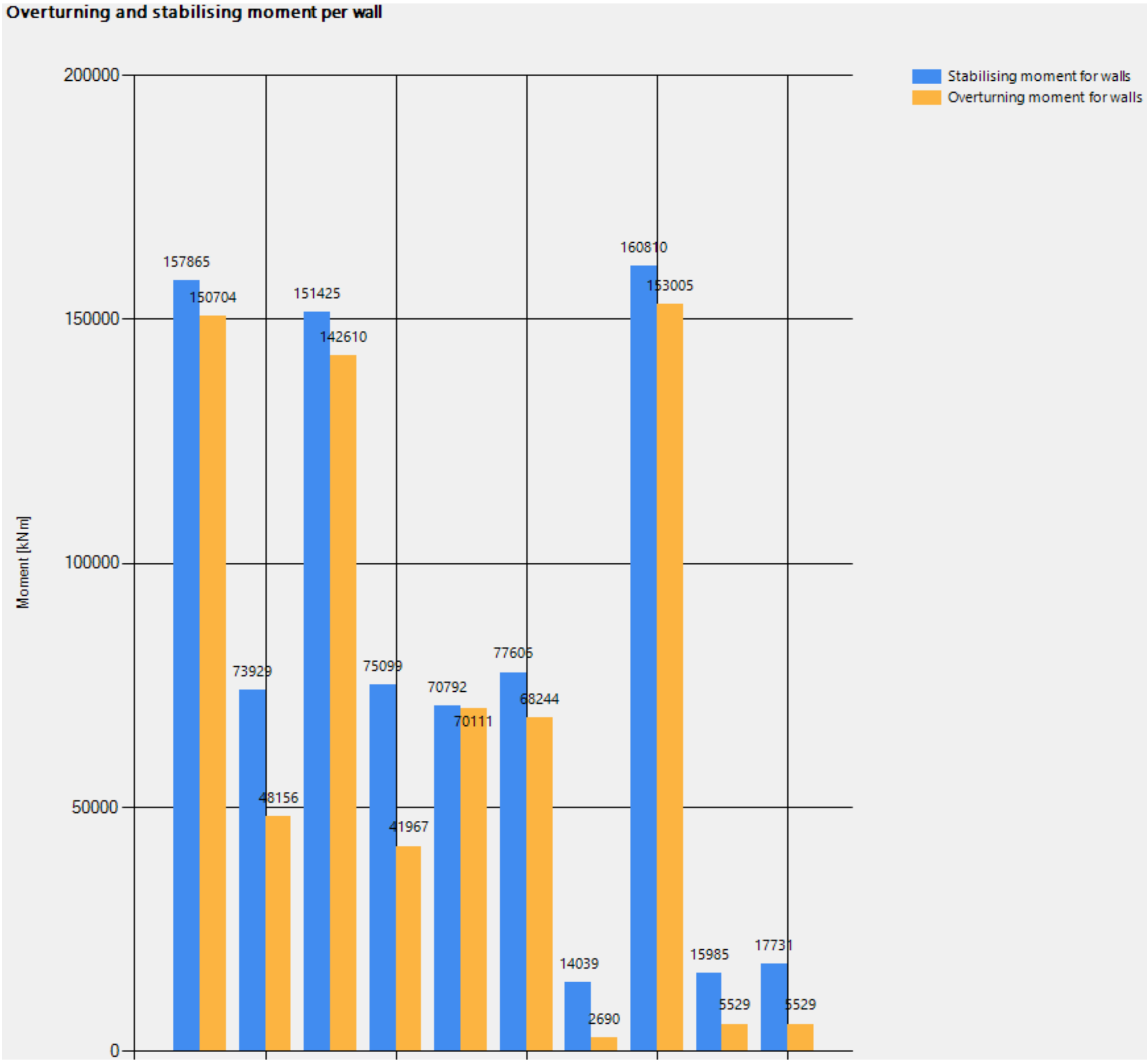


Figure 5.17: Overturning moment and stabilising moment for the modified plan drawing with 33 storey height (first 6 bar couples for shear walls against y-direction and last 4 bar couples for shear walls against x-direction.

6 Discussion and conclusion

6.1 Goal of the master thesis

The Dynamo script has been created with the idea that it should be as precise as possible while some simplifications were needed to make implementing the scripting in Dynamo doable.

Assumptions have been made to make the script as general as possible, while giving a satisfactory result. The output is presented so the calculations are easy to follow. This allows the user to trust the script output more, since they can easily follow the calculations.

The intention of the authors with the script was that it should be as automated as possible as there is as risk that the user of the script doesn't think its worth to use it if many manual inputs from the user are required. This was a problem that was avoided as much as possible but still some inputs are needed by the user. Deeper knowledge in Dynamo when making the Dynamo script might allow for fewer inputs and easier use.

6.2 Encountered Problems

Much time was spent on figuring out how the vertical load distribution could be automated while maintaining reasonable good representation of the actual vertical load distribution. Letting Dynamo script calculate influence area for vertical load bearing elements according to Voronoi partitioning proved to be a good idea and it requires minimal inputs from the user.

During initial test runs of the Dynamo script, the Revit model that the Dynamo script performed calculations based on, had the shear walls extending from foundation to top floor and were modelled as a single unit. With access to the Veddesta Revit file, it showed that the building model was built one floor at a time, and then copy pasted on top of each other. This made it more difficult to extract the walls height from Revit. This led to that a reference floor had to be created in Dynamo and then having this floor represent all floors in the building. The possibility of having different wall height was removed from the script which makes the script slightly more limited.

The idea of having Dynamo automatically optimising the shear walls placement, length and thickness was brought up during the work of the thesis. However, it was concluded that it is hard to do since it could be difficult to implement in an actual Revit file. There are usually limitations on how much changes that can be made for the shear walls as they have other functions than only maintaining horizontal stability, e.g. carrying vertical loads and being room divider for apartments. Making an optimisation in the Dynamo script that could consider those variables were deemed to be too difficult.

Finite element modelling of Veddesta was tried and it was difficult to construct a model that behaved according to theory. Vertical load bearing walls that are not supposed to take up any horizontal load did take up a significant portion of horizontal load even if they were modelled to not do so. That the vertical load bearing walls take up horizontal load also means that the shear centre in the FEM-model is adjusted accordingly which makes the result less correct. Based on this, it was concluded that a FEM-model analysis of Veddesta would not be included in this report as deviations in the FEM analysis from the Dynamo script could not be explained.

6.3 Veddesta case study

From the stability calculations of Veddesta in Dynamo, it was seen that the current configuration of the shear walls is not sufficient in terms of horizontal stability. A new configuration of the shear walls in the y-direction and slab for Veddesta were presented. This new configuration has enough horizontal stability capacity but it required many changes. These changes have kept most of the floor geometry intact, but requires 100 mm more concrete for the floor-slab on every floor, longer shear walls and two more shear walls added to the gables.

It was seen that many changes were necessary to maintain horizontal stability in the y-direction while in the x-direction, the shear walls length and thickness could be reduced vastly. As the shear walls also carry vertical load it is unclear how much they can be reduced in length/thickness or all together removed. Modifying the shear walls might also pose problems for the concrete slab as the slab might have longer span length with new configuration of shear walls. Changing the position and length of walls might affect the function of the building. Changes should be made with consultation of the architect.

For the x-direction, the walls did not utilise the capacity of the shear walls. By using the script we could change the dimensions and save material for the shear walls. Lowering the dimensions of walls without having the complete information about the project (e.g. acoustics, fire safety) may however not be a great idea, since we don't know why the original designer designed the walls this way. For the best result, the designer of Veddesta should use the script with the full knowledge of the parameters in which Veddesta should be within. All changes were made trying to keep the original look and wall/floor configuration of Veddesta as intact as possible.

On the gables there were both vertical load bearing wall and two columns. According to ELU Konsult the walls on the gables are not supposed to be vertical load bearing but in reality, they would carry some vertical load. In comparison to the columns, the walls on the gables are not supposed to carry much vertical load. This did not affect the horizontal stability calculations that much but it had great impact on the influence area for the columns and walls in the gables when it comes to the Voronoi diagram.

An improvement that we propose to increase stability without changing the original configuration is to try to add stabilising cores (preferably where the elevators are) in order to stiffen the building. This has not been verified.

6.4 Thoughts on script development in Dynamo and Revit

Dynamo and Revit where great tools for extracting data from the 3D model for calculations. The data was easy to follow with the visual interface inside the Dynamo script maker.

When it comes to the calculations, however, the coding needed in the Python language node in Dynamo posed quite some problems. The Python script is limited to the basic Python language, not allowing for important packages as for example *Numpy* to be used. This limits the way you can handle the data provided from the Revit model, and simple things like e.g. summarizing values in a list becomes much more time consuming.

In the Dynamo script, the relative stiffness of each shear wall was calculated on each floor of the building and this required a large amount of calculations for the script and thus taking a long time for the script to run. When comparing using only bending deformation or bending and shear deformation for Veddesta, it was seen that the difference in overturning moment was negligible. Using only bending stiffness for horizontal load distribution according to the proposed equation according to Eurocode would require less calculations as the bending stiffness is the same for a shear wall on all floors throughout the building. This would vastly reduce the computation time for the Dynamo script.

6.5 Further improvement of the scripts

The Dynamo script could potentially be improved by allowing different heights of the shear walls and different floor configurations for each storey. Stabilizing cores are very common in buildings, and would be interesting to be implement in the script. Currently the user can choose to model the stabilizing cores as single shear walls, but this results in a stiffness that is quite far from the true value, since the synergy given from the connected cross section will not be considered.

Implementing stabilising cores was investigated, but lead to some problems. Doing this would mean that shear deformation would be difficult to include in the calculations of the relative stiffness. However, for Veddesta, it was seen that there was almost no difference in the overturning moment if only bending was included in relative stiffness, compared to including both bending and shear. The option could be that the Dynamo script could be divided into two parts. One part only calculating the relative stiffness based on bending (script A) and the other part calculating relative stiffness including both bending and shear deformation (script B). This means that if only shear walls were used in a structure, script B could be used. If shear walls and stabilising cores were used together, the "simplified" script A only considering bending could be used.

Another improvement of the Dynamo script could be to include more controls in the script. That could for example include calculating the global buckling capacity. However the equations to analytically calculate global buckling capacity required very bold assumptions which were not realistic and thus it was deemed not necessary to implement.

The Dynamo script could also be improved to be able to calculate the ULS STR horizontal load for the shear walls and then being able to calculate the reinforcement needed for the shear wall, and by having access to the Revit API, being able to add 3D reinforcement to the shear wall in the 3D-model.

Adding capabilities to calculate the top displacement for the structure could be implemented as this is often a very important governing parameter when designing the horizontal stability system. Acceleration from wind load is also an important factor for tall and slender buildings. Making sure the acceleration does not exceed the comfort levels put on the buildings would be a good implementation.

6.6 Conclusions

It has proven that the created script in Dynamo is a suitable tool for the initial design phase when the placement of the shear walls needs to be verified in a fast way. The manual calculations that were done in Excel were very cumbersome and took long time to perform. The unique feature of the script is that the user can change the placement and dimensions of the shear wall in the actual Revit model and perform calculations on the shear walls using the Dynamo script. If an Excel sheet would be used, the user would have to manually go and adjust the Excel sheet for every little adjustment that is made in the Revit model. This really highlights the benefits by using of the created Dynamo script. The Dynamo script has just touched the surface of all capabilities computational design enables and most likely in the future, this area of structural engineering will become more prominent.

7 Appendix

7.1 Manual

Neither Erik Bolin, Emil Sjöstedt or Faculty of Engineering, (LTH) take responsibility for this program. Use the program with caution and always make control calculations.

A manual for using the script is presented in list below. To use the script, follow the steps in order.

1. Open the Revit file
2. If you have not downloaded the required Dynamo packages from the Dynamo library, download:
   (a) *Clockwork for Dynamo 2.X*
   (b) *Data-Shape*
   (c) *Shearwall package Emil and Erik*
3. If you don't have the parameters in the wall and column elements:
   (a) Open dynamo player
   (b) Check box *create parameters* and run the script (see Figure 7.2)
   (c) Select category to save said parameters (any category will work)
   (d) The parameters are now created in wall and column elements
   (e) Un-check box *create parameters*
   (f) Close dynamo player
4. In Revit, click on the shear walls, and in the properties tab, mark the *ShearWall* box
5. Click on the load-bearing walls, and in the properties tab, mark the *LoadBearingWalls* box
6. Click on the load-bearing columns, and in the properties tab, mark the *LoadBearingColumn* box
7. Open Dynamo player and enter (see Figure 7.1):
   (a) Wind speed
   (b) Terrain category
   (c) Floor height

    (d) Number of floors

8. In Dynamo player also select floor element by (see figure 7.1):
    (a) click ”select floor element”
    (b) open Revit
    (c) click of the floor element (*)
9. For Excel table and output figures, check the corresponding boxes found in the Dynamo player (see figure 7.3).
10. For voronoi figures, the following must be entered (see figure 7.4):
    (a) select view for Voronoi diagram to be placed in
    (b) select any filled region type
    (c) select fill pattern (*Solid Fill* is recommended)
11. Make sure the *Create parameters* box is un-checked.
12. Run the script

*Floor slab must be modelled as one single unit for Dynamo to work

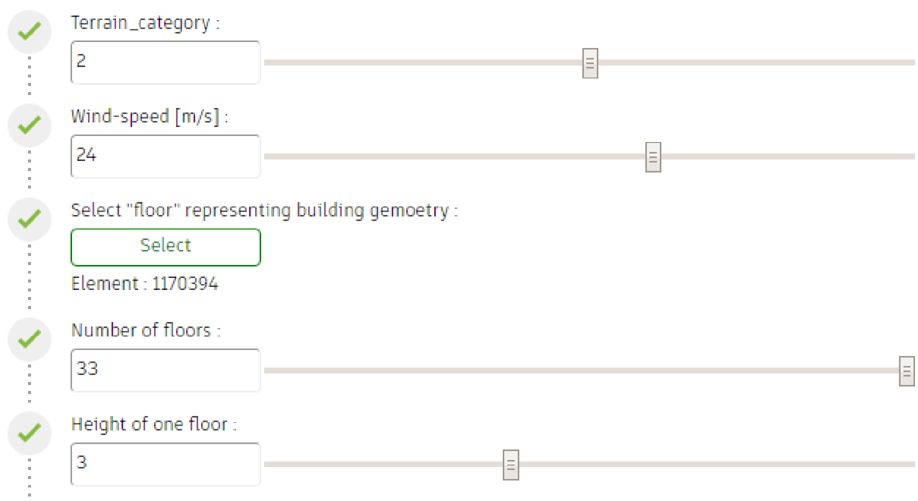


Figure 7.1: Selecting inputs in Dynamo player.

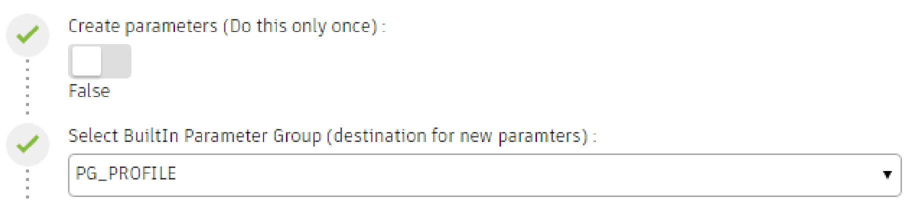


Figure 7.2: Selecting ”create parameters” and which category the parameters are to be saved in inside the Dynamo player.

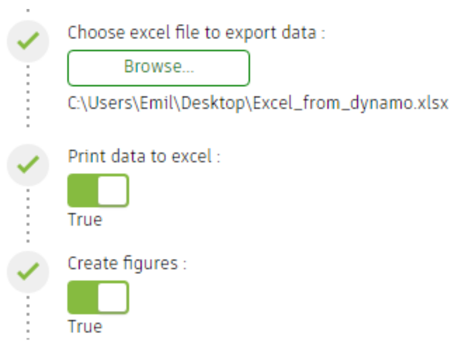


Figure 7.3: Selecting which outputs are to be presented in Dynamo player.

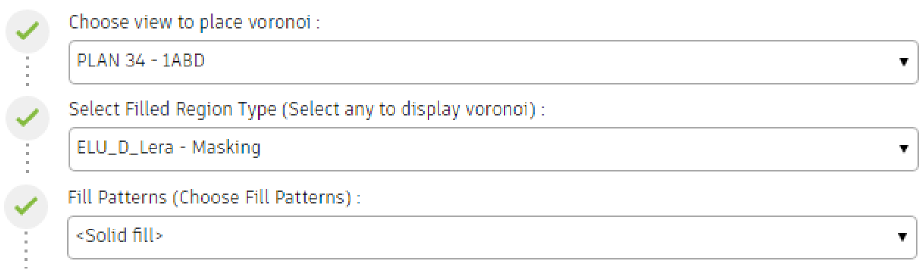


Figure 7.4: Filling in inputs in Dynamo player for displaying the Voronoi diagram in Revit.

7.2 Wind load over the height of the building

To have all the wind load calculations in one place, this chapter has some repeated information from chapter 4.2.

Method presented in EKS11 is used for the script.

Over the height of the building, the wind load is simplified into a evenly distributed load between each floor in the script. Each floor slab takes wind load equal to half the wind load from the floor above, and half the wind load from the floor below. The wind load is simplified into segments with uniform distributed load in each segment (see figure 7.5). These segments are based on the wind calculations from the EKS11.

The load w_e for the outer wind pressure is then described with equation 7.2 and the inner wind pressure is described with equation 7.1 [11].

$$w_i = q_p(z_i) \cdot c_{pi} \tag{7.1}$$

$$w_e = q_p(z_e) \cdot c_{pe} \tag{7.2}$$

The external pressure coefficient c_{pe} depend on the area of the loaded part. For design of load-bearing system of a building, the largest area of 10 m^2 is considered. The factor c_{pe} will have two coefficients, one from wind on windward side in zone D (see Figure 7.6) and one from leeward side in zone E. The coefficients are acting in unison seen from the stabilising system. The values are chosen from equations derived in chapter 4.2, presented in equation 7.3 to 7.10

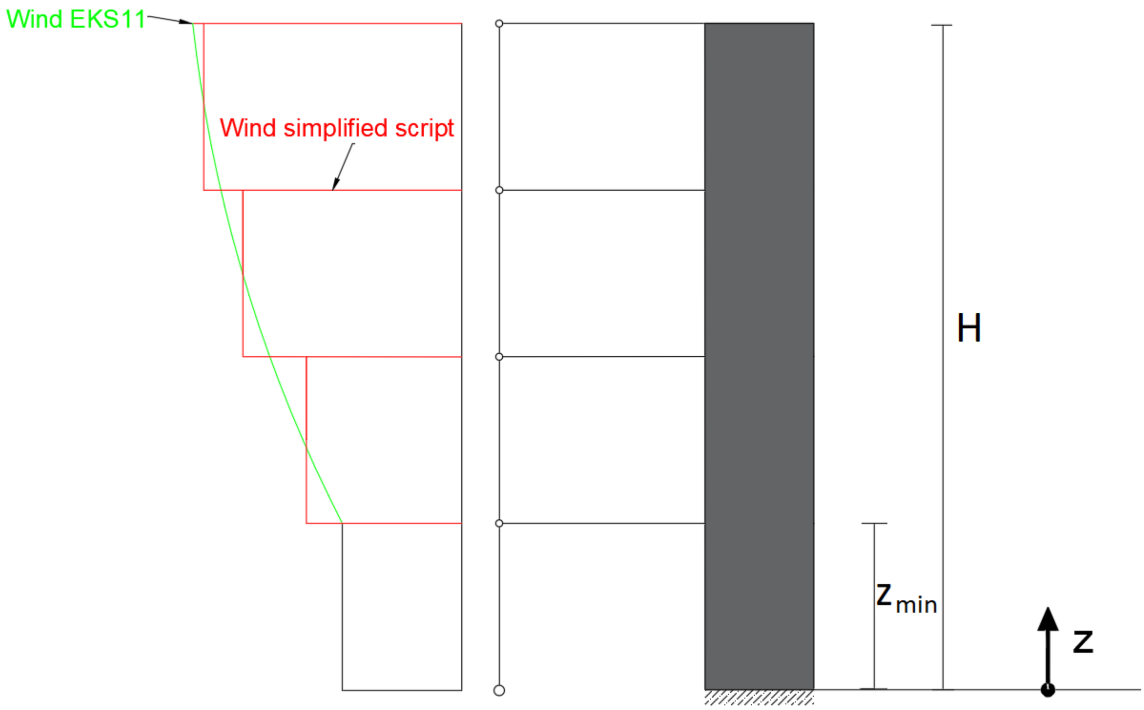


Figure 7.5: EKS11 wind load distribution (green line) compared to the simplified approach in the script (red blocks).

$$c_{pe,D} = -0.7 \text{ (for } H/d \leq 0.25) \tag{7.3}$$

$$c_{pe,E} = -0.3 \text{ (for } H/d \leq 0.25) \tag{7.4}$$

$$c_{pe,D} = -0.8 \text{ (for } H/d > 0.25 \text{ and } H/d \leq 1) \tag{7.5}$$

$$c_{pe,E} = -H/d \cdot 0.266667 - 0.23333 \text{ (for } H/d > 0.25 \text{ and } H/d \leq 1) \tag{7.6}$$

$$c_{pe,D} = -0.8 \text{ (for } H/d > 1 \text{ and } H/d \leq 5) \tag{7.7}$$

$$c_{pe,E} = -H/d \cdot 0.05 - 0.45 \text{ (for } H/d > 1 \text{ and } H/d \leq 5) \tag{7.8}$$

$$c_{pe,D} = -0.8 \text{ (for } H/d > 5) \tag{7.9}$$

$$c_{pe,E} = -H/d \cdot 0.05 - 0.45 \text{ (for } H/d > 5) \tag{7.10}$$

H is the height of the building and d is the depth of the building.

The internal wind pressure will be similar on both sides of the building, acting in opposite directions. For stability calculations, the internal wind pressure is therefore not deemed necessary to consider since they will cancel each other out.

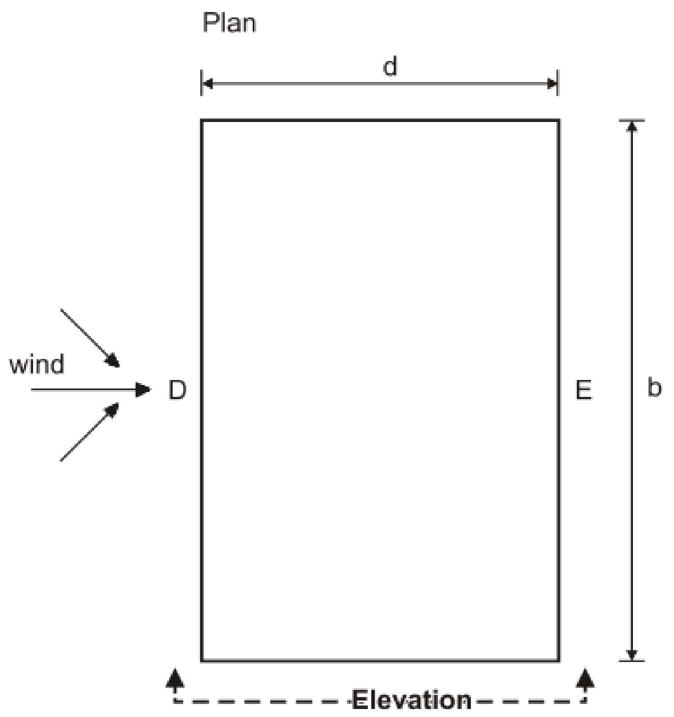


Figure 7.6: Building from above with wind-zones (source: [11], edited).

The peak velocity pressure $q_p(z)$ at the height z is decided though equation 7.11 from Swedish national document EKS11 [10].

$$q_p(z) = [1 + 2 \cdot k_p \cdot I_v(z)] \cdot [k_r \cdot \ln{(z/z_o)} \cdot c_o(z)]^2 \cdot q_b \tag{7.11}$$

Where q_b is the basic velocity pressure given by equation 7.23.

$$q_b(h) = 1/2 \cdot \rho \cdot v_b^2 \tag{7.12}$$

Where ρ is the air density, v_b is the basic wind velocity. A recommended value of the air density ρ is given as 1.25 kg/m^3 [11], and the basic wind velocity is read from figure 7.7 in Sweden.

$I_v(z)$ is the turbulence intensity at height z [11].The turbulence intensity is decided from equation 7.13 [10] and 7.14 [11] depending on the reference height.

$$I_v(z) = \frac{1}{c_o(z) \cdot \ln{(z/z_0))}} \text{ for } z_{min} \leq z \leq z_{max} \tag{7.13}$$

$$I_v(z) = I_v(z_{min}) \text{ for } z \leq z_{min} \tag{7.14}$$

$c_o(z)$ is the topography coefficient depending on a range of factors connected on the surroundings of the building. The value is in Sweden not taken into account, and therefore given a value of 1.0 [10]. k_l is the turbulence factor, and is recommended to use the value of 1.0 [11].

z_{min} is a height interval depending on the terrain category. z_0 is the roughness length, also depending on the terrain category [11]. Table 7.1 presents values of z_{min} for the different terrains. z_{max} is the maximum height, given a value of 200 m for all equations in this chapter. This means the equations being used are not suitable for taller buildings. This will not be a problem for this thesis, as buildings that tall will require other forms of horizontal stabilisation than shear walls anyway.

Table 7.1: Minimum height for different terrain categories (Source: [11])

Terrain category		z_0 m	z_{min} m
0	Sea or coastal area exposed to the open sea	0,003	1
I	Lakes or flat and horizontal area with negligible vegetation and without obstacles	0,01	1
II	Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights	0,05	2
III	Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)	0,3	5
IV	Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m	1,0	10
NOTE: The terrain categories are illustrated in A.1.			

The mean wind velocity is calculated with equation 7.15 [11].

$$v_m(z) = c_r(z) \cdot c_o(z) \cdot v_b \tag{7.15}$$

Where $c_r(z)$ is a roughness factor determined through equations 7.16 and 7.17 depending on the reference height z [11]. Reference average wind velocity v_b is presented in figure 7.7.

$$c_r(z) = k_r \cdot \ln(z/z_0) \text{ for } z_{min} \leq z \leq z_{max} \tag{7.16}$$

$$c_r(z) = c_r(z_{min}) \text{ for } z \leq z_{min} \tag{7.17}$$

where k_r is a terrain factor calculated based on the roughness length z_o and the roughness length at 0.05 m of terrain category 2 $z_{0,II}$. see equation 7.25 for equation ([11].

$$k_r = 0.19 \cdot (\frac{z_o}{z_{o,II}})^{0.07} \tag{7.18}$$

The dynamic response of the building is being considered in the peak factor k_p. For static buildings, where the dynamic factor is not considered, k_p is given a value of 3. To keep the script general and simple, the dynamic factor has not been taken into account, and is therefore given the value of 3 for all building geometries.

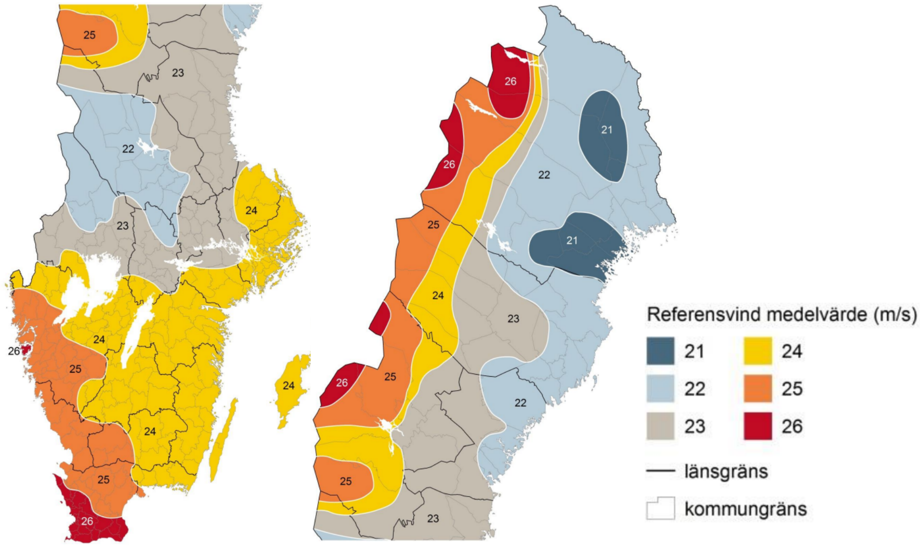


Figure 7.7: Map with reference average wind velocity over Sweden source:Source: [10], edited)

7.3 Hand calculations of case study Veddesta

For the case study object Veddesta, hand calculations are done for overturning moment and stabilising moment. Hand calculations are done in order to verify the script in Dynamo. The hand calculations should give the same result as Dynamo since they are based on same equations. The hand calculations will only be done for horizontal loading in the y-direction with symmetric wind load.

The hand calculations are done using Excel. For wind load and stabilising moment, a little more in depth explanations are included but for the other parts, only screenshots from Excel are provided. All the values from the provided screenshots from Excel are calculated using equations covered in the Theory section. In hand calculations, origo is set to lower left corner of slab geometry.

The simplified planar layout for the building can be seen in figure 7.8

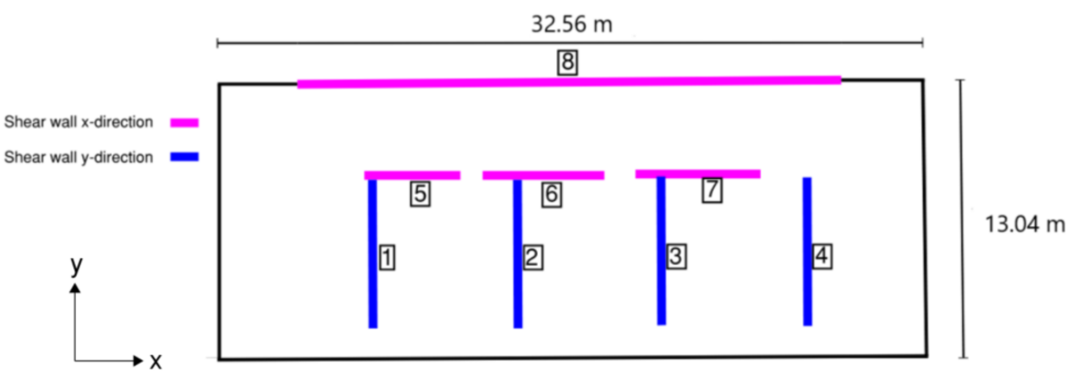


Figure 7.8: Simplified floor layout for Veddesta.

Geometry of Veddesta and distance to load centre are seen in Table 7.2.

Table 7.2: Geometry and distance to load centre form origo.

Geometry building	m
Width	32.56
Depth	13.04

Distance to load centre from origo	m
x_ref_LC	16.28
y_ref_LC	6.52

Material and geometry data for the different shear walls are seen in Table 7.3 and Table 7.4. The distance from origo to shear wall 1-8 to their distance in x and y-direction from reference point.

Table 7.3: Shear wall information.

Wall	Orientation	Length wall [m]	Height wall [m]	Young's modulus[MPa]	Thickness [m]
1	y	6.8	3	33	0.35
2	y	6.8	3	33	0.35
3	y	6.8	3	33	0.35
4	y	6.8	3	33	0.35
5	x	4.64	3	33	0.35
6	x	5.69	3	33	0.35
7	x	5.705	3	33	0.35
8	x	25.04	3	33	0.5

Table 7.4: Distance from origo to shear wall centroid for the different shear walls.

	Distance from origo to centroid [m]	Distance from origo to centroid [m]
Wall	x_ref_i	y_ref_i
1	6.914	
2	13.614	
3	20.314	
4	27.014	
5	0	8.575
6	0	8.575
7	0	8.575
8	0	12.79

7.3.1 Wind load

Veddesta has 33 storeys and each floor has a height of 3 m. That gives that the total height h of the building is $33 \cdot 3$=99 m. The terrain category is II and thus $z_0 = 0.05$

and $z_{min} = 2$. $c_0(z)$ is the topography coefficient and is set to 1.0. Since Veddesta is located adjacent to Stockholm region, that gives $v_b = 24$ m/s.

The wind load is calculated below for the first floor. For this, the variable z_{min} is between the influence height. To determine the wind load the wind load is calculated for $z = z_{min}$ and $z = 4.5$.

The turbulence intensity $I_v(z)$ is calculated :

$$I_v(z) = \frac{1}{c_o(z) \cdot \ln{(z/z_0)}} \tag{7.19}$$

$$I_v(z_{min}) = \frac{1}{1 \cdot \ln{(\frac{2}{0.05})}} = 0.27 \tag{7.20}$$

$$I_v(4.5) = \frac{1}{1 \cdot \ln{(\frac{4.5}{0.05})}} = 0.22 \tag{7.21}$$

$$I_v(z) = I_v(z_{min}) \text{ for } z \leq z_{min} \tag{7.22}$$

The basic velocity pressure is calculated:

$$q_b(h) = 1/2 \cdot \rho \cdot v_b^2 = \frac{1}{2} \cdot 1.25 \cdot 24^2 = 360 \text{ N/m}^2 \tag{7.23}$$

The ratio between height and depth H/d is:

$$H/d = 99/13.04 = 7.59 \tag{7.24}$$

H/d is used with equations 7.3 to 7.10. For zone D $c_{pe,10}$=0.8 For zone E $c_{pe,10}$=0.829 (note the sign). These two are added together to form a common load.

The terrain factor k_r is calculated for terrain category 2:

$$k_r = 0.19 \cdot (\frac{z_o}{z_{o,II}})^{0.07} = 0.19 \cdot (\frac{0.05}{0.05})^{0.07} = 0.19 \tag{7.25}$$

The peak wind pressure is calculated with equation 7.26. Inserted values are presented in equation 7.27.

$$q_p(z) = [1 + 2 \cdot k_p \cdot I_v(z)] \cdot [k_r \cdot \ln{(z/z_o)} \cdot c_o(z)]^2 \cdot q_b \tag{7.26}$$

$$q_p(z_{min}) = [1 + 2 \cdot 3 \cdot 0.27] \cdot [0.19 \cdot \ln{(2/0.05)} \cdot 1.0]^2 \cdot 360 = 463.4 \text{ N/m}^2 \tag{7.27}$$

$$q_p(4.5) = [1 + 2 \cdot 3 \cdot 0.22] \cdot [0.19 \cdot \ln{(4.5/0.05)} \cdot 1.0]^2 \cdot 360 = 613.7 \text{ N/m}^2 \tag{7.28}$$

The wind load w_e is calculated:

$$w_e(z_{min}) = q_p(z_{min}) \cdot c_{pe,10} = 464.4 \cdot (0.8 + 0.83) = 756.9 \text{ N/m}^2 \tag{7.29}$$

$$w_e(4.5) = q_p(4.5) \cdot c_{pe,10} = 613.5 \cdot (0.8 + 0.83) = 1000.3 \text{ N/m}^2 \tag{7.30}$$

In figure 7.5 the wind load per floor is shown where wind load $w_e(z)$ has been integrated over the influence height of each floor.

Calculations for the first floors are presented in equation 7.31.

$$\text{First floor: } \int_{1.5}^{4.5} w_e(z)\, dz = \int_{1.5}^{z_{min}} w_e(z_{min})\, dz + \int_{z_{min}}^{4.5} w_e(4.5)\, dz \tag{7.31}$$

$= 756.9 \cdot (2 - 1.5) + 1000.3 \cdot (4.5 - 2) = 2.880$ kN/m

The result matches the calculated value from the script presented in Figure 7.5.

For the rest of the floors, where z_{min} is not between the influence height, the influence height is split with half the integrated zone above and half the integrated zone below. Calculations for floor 2 are presented in equation 7.3.1. For equation two, wind load for height 7.5 needs to be determined.

$$I_v(7.5) = \frac{1}{1 \cdot \ln\left(\frac{7.5}{0.05}\right)} = 0.200 \tag{7.32}$$

$$q_p(7.5) = [1 + 2 \cdot 3 \cdot 0.22] \cdot [0.19 \cdot \ln(7.5/0.05) \cdot 1.0]^2 \cdot 360 = 717.0 \text{ N/m}^2 \tag{7.33}$$

$$w_e(z_{min}) = q_p(z_{min}) \cdot c_{pe,10} = 717.0 \cdot (0.8 + 0.83) = 1168.7 \text{ N/m}^2 \tag{7.34}$$

$$\text{Second floor: } \int_{4.5}^{7.5} w_e(z)\, dz = \int_{4.5}^{6} w_e(4.5)\, dz + \int_{6}^{7.5} w_e(7.5)\, dz \tag{7.35}$$

$= 1000.3 \cdot (6 - 4.5) + 1168.7 \cdot (7.5 - 6) = 3.253$ kN/m

The result matches the calculated value from the script presented in Table 7.5.

Table 7.5: Wind load per floor from first to last floor with values from Veddesta.

Height [m]	Wind load y direction [kN/m]	Wind load y-direction[kN]
3	2.880	93.771
6	3.253	105.933
9	3.680	119.821
12	3.990	129.918
15	4.236	137.934
18	4.441	144.615
21	4.618	150.360
24	4.773	155.410
27	4.912	159.922
30	5.037	164.005
33	5.152	167.736
36	5.257	171.175
39	5.355	174.365
42	5.447	177.341
45	5.532	180.132
48	5.613	182.760
51	5.689	185.245
54	5.762	187.601
57	5.831	189.842
60	5.896	191.979
63	5.959	194.022
66	6.019	195.978
69	6.077	197.856
72	6.132	199.662
75	6.186	201.401
78	6.237	203.078
81	6.287	204.698
84	6.335	206.265
87	6.381	207.781
90	6.427	209.252
93	6.470	210.678
96	6.513	212.063
99	3.267	106.373

7.3.2 Calculating relative stiffness

In Table 7.6 and 7.7, the relative stiffness have been calculated for the shear walls.

Table 7.6: Relative stiffness for shear walls orientated in y-direction.

	Relative stiffness considering bending and shear deformation					ΣS_yi for each floor
Floor	Wall 1 S_y1	Wall 2 S_y2	Wall 3 S_y3	Wall 4 S_y4	Height [m]	ΣS_yi
Floor 1	6.929	6.929	6.929	6.929	3	27.714
Floor 2	2.141	2.141	2.141	2.141	6	8.564
Floor 3	0.872	0.872	0.872	0.872	9	3.488
Floor 4	0.423	0.423	0.423	0.423	12	1.694
Floor 5	0.233	0.233	0.233	0.233	15	0.932
Floor 6	0.141	0.141	0.141	0.141	18	0.563
Floor 7	0.091	0.091	0.091	0.091	21	0.364
Floor 8	0.062	0.062	0.062	0.062	24	0.248
Floor 9	0.044	0.044	0.044	0.044	27	0.176
Floor 10	0.032	0.032	0.032	0.032	30	0.130
Floor 11	0.024	0.024	0.024	0.024	33	0.098
Floor 12	0.019	0.019	0.019	0.019	36	0.076
Floor 13	0.015	0.015	0.015	0.015	39	0.060
Floor 14	0.012	0.012	0.012	0.012	42	0.048
Floor 15	0.010	0.010	0.010	0.010	45	0.039
Floor 16	0.008	0.008	0.008	0.008	48	0.032
Floor 17	0.007	0.007	0.007	0.007	51	0.027
Floor 18	0.006	0.006	0.006	0.006	54	0.023
Floor 19	0.005	0.005	0.005	0.005	57	0.019
Floor 20	0.004	0.004	0.004	0.004	60	0.017
Floor 21	0.004	0.004	0.004	0.004	63	0.014
Floor 22	0.003	0.003	0.003	0.003	66	0.013
Floor 23	0.003	0.003	0.003	0.003	69	0.011
Floor 24	0.002	0.002	0.002	0.002	72	0.010
Floor 25	0.002	0.002	0.002	0.002	75	0.009
Floor 26	0.002	0.002	0.002	0.002	78	0.008
Floor 27	0.002	0.002	0.002	0.002	81	0.007
Floor 28	0.002	0.002	0.002	0.002	84	0.006
Floor 29	0.001	0.001	0.001	0.001	87	0.005
Floor 30	0.001	0.001	0.001	0.001	90	0.005
Floor 31	0.001	0.001	0.001	0.001	93	0.004
Floor 32	0.001	0.001	0.001	0.001	96	0.004
Floor 33	0.001	0.001	0.001	0.001	99	0.004

Table 7.7: Relative stiffness for shear walls orientated in x-direction.

Relative stiffness considering bending and shear deformation				ΣS_xi for each floor
Vägg 5 S_x5	Vägg 6 S_x6	Vägg 7 S_x7	Vägg 8 S_x8	ΣS_xi
3.824	5.328	5.349	45.045	59.545
0.922	1.471	1.479	21.321	25.193
0.330	0.561	0.565	13.054	14.510
0.150	0.263	0.265	8.786	9.465
0.080	0.142	0.143	6.210	6.575
0.047	0.085	0.085	4.530	4.747
0.030	0.054	0.055	3.384	3.524
0.020	0.037	0.037	2.579	2.674
0.014	0.026	0.026	2.000	2.067
0.010	0.019	0.019	1.575	1.624
0.008	0.014	0.015	1.259	1.296
0.006	0.011	0.011	1.019	1.047
0.005	0.009	0.009	0.834	0.856
0.004	0.007	0.007	0.690	0.708
0.003	0.006	0.006	0.577	0.591
0.003	0.005	0.005	0.486	0.498
0.002	0.004	0.004	0.413	0.424
0.002	0.003	0.003	0.354	0.363
0.002	0.003	0.003	0.305	0.313
0.001	0.002	0.002	0.265	0.271
0.001	0.002	0.002	0.232	0.237
0.001	0.002	0.002	0.203	0.208
0.001	0.002	0.002	0.179	0.184
0.001	0.001	0.001	0.159	0.163
0.001	0.001	0.001	0.142	0.145
0.001	0.001	0.001	0.127	0.130
0.001	0.001	0.001	0.114	0.116
0.000	0.001	0.001	0.102	0.105
0.000	0.001	0.001	0.093	0.095
0.000	0.001	0.001	0.084	0.086
0.000	0.001	0.001	0.076	0.078
0.000	0.001	0.001	0.070	0.071
0.000	0.001	0.001	0.064	0.065

In Table 7.8, the distance in x-direction from wall 1, 2, 3, 4 to shear centre and distance between load centre and shear centre are seen.

Table 7.8: Distance between e_x and x_i.

Distance between LC to SC on each floor x-dir	Distance in x-dir from SC to wall i			
e_x	x_1	x_2	x_3	x_4
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050
-0.684	-10.050	-3.350	3.350	10.050

In Table 7.9, the distance in y-direction from wall 1, 2, 3, 4 to shear centre and distance (y-direction) between load centre and shear centre are seen. One column also shows the the sum of a product with relative stiffness and distance between shear centre to wall i.

Table 7.9: Distance e_y, y_i and product of relative stiffnes and distance.

Distance between LC	Distance in y-dir from SC to wall i				
e_y	y_5	y_6	y_7	y_8	ΣS_yi*x_i^2+S_xi*y_i^2
-5.244	-3.189	-3.189	-3.189	1.026	1750.003
-5.622	-3.567	-3.567	-3.567	0.648	538.746
-5.847	-3.792	-3.792	-3.792	0.423	219.013
-5.968	-3.913	-3.913	-3.913	0.302	106.236
-6.036	-3.981	-3.981	-3.981	0.234	58.446
-6.077	-4.022	-4.022	-4.022	0.193	35.250
-6.103	-4.048	-4.048	-4.048	0.167	22.778
-6.121	-4.066	-4.066	-4.066	0.149	15.522
-6.134	-4.079	-4.079	-4.079	0.136	11.032
-6.143	-4.088	-4.088	-4.088	0.127	8.112
-6.150	-4.095	-4.095	-4.095	0.120	6.134
-6.155	-4.100	-4.100	-4.100	0.115	4.748
-6.159	-4.104	-4.104	-4.104	0.111	3.749
-6.162	-4.107	-4.107	-4.107	0.108	3.011
-6.165	-4.110	-4.110	-4.110	0.105	2.454
-6.167	-4.112	-4.112	-4.112	0.103	2.026
-6.169	-4.114	-4.114	-4.114	0.101	1.692
-6.171	-4.116	-4.116	-4.116	0.099	1.427
-6.172	-4.117	-4.117	-4.117	0.098	1.215
-6.173	-4.118	-4.118	-4.118	0.097	1.043
-6.174	-4.119	-4.119	-4.119	0.096	0.902
-6.175	-4.120	-4.120	-4.120	0.095	0.785
-6.176	-4.121	-4.121	-4.121	0.094	0.687
-6.176	-4.121	-4.121	-4.121	0.094	0.605
-6.177	-4.122	-4.122	-4.122	0.093	0.536
-6.177	-4.122	-4.122	-4.122	0.093	0.476
-6.178	-4.123	-4.123	-4.123	0.092	0.426
-6.178	-4.123	-4.123	-4.123	0.092	0.382
-6.179	-4.124	-4.124	-4.124	0.091	0.344
-6.179	-4.124	-4.124	-4.124	0.091	0.311
-6.179	-4.124	-4.124	-4.124	0.091	0.282
-6.179	-4.124	-4.124	-4.124	0.091	0.256
-6.180	-4.125	-4.125	-4.125	0.090	0.234
				Σ	**2798.860**

7.3.3 Stabilising moment

The stabilising moment from self-weight for shear walls are calculated using equation:

$$M_{stab,self-weight,shearwall,i} = \gamma_{concrete} \cdot t_{shearwall,i} \cdot H_{building} \cdot l_{shearwall,i} \cdot \frac{l_{shearwall,i}}{2} \quad (7.36)$$

Where:
$\gamma_{concrete}$ is 25 kN/m^3
$t_{shearwall,i}$ is the thickness of shear wall i
$H_{building}$ is the height of the building
$l_{shearwall,i}$ is the length of shear wall i
$\frac{l_{shearwall,i}}{2}$ is the lever arm

The stabilising moment from self-weight of concrete slab on shear wall i is calculated using equation:

$$M_{stab,self-weight,floorslab,i} = \gamma_{concrete} \cdot t_{slab} \cdot A_{influencearea,i} \cdot \frac{l_{shearwall,i}}{2} \cdot n_{floors} \quad (7.37)$$

Where:
t_{slab} is thickness of the concrete slab
$A_{influencearea}$ is the influence area for shear wall i
$n_{storeys}$ is the number floors of the building

The influence area according to Voronoi (from Dynamo script) are seen in Table 7.10.

Table 7.10: Influence area presented from Voronoi diagram with corresponding material data.

Influence area Voronoi		
Wall 1	48.371	m^2
Wall 2	46.856	m^2
Wall 3	46.925	m^2
Wall 4	52.989	m^2

Selfweight concrete	25	kN/m^3
Slab thickness	0.25	m

The stabilising moment due to self weight of shear walls and self weight from concrete slab are seen in Table 7.11

Table 7.11: Stabilising moment from self weight of walls and concrete slabs.

Stabilising moment M__stab self weight walls	M_stab Wall 1	M_stab Wall 2	M_stab Wall 3	M_stab Wall 4
[kNm]	20027.7	20027.7	20027.7	20027.7

Stabilising moment M_stab self weight concrete	M_stab Wall 1	M_stab Wall 2	M_stab Wall 3	M_stab Wall 4
[kNm]	33920.16375	32857.77	32906.15625	37158.53625

7.3.4 Imperfection load

Values used for calculating imperfection load are shown in Table 7.12.

Table 7.12: Values used for calculating imperfection load.

Imperfection load variables		
θ_0	0.005	
α_h	0.2010075631	Must be between 2/3 and 1
a_h	0.6666666667	α_h set to 2/3 since original value not between limits
m	6	Vertical members contributing to the total effect (number of columns)
α_m,1	0.7637626158	Reduction factor for number of members (6 columns)
α_m_2	0.7359800722	Reduction factor for number of members (2*6 columns)
θ_i,1	0.002545875386	This value used on top floor since columns only below floor
θ_i_2	0.002453266907	This value used on all other floors

In Table 7.13, under Excel column "Imperfection load multiplied with θ_i", the design imperfection load subjecting every floor is presented. The imperfection load is added together with design value of wind load.

Table 7.13: Imperfection load calculated for Veddesta

Self-weigh concrete slab, G_d [kN]	2620.0625	Dead load from concrete slab for one floor (Design value) (1.1*25*0.25*A_slab)

Design value of imperfection load		**Imperfection load multiplied with θ_i**
Floor	**Upper and lower ver**	**H_i [kN]**
Floor 1	86462.0625	6.427712626
Floor 2	83842	6.427712626
Floor 3	81221.9375	6.427712626
Floor 4	78601.875	6.427712626
Floor 5	75981.8125	6.427712626
Floor 6	73361.75	6.427712626
Floor 7	70741.6875	6.427712626
Floor 8	68121.625	6.427712626
Floor 9	65501.5625	6.427712626
Floor 10	62881.5	6.427712626
Floor 11	60261.4375	6.427712626
Floor 12	57641.375	6.427712626
Floor 13	55021.3125	6.427712626
Floor 14	52401.25	6.427712626
Floor 15	49781.1875	6.427712626
Floor 16	47161.125	6.427712626
Floor 17	44541.0625	6.427712626
Floor 18	41921	6.427712626
Floor 19	39300.9375	6.427712626
Floor 20	36680.875	6.427712626
Floor 21	34060.8125	6.427712626
Floor 22	31440.75	6.427712626
Floor 23	28820.6875	6.427712626
Floor 24	26200.625	6.427712626
Floor 25	23580.5625	6.427712626
Floor 26	20960.5	6.427712626
Floor 27	18340.4375	6.427712626
Floor 28	15720.375	6.427712626
Floor 29	13100.3125	6.427712626
Floor 30	10480.25	6.427712626
Floor 31	7860.1875	6.427712626
Floor 32	5240.125	6.427712626
Floor 33	2620.0625	6.670352629

7.3.5 Horizontal load on each wall on each floor

The horizontal load on each shear wall due to wind load (from Table 7.5, multiplied with 1.5) and imperfection load (from Table 7.13) can be seen in Table 7.14. The load subjecting each wall is the design value. The horizontal load on each shear wall is calculated considering torsion.

Table 7.14: Horizontal load to each wall at each floor.

	Horizontal load to each wall at each floor				
Floor	Wall 1 [kN]	Wall 2 [kN]	Wall 3 [kN]	Wall 4 [kN]	Height [m]
Floor 1	40.774	38.105	35.437	32.768	3.0
Floor 2	45.848	42.837	39.826	36.815	6.0
Floor 3	51.635	48.238	44.841	41.444	9.0
Floor 4	55.842	52.165	48.488	44.811	12.0
Floor 5	59.181	55.282	51.383	47.484	15.0
Floor 6	61.963	57.879	53.796	49.712	18.0
Floor 7	64.355	60.113	55.871	51.629	21.0
Floor 8	66.457	62.076	57.695	53.314	24.0
Floor 9	68.336	63.830	59.325	54.820	27.0
Floor 10	70.035	65.418	60.800	56.182	30.0
Floor 11	71.589	66.868	62.148	57.428	33.0
Floor 12	73.020	68.205	63.390	58.575	36.0
Floor 13	74.347	69.445	64.542	59.640	39.0
Floor 14	75.586	70.602	65.618	60.633	42.0
Floor 15	76.748	71.687	66.626	61.565	45.0
Floor 16	77.842	72.709	67.575	62.442	48.0
Floor 17	78.876	73.674	68.473	63.271	51.0
Floor 18	79.857	74.590	69.324	64.058	54.0
Floor 19	80.789	75.461	70.134	64.806	57.0
Floor 20	81.679	76.292	70.906	65.519	60.0
Floor 21	82.529	77.086	71.644	66.201	63.0
Floor 22	83.343	77.847	72.351	66.854	66.0
Floor 23	84.125	78.577	73.029	67.481	69.0
Floor 24	84.876	79.279	73.681	68.084	72.0
Floor 25	85.600	79.955	74.310	68.664	75.0
Floor 26	86.298	80.607	74.916	69.224	78.0
Floor 27	86.972	81.237	75.501	69.765	81.0
Floor 28	87.624	81.846	76.067	70.288	84.0
Floor 29	88.256	82.435	76.615	70.794	87.0
Floor 30	88.867	83.007	77.146	71.285	90.0
Floor 31	89.461	83.561	77.661	71.761	93.0
Floor 32	90.038	84.100	78.162	72.224	96.0
Floor 33	46.120	43.078	40.037	36.995	99.0

7.3.6 Overturning moment

In Table 7.15, the horizontal load to each wall at each floor have been multiplied with their respective lever arm.

Table 7.15: Overturning moment M_e for each wall [kNm].

Overturning moment M_overturn				
Floor	**M_overturn Wall 1**	**M_overturn Wall 2**	**M_overturn Wall 3**	**M_overturn Wall 4**
Floor 1	122.322	114.316	106.310	98.304
Floor 2	275.088	257.023	238.958	220.893
Floor 3	464.717	434.144	403.571	372.998
Floor 4	670.104	625.979	581.853	537.728
Floor 5	887.710	829.226	770.742	712.259
Floor 6	1115.328	1041.826	968.324	894.823
Floor 7	1351.449	1262.370	1173.290	1084.210
Floor 8	1594.974	1489.829	1384.684	1279.540
Floor 9	1845.064	1723.422	1601.779	1480.137
Floor 10	2101.058	1962.530	1824.001	1685.472
Floor 11	2362.421	2206.652	2050.883	1895.114
Floor 12	2628.709	2455.376	2282.042	2108.708
Floor 13	2899.550	2708.352	2517.154	2325.955
Floor 14	3174.625	2965.283	2755.941	2546.600
Floor 15	3453.659	3225.913	2998.167	2770.421
Floor 16	3736.413	3490.017	3243.621	2997.225
Floor 17	4022.674	3757.398	3492.121	3226.844
Floor 18	4312.255	4027.879	3743.503	3459.126
Floor 19	4604.988	4301.305	3997.621	3693.937
Floor 20	4900.722	4577.533	4254.344	3931.155
Floor 21	5199.320	4856.437	4513.554	4170.671
Floor 22	5500.657	5137.899	4775.142	4412.384
Floor 23	5804.620	5421.815	5039.009	4656.204
Floor 24	6111.105	5708.086	5305.066	4902.047
Floor 25	6420.017	5996.623	5573.229	5149.836
Floor 26	6731.265	6287.343	5843.422	5399.500
Floor 27	7044.769	6580.170	6115.572	5650.973
Floor 28	7360.451	6875.032	6389.613	5904.194
Floor 29	7678.242	7171.863	6665.485	6159.106
Floor 30	7998.073	7470.601	6943.128	6415.656
Floor 31	8319.885	7771.187	7222.490	6673.793
Floor 32	8643.616	8073.568	7503.520	6933.472
Floor 33	4565.870	4264.749	3963.629	3662.508
ΣOverturning moment (Design value)	**133902**	**125072**	**116242**	**107412**

7.3.7 Summary of hand calculations

In Table 7.16, the design values of stabilising moment and overturning moment from hand calculations are listed together for each wall.

Table 7.16: Design values of stabilising M_r and overturning moment M_e [kNm].

Design values	Wall 1	Wall 2	Wall 3	Wall 4
Stabilising moment [Multiplied with 0.9]	48553	47597	47640	51468
Overturning moment	133902	125072	116242	107412

These values can be compared to the Dynamo script result seen in Figure 7.9

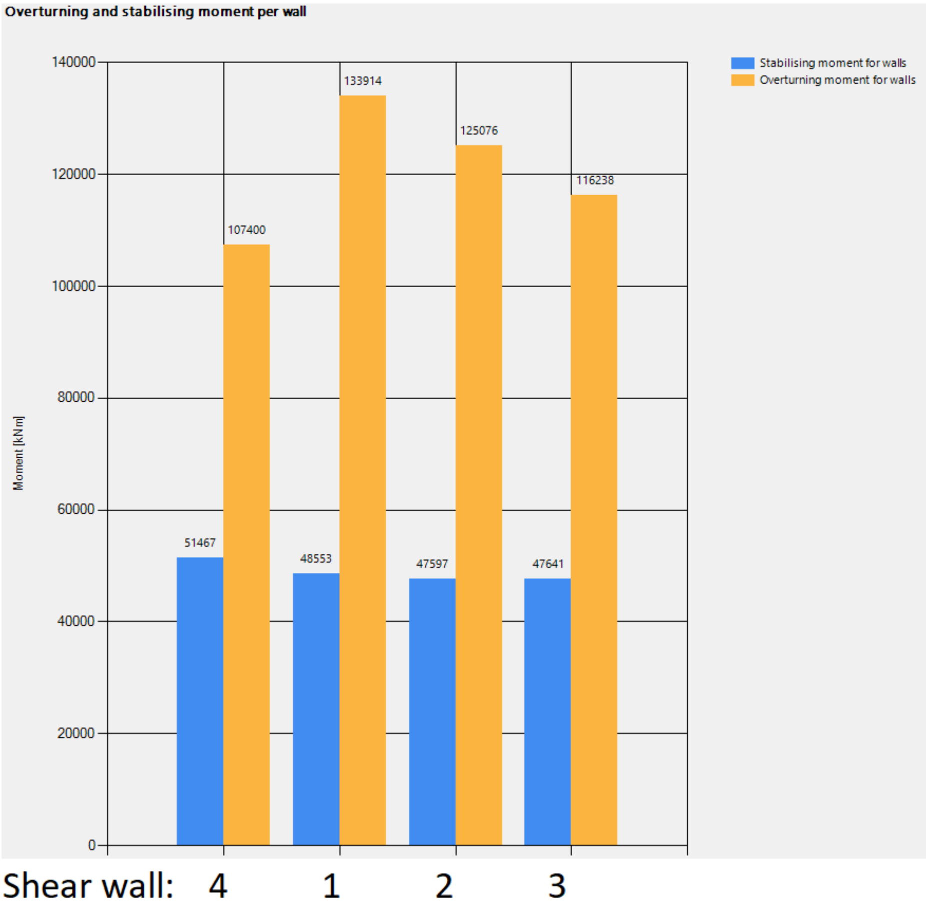


Figure 7.9: Overturning moment and stabilising moment for shear wall 1, 2, 3, 4.

The overturning moment from hand calculations and Dynamo deviates a few kN. The difference between the overturning moment in Dynamo and hand calculations are due to difference in meassurements of coordinates for hand calculations and Dynamo script

7.4 Dynamo output excel

Table 7.17: Wind load calculations for Veddesta, where all 6 load cases per wall are presented for one wall at the time. In the rightmost excel column the load for the highest resulting overturning moment for said wall is displayed (based on the numbers presented in this sheet).

Wall_ID	Floor height	Triangle windload per floor [kN]	%_tri_lc1_	%_tri_lc1_	%_tri_lc2_	%_tri_lc2_dir_2	Square windload per floor [kN]	%_squ_di	%_squ_dir_1	Wall characteristic load [kN]
1170364	3	70,7582337	15,25	34,75	29,3	20,7	93,77074949	22,28	27,72	25,9971
	6	79,9355869	15,22	34,78	29,32	20,68	105,9328293	22,27	27,73	29,37965
	9	90,41524763	15,2	34,8	29,32	20,68	119,8207628	22,26	27,74	33,23786
	12	98,03482767	15,19	34,81	29,33	20,67	129,918439	22,26	27,74	36,04254
	15	104,0834763	15,18	34,82	29,33	20,67	137,9342738	22,26	27,74	38,26845
	18	109,1245925	15,18	34,82	29,33	20,67	144,6148991	22,26	27,74	40,12325
	21	113,4597048	15,18	34,82	29,33	20,67	150,3599086	22,25	27,75	41,71808
	24	117,2704805	15,17	34,83	29,33	20,67	155,410053	22,25	27,75	43,11987
	27	120,6753816	15,17	34,83	29,33	20,67	159,9223212	22,25	27,75	44,37228
	30	123,756188	15,17	34,83	29,34	20,66	164,0050903	22,25	27,75	45,50542
	33	126,5718519	15,17	34,83	29,34	20,66	167,7364852	22,25	27,75	46,541
	36	129,1663394	15,17	34,83	29,34	20,66	171,1747711	22,25	27,75	47,4952
	39	131,5733545	15,17	34,83	29,34	20,66	174,3646134	22,25	27,75	48,38043
	42	133,8193271	15,17	34,83	29,34	20,66	177,3410379	22,25	27,75	49,20642
	45	135,9253828	15,17	34,83	29,34	20,66	180,1320406	22,25	27,75	49,98094
	48	137,9086841	15,17	34,83	29,34	20,66	182,7603657	22,25	27,75	50,71031
	51	139,7833709	15,17	34,83	29,34	20,66	185,244752	22,25	27,75	51,39972
	54	141,5612367	15,17	34,83	29,34	20,66	187,6008285	22,25	27,75	52,05352
	57	143,2522245	15,17	34,83	29,34	20,66	189,8417718	22,25	27,75	52,67537
	60	144,8647976	15,17	34,83	29,34	20,66	191,9787979	22,25	27,75	53,26838
	63	146,4062216	15,17	34,83	29,34	20,66	194,0215352	22,25	27,75	53,83522
	66	147,8827821	15,17	34,83	29,34	20,66	195,9783136	22,25	27,75	54,3782
	69	149,2999543	15,17	34,83	29,34	20,66	197,856389	22,25	27,75	54,89934
	72	150,6625382	15,17	34,83	29,34	20,66	199,6621225	22,25	27,75	55,40041
	75	151,9747656	15,17	34,83	29,34	20,66	201,4011223	22,25	27,75	55,88295
	78	153,2403875	15,17	34,83	29,34	20,66	203,078359	22,25	27,75	56,34836
	81	154,4627443	15,17	34,83	29,34	20,66	204,6982598	22,25	27,75	56,79786
	84	155,6448246	15,17	34,83	29,34	20,66	206,2647849	22,25	27,75	57,23254

Table 7.18: Overturning moment calculations for Veddesta, where all the forces and respective lever arms are presented so the user can view the overturning and stabilising moment calculations. The data is presented one wall at the time with load into every floor present. The result in overturning moment, stabilising moment and % capacity reached are presented and the bottom of the table.

Wall Id	Floor Height [m]	Design Wind load in wall per floor [kN]	Imperfection load in wall per floor [kN]	Vertical load per floor [kN]	Lever-arm (L/2) [m]	Selfweight wall [kN]	Lever-arm (L/2) [m]
1170364	3	38,99564851	1,431776488	301,6226756	3,4	5301,45	3,4
	6	44,06947729	1,43112575	301,6226756	3,4		
	9	49,85679647	1,430776979	301,6226756	3,4		
	12	54,06381686	1,430597892	301,6226756	3,4		
	15	57,40267998	1,430499076	301,6226756	3,4		
	18	60,18487616	1,430440128	301,6226756	3,4		
	21	62,57711465	1,43040256	301,6226756	3,4		
	24	64,67981091	1,430377299	301,6226756	3,4		
	27	66,5584251	1,430359562	301,6226756	3,4		
	30	68,25813569	1,430346659	301,6226756	3,4		
	33	69,81150279	1,430336993	301,6226756	3,4		
	36	71,24280515	1,430329573	301,6226756	3,4		
	39	72,57065167	1,430323757	301,6226756	3,4		
	42	73,80963333	1,430319115	301,6226756	3,4		
	45	74,9714114	1,430315354	301,6226756	3,4		
	48	76,06545876	1,430312264	301,6226756	3,4		
	51	77,09957945	1,430309696	301,6226756	3,4		
	54	78,08028204	1,430307538	301,6226756	3,4		
	57	79,01305352	1,430305708	301,6226756	3,4		
	60	79,90256412	1,430304142	301,6226756	3,4		
	63	80,75282301	1,430302793	301,6226756	3,4		
	66	81,5672984	1,430301622	301,6226756	3,4		
	69	82,34901141	1,430300599	301,6226756	3,4		
	72	83,10061036	1,4302997	301,6226756	3,4		
	75	83,82443012	1,430298907	301,6226756	3,4		
	78	84,52254013	1,430298202	301,6226756	3,4		
	81	85,19678341	1,430297574	301,6226756	3,4		
	84	85,84880872	1,430297012	301,6226756	3,4		
	87	86,48009713	1,430296506	301,6226756	3,4		
	90	87,0919842	1,43029605	301,6226756	3,4		
	93	87,68567854	1,430295637	301,6226756	3,4		
	96	88,2622776	1,430295262	301,6226756	3,4		
	99	44,27322105	148,4287186	301,6226756	3,4		

Overturning moment [kNm]	Stabalising moment [kNm]	Percent of capacity [%]
147865,5937	51866,99421	285,0861053

Table 7.19: Characteristic wind load calculations from EKS11 at each height presented in an Excel sheet with data from dynamo for the both directions.

Calculatti	Charateristic wind-load per square meter in y dir. at height z [N/m^2]	Charateristic wind-load per square meter in x dir. at height z [N/m^2]
1,5	643,4781535	553,6176544
4,5	1000,614097	860,8802432
7,5	1168,412559	1005,24597
10,5	1284,976348	1105,531848
13,5	1375,167434	1183,127919
16,5	1449,104499	1246,739815
19,5	1511,956515	1300,814666
22,5	1566,736388	1347,944634
25,5	1615,360698	1389,778652
28,5	1659,127292	1427,433325
31,5	1698,9574	1461,701233
34,5	1735,529433	1493,166052
37,5	1769,357991	1522,270516
40,5	1800,843019	1549,358719
43,5	1830,301743	1574,703589
46,5	1857,990169	1598,525379
49,5	1884,117989	1621,004499
52,5	1908,859197	1642,290644
55,5	1932,359811	1662,509443
58,5	1954,743612	1681,767388
61,5	1976,116468	1700,155566
64,5	1996,569661	1717,752509
67,5	2016,182465	1734,626422
70,5	2035,024178	1750,836926
73,5	2053,155744	1766,436453

Table 7.20: List of inputs used for calculations in the Dynamo script presented in an Excel table (in Swedish).

Referensvindhastighet [m/s]	24
Terrängtyp	2
Våningshöjd [m]	3
Antal våningar	33
Byggnadshöjd [m]	99
Byggnads bredd (y-riktning [m])	32,56
Byggnads bredd (x-riktning [m])	13,04
Vindlast säkerhets koefficient	1,5
Imperfektionslast säkerhets koefficient	1,1
Egentynd säkerhets koefficient	0,9
H/d mot (y-riktning)	7,59
H/d mot (x-riktning)	3,04
C_pe zon D (y riktning)	-0,8
C_pe zon E (y riktning)	-0,8296
C_pe zon D (y riktning)	-0,8
C_pe zon E (y riktning)	-0,60203
Dynamiskvindfaktor K_p	3

8 Appendix Matlab code

```
%-------Matlab code for comparison between methods regarding stiffness
 in walls with openings%

clc
format compact
clear all
close all

% Inputs for building geometrý, material parameters, mesh settings.
h1 = 0.5;
h2 = 2;
h3 = 0.5;
L1 = 1;
L2 = 2;
L3 = 1;
level = 6;

E = 33*10E9; %Pa
G = 12.5E9; %Pa
v = 0.2; %-
t = 1; %m
unit_load=1;

seed_nr = 2; %Number of nodes

door = 0; %1 means door type opening, 0 is window type opening
hole = 1; %1 means there is a hole (always 1)


% Building geometry, solveing equation
floor_height = (h1+h2+h3);
height = floor_height*level;
width = (L1+L2+L3);

if door == 1;
    relative_hole_area = L2*(h2+h3)/(floor_height*width);
else
    relative_hole_area = L2*h2/(floor_height*width);
end

nen=4;
dofsPerNode=2;

x1=0;
x2=x1+L1;
x3=x2+L2;
x4=x3+L3;

y1=0;
y2=y1+h3;
y3=y2+h2;
y4=y3+h1;
```

```
Vertices = [%Nodes
    x1 y1; %1
    x2 y1;
    x3 y1;
    x4 y1;
    x1 y2; %5
    x2 y2;
    x3 y2;
    x4 y2;
    x1 y3; %9
    x2 y3;
    x3 y3;
    x4 y3;
    x1 y4; %1
    x2 y4;
    x3 y4;
    x4 y4;
    ];

Segments = [ %Segments between nodes
    1 2;
    2 3;
    3 4;
    5 6; %4
    6 7;
    7 8;
    9 10;
    10 11; %8
    11 12;
    13 14;
    14 15;
    15 16; %12
    1 5;
    2 6;
    3 7;
    4 8; %16
    5 9;
    6 10;
    7 11;
    8 12; %20
    9 13;
    10 14;
    11 15;
    12 16 %24
    ];

Surfaces = [ %Surface from segments
    1 14 4 13;%1
    2 15 5 14;
    3 16 6 15;
    4 18 7 17;%4
    6 20 9 19;
    7 22 10 21; %6
```

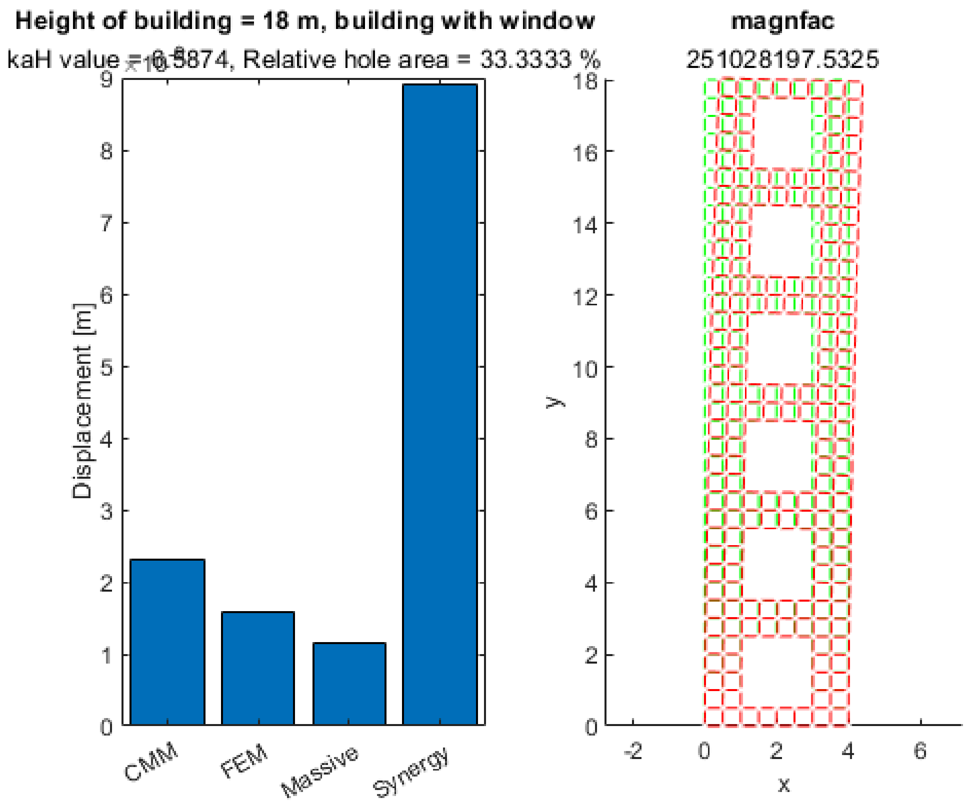
```
        8 23 11 22;
        9 24 12 23; %8
        ];
    if door == 1;
        Surfaces(2,:)=[];
    end

    if hole == 0;
        Surfaces(end+1,:)=[5,19,8,18];
    end

    top_segm = [10 11 12];
    window_segm = [5 19 8 18];
    if level >= 2 %Adding another floor ontop of the first floor, loop for
     every floor
        for loop = 2:level
            k_node = 16+(loop-2)*12;
            k_vert = 24+(loop-2)*21;

            y1 = y4;
            y2 = y1 + h3;
            y3 = y2 + h2;
            y4 = y3 + h1;

            Vertices_loop = [
                x1 y2; %1
                x2 y2;
                x3 y2;
                x4 y2;
                x1 y3; %5
                x2 y3;
                x3 y3;
                x4 y3;
                x1 y4; %9
                x2 y4;
                x3 y4;
                x4 y4; %12
                ];

            Segments_loop = [
                1 2;
                2 3;
                3 4;
                5 6; %4
                6 7;
                7 8;
                9 10;
                10 11; %8
                11 12;
                -3 1; %10
                -2 2;
                -1 3;
                0 4;
                1 5; %14
```

```
            2 6;
            3 7;
            4 8; %17
            5 9;
            6 10;
            7 11;
            8 12; %20
            ]+k_node;

        Surfaces_loop = [
            -14 11 1 10;
            -13 12 2 11;
            -12 13 3 12;
            1 15 4 14
            3 17 6 16;
            4 19 7 18;
            5 20 8 19;
            6 21 9 20;
            ]+k_vert;

        if door == 1;
            Surfaces_loop(2,:)=[];
        end

        if hole == 0;
            Surfaces_loop(2,:)=[2,16,5,15];
        end

        Vertices = [Vertices; Vertices_loop];
        Segments= [Segments; Segments_loop];
        Surfaces = [Surfaces; Surfaces_loop];

        top_segm = [top_segm, [7 8 9]+k_vert];
        window_segm = [window_segm, [2, 15, 16, 5]+k_vert];
    end
end

Seed = [];
seed_per_floor=[h1,h2,h3]./(h1+h2+h3)*8;
seed_per_floor=ceil(seed_per_floor);

for i = 1:level
    Seed = [Seed,seed_per_floor];
end
Segp=[];
Seed=ceil(([L1 L2 L3 L1 L2 L3 L1 L2 L3 L1 L2 L3 h1 h1 h1 h1 h2 h2 h2
 h2 h3 h3 h3 h3])*seed_nr); %Even size of element mesh
Seed_2 = ceil(([L1 L2 L3 L1 L2 L3 L1 L2 L3 h1 h1 h1 h1 h2 h2 h2 h2 h3
 h3 h3 h3])*seed_nr);
if level > 1
    for i = 2:level
        Seed=[Seed, Seed_2];
    end
end
```

```
Segp=Seed;

mp=[dofsPerNode, nen];
% Generate element mesh
[Coord Edof Dof
 meshdb]=strMeshgen(Vertices,Segments,Surfaces,Segp,mp);

% Generate element coordiantes
[Ex Ey]=coordxtr(Edof,Coord,Dof,nen);
ptype=1; %ptype = 1=>plane stress; 2=>plane strain
ep=[ptype,t]; %plane stress, 1m thick
D = hooke(ptype,E,v);

ne1=size(Edof,1);
ndof=max(Edof(:));

K=zeros(ndof,ndof);
f=zeros(ndof,1);

%Skapar element stiffness och assemblerar till global stiff
for i=1:ne1
    Ke=planqe(Ex(i,:),Ey(i,:),ep,D);
    index = Edof(i,2:end);
    K(index,index)=K(index,index) + Ke;
end

find_y=find(Coord(:,2)>=y4);

find_index = find_y;

load_node=find_index*2-1;
f(load_node)=unit_load/(length(load_node)); %load is distributed over
 the nodes in the roof to avoid local deformation

%degrees of fredom along segmenten for boundary condition
bc1=extrSeg([1,2,3]',meshdb,[1 2]);
bc2=extrSeg(top_segm',meshdb,[2]);

%Define BC
bc=[
    bc1, bc1*0 %Ground BC
    ];

%Solve equation
[a,r]=solveq(K,f,bc);

disp_load_node=a(load_node);

% CMM

l = L1/2 + L2 + L3/2;

I1=t*L1^3/12;
I2=t*L3^3/12;
```

```
A1 = L1 * t;
A2 = L3 * t;

d = (h1 + h3);

I=I1+I2;

H=height;

b = L2;
be = b+1/2*d; %page 36

A = A1 + A2;
Ab = d * t;
Ib1 = t*d^3/12;
Ib = Ib1 / (1 + 2.4*(d/L2)^2*(1+v));

lambda = 1.2; % for square cross-sections
r=12*E*Ib1/(b^2*G*Ab) * lambda;
Ie = Ib1/(1+r); %2.4;

n_b = 2;

k_sq = (1+A*I/(A1*A2*l^2));
k = sqrt(k_sq);

h=h1+h2+h3;

alpha_2_sq = 12*Ie*l^2/((be)^3*h*I);

alpha_2 = sqrt(alpha_2_sq);

F_3 = 1 - 1/k^2 + 3/(H^2*k^4*alpha_2^2*n_b)-
(3*sinh(sqrt(n_b)*k*alpha_2*H))/
(H^3*alpha_2^3*k^5*n_b^(3/2)*cosh(sqrt(n_b)*k*alpha_2*H));
x_H = 1 * H^3/(3*E*I)*F_3;

kaH=k*alpha_2*H;

if kaH <= 1
    disp("kaH is " + kaH + " Model as wall piers with synergy")
end

% Comparison
h = h1+h2+h3;
L = L1+L2+L3;
n_floor = level;
height = (h*level);

% Matlab Model
disp_wall_matlab = mean(disp_load_node(1));

% Massive Wall
```

```
disp_wall_massive=4*(height)^3/(L)^3+3*height/(L);
disp_wall_massive=disp_wall_massive/(E*t);

% Synergy Walls
disp_wall_synergy=4*(height)^3/(L1+L3)^3+3*height/(L1+L3);
disp_wall_synergy=disp_wall_synergy/(E*t);

% Continuous medium method
disp_CMM = x_H;

%
ed1=extract(Edof,a);
magnfac=(x1+x2+x3)/max(a)*0.1; %Change last scalar for different
 scaling of dispalcement in figure

ed1=extract(Edof,a);
figure(6)
hold on
subplot(1,2,1);
X=categorical({'FEM', 'Massive', 'Synergy' , 'CMM'});
bar(X,[disp_wall_matlab(1), disp_wall_massive,
 disp_wall_synergy,disp_CMM]);
if door == 1;
    title("Height of building = " + height + " m, building with
 door");
else
    title("Height of building = " + height + " m, building with
 window");
end

subtitle("kaH value = " + kaH + ", Relative hole area = " +
 relative_hole_area*100 + " %")
ylabel('Displacement [m]')
hold off
subplot(1,2,2);
title('magnfac', magnfac);
eldraw2(Ex,Ey,[1, 2, 0]);
eldisp2(Ex,Ey,ed1,[1 4 0],magnfac);

hold off
```



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