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Cost Evaluation of Seismic Load Resistant Structures Based on the Ductility Classes in Eurocode 8

Georgios Valdemar Drivas

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KTH Royal Institute of Technology
Department of Civil and Architectural Engineering
Division of Concrete Structures
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Preface

This master thesis was carried out at the *Department of Building Structures* at *Rambøll Norge AS* in collaboration with the *Department of Civil and Architectural Engineering*, the division of *Concrete Structures*, at the KTH *Royal Institute of Technology* in Stockholm. Dr. Farzin Shahrokhi supervised the project kindly offering his valuable guidance and advice; therefore, I wish to express my sincere gratitude to him, along with thanking the staff at Rambøll Norge AS for their assistance during the degree project. Additionally, I express my appreciation to Professor Anders Ansell, examiner of the project, for the support and input throughout the report writing process.

At last I want to thank my family and friends for their support during the five years of my engineering studies at KTH *Royal Institute of Technology*, as well as abroad at the ETH *Swiss Federal Institute of Technology Zürich* and the ΕΜΠ *National Technical University of Athens*.

Oslo, September 2014

Georgios Valdemar Drivas

Abstract

Most people do not associate Scandinavia with seismic activity and earthquakes; however, there is in fact seismic activity in the region. Although in comparison with southern Europe the return periods of earthquakes with large magnitudes are quite long, it is critical to consider earthquake impact when designing structures. Earthquake impact is difficult to predict, but building standards provide guidance to safely design structures based on statistical and empirical data specific to regional conditions and circumstances. Crucial for the final impact and response of a structure is not only the ground acceleration, but also the ground type, which can amplify seismic vibrations and ultimately cause unfortunate damage to the structural elements.

Since 2010 *Eurocode 8*, the European standards for seismic design has been in effect for building structures in Norway. The main difference with the application of the standards in Norway compared to Southern Europe is the choice between *elastic* and *ductile design* in some cases. Presumably, the same design regulations are applicable for design of structures in Sweden, because parts of Sweden share similar conditions as in Norway. This master thesis examines the results of selecting between elastic and ductile design based on an arbitrary finite element model, and ultimately, presents the differences in cost efficiency in both quantitative and qualitative measures.

In the arbitrary structure that is analyzed, the lateral bearing system contains a concrete wall shaft. In order to evaluate profitability, the cost development of reinforcement in the walls, is analyzed based on ground acceleration and ductility class. The study ultimately implies a breaking point when structures in ductility class medium are more cost efficient than structures in ductility class low and vice versa, with the condition that the governing lateral force is the seismic vibration and that the normalized axial force is less than 15%.

Keywords: Seismic Design, Eurocode 8, Norwegian Annex, Ductility class low, Ductility class medium, Economical Assessment, Precast Structures

Sammanfattning

Skandinavien förknippas inte i första hand med seismisk aktivitet och jordbävningar. I regionen förekommer seismisk aktivitet, dock är returperioderna för jordbävningar med stor magnitud förhållandevis lång i relation till södra Europa. Jordbävningslaster är svåra att förutse, men byggnormerna vägleder till säkert utformande och dimensionering mot dess påverkan, baserat på statistiska och empiriska data för regionala förutsättningar och omständigheter. En avgörande faktor för konstruktioners inverkan och respons är inte endast markaccelerationen utan även marktypen som kan förstärka de seismiska vibrationerna och eventuellt orsaka skada på byggnader.

I Norge används sedan 2010 de europeiska normerna för jordbävningsdimensionering, *Eurokod 8*. Den väsentliga skillnaden jämfört med utförandet av konstruktioner i södra Europa är att valet mellan *elastiska* och *duktila* utformanden ges i vissa fall. Hypotetiskt kan samma normer användas för dimensionering av byggnader i Sverige, eftersom vissa regioner i Sverige har samma förutsättningar som i Norge.

I detta examensarbete undersöks valet mellan elastisk och duktil dimensionering med hjälp av finita element modellering av en godtycklig konstruktion samt en jämförelse av de två fallen som slutligen leder till en analys av kostnadseffektiviteten, både kvantitativt och kvalitativt.

Det horisontella bärsystemet i den använda modellen är ett schakt bestående av betongväggar. För att kunna uppskatta lönsamheten analyseras kostnadsutvecklingen av armeringsinnehållet, beroende av markacceleration och duktilitetsklass. Studien har resulterat i definitionen av en brytpunkt som anger när dimensionering enligt duktilitetsklass medium är effektivare än dimensionering enligt duktilitetsklass låg och vice versa, under förutsättning att jordbävningslasten är dimensionerande och den normaliserade axialkraften är lägre än 15%.

Nyckelord: Jordbävningsdimensionering, Eurokod 8, Norskt annex, Duktilitetsklass låg, Duktilitetsklass medium, Lönsamhetsbedömning, Prefabricerade konstruktioner.

Nomenclature

Acronyms

(+)	Notation of positive sign (direction) seismic load in the load combination for analysis
(-)	Notation of negative sign (direction) seismic load in the load combination for analysis
CQC	Complete quadratic combination
CW3	Core wall 3
CW4	Core wall 5
CW5	Core wall 4
CW6	Core wall 6
DCH	Ductility class high
DCL	Ductility class low
DCM	Ductility class medium
DNB	Dimensionering av Nukleära Byggnadskonstruktioner
EC0	Eurocode 0
EC2	Eurocode 2
EC8	Eurocode 8
NOK	Norwegian kroner
P-wave	Primary wave
RSA2014	Robot Structural Analysis Professional 2014
S-wave	Secondary wave

SDOF	Single-degree-of-freedom
SEK	Swedish kronor
ULS	Ultimate limit state

Greek letters

θ	Factor related to the viscous damping
α	Confinement effectiveness factor
α_0	Prevailing aspect ratio of the walls of the structural system
α_1	Multiplier of horizontal design seismic action at formation of first plastic hinge in the system
α_b	Ratio of balanced reinforced compression zone
α_u	Multiplier of horizontal seismic design action at formation of global plastic mechanism
β	Lower bound factor
β_f	Frequency ratio
ΔE	Absorbed energy
δ	Distance to seismological station
η	Damping correction factor
γ_I	Importance factor
γ_c	Partial factor for concrete
γ_{Rd}	Overstrength factor
γ_s	Partial factor for steel
λ	Slenderness
λ_d	Factor for balanced reinforced cross-section
λ_{lame}	First Lamé parameter
μ	Ductility factor
μ_ϕ	Curvature ductility factor
ν_d	Axial force due in the seismic design situation, normalized to $A_c f_{cd}$

Ω	Frequency of input force excitation
ω_D	Damped natural frequency
ω_n	Natural frequency
ω_v	Mechanical ratio of vertical web reinforcement
ω_{wd}	Mechanical volumetric ratio of confining hoops within the critical regions
ϕ	Combined with $\psi_{2,i}$ to determine the effects of the design seismic actions
$\phi(t)$	Phase-angle
ϕ_h	Reinforcement diameter (horizontal)
ϕ_v	Reinforcement diameter (vertical)
ϕ_w	Reinforcement diameter (hoop)
$\psi_{2,i}$	Combination coefficient for the quasi-permanent value of a variable action i
$\psi_{E,i}$	Combination coefficient for a variable action i, to be used when determining the effects of the design seismic action
$\rho(t)$	Amplitude of vibration
ρ_{dens}	Soil density
$\rho_{h,min}$	Minimum ratio of horizontal reinforcement
ρ_h	Ratio of horizontal reinforcement
$\rho_{v,min}$	Minimum ratio of vertical reinforcement
ρ_v	Ratio of vertical reinforcement
σ_c	Concrete capacity
σ_{cp}	Limitation of compression strain
τ	Time step
ε	Compressive strain
ε_0	Concrete strain limit

$\varepsilon_{cu2,c}$	Ultimate compressive strain
ε_{cu2}	Spalling compressive strain
ε_{cu}	Ultimate compressive strain in the concrete
ε_{c}	Compressive strain in the concrete
$\varepsilon_{sy,d}$	Strain in reinforcement
ε_{s}	Strain in tensional reinforcement
ξ	Viscous damping ratio

Latin letters

$\ddot{u}_g(t)$	Ground acceleration
$\ddot{u}(t)$	Dynamic acceleration
$\ddot{u}_k(t)$	Acceleration of k^{th} mode
$\dot{u}(t)$	Dynamic velocity
$\dot{u}_0$	Initial velocity
$\dot{u}_k(t)$	Velocity of k^{th} mode
ω_k	Natural frequency of k^{th} mode
$\rho_{c,min}$	Minimum ratio of longitudinal reinforcement
$a(t)$	Acceleration
A_0	Factor depending on epicentral distance
A_c	Area of critical zone/concrete (cross-section)
a_g	Design ground acceleration on type A ground
A_i	Cross-section area of wall
A_W	Maximum excursion of the *Wood-Anderson seismograph*
$A_{\phi,h,tot}$	Total area of horizontal reinforcement bars
$A_{\phi,h}$	Area of horizontal reinforcement bar
$A_{\phi,v,boundary,tot}$	Total area of vertical reinforcement bars in boundary
$A_{\phi,v,web,tot}$	Total area of vertical reinforcement bars in web

$A_{\phi,v}$	Area of vertical reinforcement bar
$A_{\phi,w}$	Area of hoop reinforcement bar
a_{g40Hz}	Peak acceleration of the bedrock for the return period of 475 years
a_{gR}	Reference peak ground acceleration on type A ground
$A_{h,min}$	Minimum total area of horizontal reinforcement
$A_{s,b}$	Area of balanced reinforced cross-section
$A_{s,h}$	Required total horizontal reinforcement area
$A_{s,v}$	Required total vertical reinforcement area
A_s	Required total reinforcement area
$A_{v,max}$	Maximum total area of vertical reinforcement
$A_{v,min}$	Minimum total area of vertical reinforcement
A_v	Total area of vertical reinforcement
b_c	Cross-sectional dimension of wall/column
b_i	Distance between consecutive engaged bars (cross-ties)
b_o	Width of confined core in a column or in then boundary element of a wall (to centerline of hoops)
b_w	Thickness of confined parts of a wall section
b_o	Width of confined core in a column or in the boundary element of a wall (to centerline of hoops)
b_{wo}	Thickness of web of a wall
c	Damping coefficient
c_1	Location of the tension resultant
c_2	Location of the compression resultant
c_u	Undrained soil shear strength
c_{conf}	Concrete confinement
c_{cr}	Critical damping coefficient
D	Maximum displacement

d	Depth to center of reinforcement
D_e	Elastic remaining displacement
D_L	Limit value for displacement
d_l	Length from outermost fiber in the compression zone to the center of the reinforcement
D_m	Displacement corresponding to force S_m
D_p	Plastic remaining displacement
D_u	Ultimate displacement
D_y	Yield displacement
d_{bL}	Longitudinal bar diameter
d_{bw}	Diameter of hoop
E	Elastic energy
E_d	Design value of action effects
e_{0x}	Structural eccentricity
E_{load}	Seismic load
F_c	Total compressive force
$f_D(t)$	Damping force
$f_I(t)$	Inertial force
$f_S(t)$	Structural force
f_{cd}	Design compressive strength of concrete
f_{ck}	Characteristic compressive strength of concrete
f_{ctd}	Design tensile strength of concrete
$f_{ctk,0.05}$	Characteristic tensile strength of concrete
F_s	Seismic force
f_{yd}	Design yield strength of steel
f_{yk}	Characteristic steel strength

g	Gravitational acceleration
$G_{k,j}$	Characteristic value of permanent action j
G_{load}	Gravity load
G_{shear}	Soil shear modulus
h	Height
h_s	Clear story height
h_w	Height of wall
h_{cr}	Height of the critical region
k	Stiffness
k_p	Factor reflecting the prevailing failure mode in precast structural systems with walls
k_w	Factor reflecting the prevailing failure mode in structural systems with walls
l	Length
l_c	Length of critical zone
l_s	Radius of gyration of the floor mass in plan
l_w	Length of cross-section of wall
L_{max}	Larger in plan dimension of the building measured in orthogonal directions
L_{min}	Smaller in plan dimension of the building measured in orthogonal directions
M	Earthquake magnitude
m	Mass
$M_b(t)$	Moment at base
M_L	Magnitude on the Richter-scale
M_{DCL}	Moment for design (DCL)
M_{DCM}	Moment for design (DCM)

M_{Ed}	Design bending moment from the analysis for the seismic design situation
M_{Rd}	Design flexural resistance
M_{RSA}	Moment from analysis in RSA2014
n	Amount of reinforcement bars
N_c	Compression resultant
N_{DCL}	Vertical force for design (DCL)
N_{DCM}	Vertical force for design (DCM)
N_{Ed}	Design axial force from the analysis for the seismic design situation
N_{RSA}	Vertical force from analysis in RSA2014
N_{SPT}	Standard Penetration Test blow-count
p	Static force
$p(t)$	Dynamic force
P_e	Elastic force
P_s	Elastic force
P_y	Yield force
$p_{eff}(t)$	Effective earthquake force
$PSA(T_k,\xi)$	Spectral pseudo-acceleration of the k^{th} mode
$PSV(T_k,\xi)$	Spectral pseudo-velocity of the k^{th} mode
q	Behavior factor
q_o	Basic value of the behavior factor
q_p	Behavior factor for precast structures
$Q_{k,i}$	Characteristic value of the accompanying variable action i
Q_{load}	Live load
$R_d(t)$	Deformation response factor
r_x	Torsional radius

S	Soil factor
$S_a(T_k,\xi)$	Spectral pseudo-acceleration of the k^{th} mode
$S_d(T_k,\xi)$	Spectral displacement of the k^{th} mode
$S_d(T_n)$	Design spectrum (for elastic analysis). At $T_n = 0$, the spectral acceleration given by this spectrum equals the design ground acceleration on type A ground multiplied by the soil factor S
$S_e(T_n)$	Elastic horizontal ground acceleration response spectrum also called elastic response spectrum. At $T_n = 0$, the spectral acceleration given by this spectrum equals the design ground acceleration on type A ground multiplied by the soil factor S.
S_L	Force corresponding to displacement D_L
S_m	Maximum force
S_u	Force corresponding to displacement D_u
$S_v(T_k,\xi)$	Spectral pseudo-velocity of the k^{th} mode
s_w	Spacing of confinement hoops
$S_{a,e}$	Acceleration response for elastic system
$S_{a,p}$	Acceleration response for elastoplastic system
$S_{d,e}$	Displacement response for elastic system
$S_{d,p}$	Displacement response for elastoplastic system
s_h	Minimum spacing between horizontal reinforcement bars
S_{load}	Snow load
$s_{v,web}$	Spacing between vertical reinforcement bars in web
s_v	Spacing between vertical reinforcement bars
s_w	Spacing of reinforcement hoops
$SD(T_k,\xi)$	Spectral displacement of the k^{th} mode
T_1	Fundamental natural period
T_B	Corner period at the lower limit of the constant acceleration region of the elastic spectrum

T_C	Corner period at the upper limit of the constant acceleration region of the elastic spectrum
T_D	Period at the lower limit of the constant displacement region of the elastic spectrum
T_k	Natural period of the k^{th} mode
T_n	Natural period
T_p	Time of arrival of first P-wave
T_s	Time of arrival of first S-wave
$T_{assumed}$	Initial assumption of internal tension in wall section
$T_{computed}$	Computed internal tension in wall section
T_{kin}	Kinetic energy
U	Deformation energy
$u(t)$	Relative dynamic displacement
$u^t(t)$	Total dynamic displacement
u_0	Initial displacement
u_e	Elastic displacement
$u_g(t)$	Ground motion
$u_k(t)$	Relative displacement of k^{th} mode
u_y	Yield displacement
u_{max}	Maximum displacement
u_{st}	Static displacement
$V_b(t)$	Shear force at base
V_s	Requirement of shear force resistance in wall
$V_{c,vol}$	Volume of confined concrete
V_{DCL}	Lateral force for design (DCL)
V_{DCM}	Lateral force for design (DCM)
V_{p-wave}	Velocity of primary wave

$V_{Rd,c,N}$	Resistance contribution of axial force
$V_{Rd,c,V}$	Shear resistance of the wall
$V_{Rd,c}$	Lateral resistance without horizontal reinforcement
$V_{Rd,i}$	Lateral force resistance of of an non-reinforced connections
V_{RSA}	Lateral force from analysis in RSA2014
$v_{s,30}$	Average value of propagation velocity of S-waves in the upper 30 m
$V_{s,vol}$	Accumulated volume of the hoop reinforcement per 1 m
V_{s-wave}	Velocity of secondary wave
$V_{wall,base}$	Shear force at base of wall
$V_{wall,top}$	Shear force at top of wall
WFX	Reduced horizontal force (X-direction)
WFY	Reduced horizontal force (Y-direction)
WFZ	Reduced vertical force (Z-direction)
WMX	Reduced moment (around X-axis)
WMY	Reduced moment (around Y-axis)
x	Length of the compression zone
z	Internal lever arm

Contents

1
Introduction

1.1 Background

Since 2010 buildings in Norway must meet the design regulations specified in Eurocode 8 (EC8), in addition to those in the Norwegian national annex (NA:2014), which prescribes regulations specific to the region. The seismic forces are dynamic and in order to calculate the impact of earthquakes on building structures ground accelerations for the seismic zones in Norway are given in the norms. Supplemental factors for the soil's acceleration amplification, including importance factors, also affect the final design value for the acceleration used in an analysis model. Hypothetically, the same rules could be applied for structures in Sweden, because some regions in Sweden have conditions similar to those in regions in Norway. It is important to note that seismic design is taken into account when designing hazardous facilities in Sweden. Hazardous facilities are mostly industrial sites that handle large quantities of dangerous goods which could harm the environment and society significantly as a result of structural collapse. An example of such structures in Sweden are nuclear power plants.

Ductility class for design is chosen depending on the dimensioning value of ground acceleration at the location of the specific building. This means that the seismic forces and acceleration can be reduced if higher ductility class is selected for the structure, which also means a more inelastic and energy dissipative behavior.

The building standards allow for the design to be conducted according to ductility classes low (DCL) or medium (DCM) when the design ground acceleration exceeds 0.10 g (EN-1998-1:2004). The ductility classes define the allowed remaining deformation in structural elements, which ultimately is connected to the energy dissipation capacity that reduces the structural response due to earthquake excitation.

The main idea in seismic design is to control the structures behavior by introducing *plastic hinges*. This means that plastic deformations are accepted during an earthquake event without causing the structure to collapse due to its incapability to resist the vertical loads as result of damage occurrence.

1.2 Previous work

Recent reports have been produced about seismic design with a focus on nuclear power plants in Sweden. These are interesting and relevant because they are addressing seismic design in Scandinavia.

Rydell (2014) wrote a licentiate thesis that addresses the seismic response of large concrete structures and summarizes the important factors when the seismic load content is mainly high frequencies. The study evaluates two case studies which indicate that low frequency content and high frequency content have significantly different responses. The report indicates that high frequency seismic vibration may not be damaging to the structure, but should not be neglected for the non-structural elements that are attached to the primary bearing system. Furthermore, the report looks at the change of dynamic properties due to fluid-structure interaction, increasing the structure's vulnerability.

In Tabatabei-Araghi (2014) the differences between the Eurocode 8 and the Swedish standard used for design of nuclear power plants, *Dimensionering av Nukleära Byggnadskonstruktioner* (DNB), are presented. In order to compare the two standards, design examples are computed. The Swedish standard is compared to design in ductility class high (DCH) in Eurocode 8. The results of the study show that Eurocode 8 in combination with the Swedish elastic ground response spectrum gives a more conservative design than DNB.

1.3 Aim and objective

The aim of this thesis is to compare alternative designs for precast structures in EC8. Depending on the site and prerequisites of the structure different ductility classes are prescribed in the standards. The scope is to perform detailing of the horizontal bearing parts of a building according to the two ductility classes allowed in Norway, DCL and DCM.

In order to compare the two design solutions a quantitative and a qualitative comparison will be conducted. The quantitative comparison is based on an arbitrary structural model in which various seismic design inputs act on the structure. Ultimately this will give the reinforcement content needed to obtain sufficient structural capacity. Eventually the designs are compared in measures of reinforcement content depending on the seismic design acceleration input. The principal question is here:

> *Can any conclusions of the structure's cost be drawn regarding the selection of ductility class for the design of the lateral bearing system depending on the location and importance of the building?*

Qualitatively, the detailing of arbitrary bearing elements is compared in measures of *reinforcement set-up*. Furthermore, the seismic forces will be addressed and the consequences of design selection will be evaluated for the structural system.

1.4 Structure of the thesis

Many of the concrete buildings constructed today are precast and thus, this master thesis focus on this type of structures. Knowledge about structural dynamics, seismic design and the relevant norms from the Eurocodes will be introduced in the theoretical part of the thesis to lay the background for the actual design of a precast structure according to the two ductility classes DCL and DCM.

Chapter 2 - Earthquakes
A brief introduction into earthquake mechanisms and relevant quantities are presented in the chapter, as consolidation of the source of seismic forces is crucial.

Chapter 3 - Seismic Design
In this chapter, the behavior of structural systems undergoing seismic excitation is presented. Moreover, their dynamic properties are addressed and the dynamic output is further modified.

Chapter 4 - European Standards
The standards required for building structures in Europe with modifications for the conditions in Norway are presented in this chapter.

Chapter 5 - Computation of the DCL- and DCM-designs
This chapter describes the specific arbitrary structure used for analysis. Further analysis on the procedure of design and detailing is conducted, based on the output from *Robot Structural Analysis Professional 2014* (RSA2014). The design results are ultimately graphically presented so as to get an overview of the relation in reinforcement cost between the DCL-and DCM-designs.

Chapter 6 - Conclusions
The results of the previous chapter are discussed and suggestions for further research are put forward.

2
Earthquakes

The closest tectonic plate boundary to the Scandinavian peninsula is the mid-Atlantic ridge, as seen in Figure 2.1. Norway experiences most seismic excitation in relation to other countries in Scandinavia, even though both Norway and Sweden are considered low seismicity areas (NORSAR, 2014).

Analysis of historical data indicates that earthquakes of magnitude $M_L \geq 5$ (Richter-scale) are anticipated in Norway with a return period of 10 years (NORSAR, 2014).

The largest earthquake recorded to date that caused minor damage to building structures occurred in the outer Oslofjord in 1904 and was estimated to $M_L = 5.4$ (Richter-scale) (Rønnquist *et al.*, 2012), while the most recent moderate one occurred on 15th of September 2014 with its epicentre located 70 km north of Mora. According to seismological measurements conducted by the University of Uppsala the earthquake's magnitude was approximately $M_L \approx 4$ (Richter-scale) (Sveriges Radio, 2014).

Earthquakes are though to be observed every day in Scandinavia, but these are normally negligible and harmless. A quake of magnitude 9 is considered highly improbable on the time scale of relevance (Bödvarsson *et al.*, 2006).

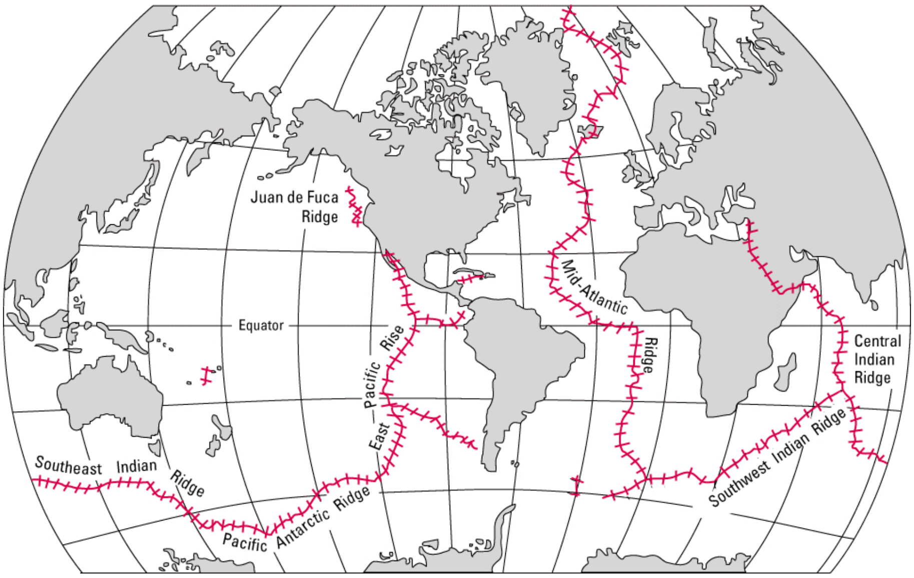


Figure 2.1: *Tectonic plates showing the mid-Atlantic ridge (U.S. Geological Survey, 2014).*

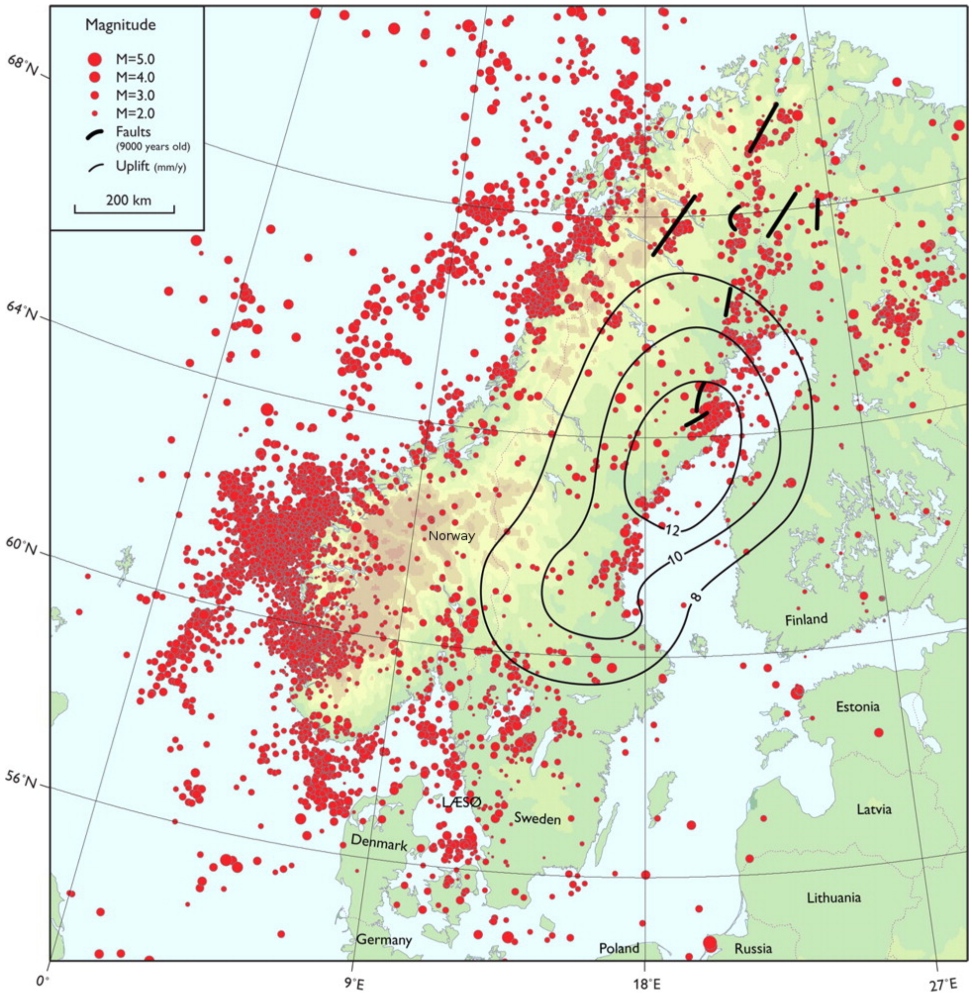


Figure 2.2: *Earthquakes recorded from January 1970 to December 2004 in Northern Europe (Gregersen & Voss, 2014).*

2.1 Seismology and ground parameters

Earthquakes are a scientific phenomenon resulting from natural-geological processes. When there is a disturbance in the balance of mechanical rocks, energy is released in the form of seismic waves, which eventually results in ground movements.

2.1.1 Plate tectonics

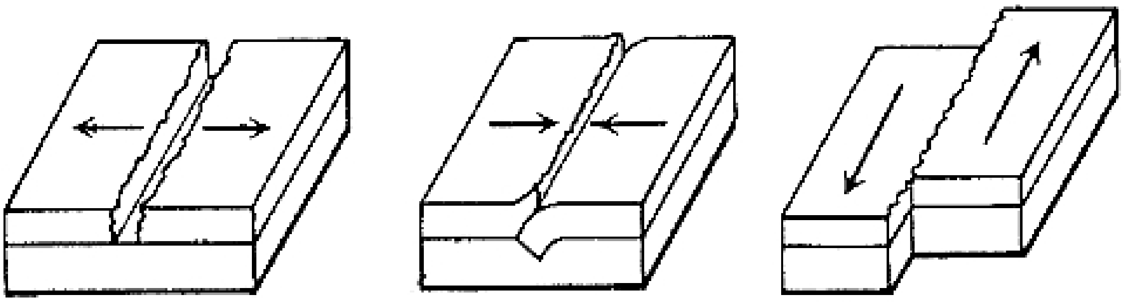
In 1912, Wegener discovered that the different large land masses of the Earth almost fit together like a jigsaw-puzzle, and made the claim that all the continents were once connected as one mass. This large, coherent mass or plate was named *Pangea* and he suggested that over time the plates slowly drifted apart until reaching the location where they are today (Spyrakos & Toutoudaki, 2011).

In the 1960's, Holmes proposed that the Earth's mantle contained convection cells that dissipated radioactive heat and moved the crust at its the surface.

Ultimately this lead to the theory of lithospheric plates as known today. This theory states that the surface of the Earth, the lithosphere, is a stiff crust 80 kilometers thick. It is divided into six continental-sized plates, including *the African*, *the American*, *the Antarctic*, *the Eurasian*, *the Australia-Indian* and *the Pacific*, and 14 of subcontinental sized plates (e.g. *the Caribbean*, *the Cocos*, *the Nazca*, *the Philippine*, etc.). These plates move on the asthenosphere, a plastic layer 100-200 kilometers thick, relative to each other at different velocities. This deformation of the plates can occur slowly and continuously or can occur spasmodically in the form of earthquakes.

The reason for the movement of the lithosphere is not yet fully understood. Some assert that currents in the underlying asthenosphere cause the movement, while others claim that differences in density between the continental and oceanic plates generate the movements in question.

The tectonic plate boundaries are areas of intense geological activity. Tectonic activity is manifested as, earthquakes, and has also resulted in mountain chains, volcanoes and oceanic trenches. When examining the location of earthquake epicenters, one can see that they are mainly concentrated along these plate boundaries. The movement can be characterized as *spreading, subduction* or *transform boundaries*, as seen in Figure 2.3.



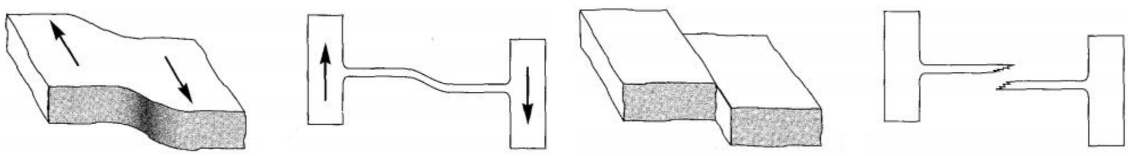
(a) *Spreading (divergent).* **(b)** *Subduction (convergent).* **(c)** *Transform.*

Figure 2.3: *Movement of tectonic plate boundaries (Metzger, 2014).*

2.1.2 Elastic rebound theory

The elastic rebound theory explains how energy is spread during earthquakes. Fielding Reid examined the ground displacements along the San Andreas Fault, that occurred as a result of the 1906 *San Francisco Earthquake.* Observations led to the conclusion that as a relative movement of the plates occurs, elastic strain energy is stored in the materials near the boundary as shear stresses increase on the fault planes that separate the plates (Kramer, 1996).

Ultimately, the maximum shear strength of the rock is achieved and the rock fails, which results in accumulated strain energy release. The effects of the release depend on the nature of the rock. If the rock is *weak and ductile* a small amount of strain energy can be stored and consequently the release will occur slowly and the movement will take place aseismically. If the rock is *strong and brittle,* the failure is rapid. In other words, the stored energy will be released explosively, partly in the form of heat and partly in the form of stress waves that are felt as earthquakes. Figure 2.4 illustrate the elastic rebound theory.



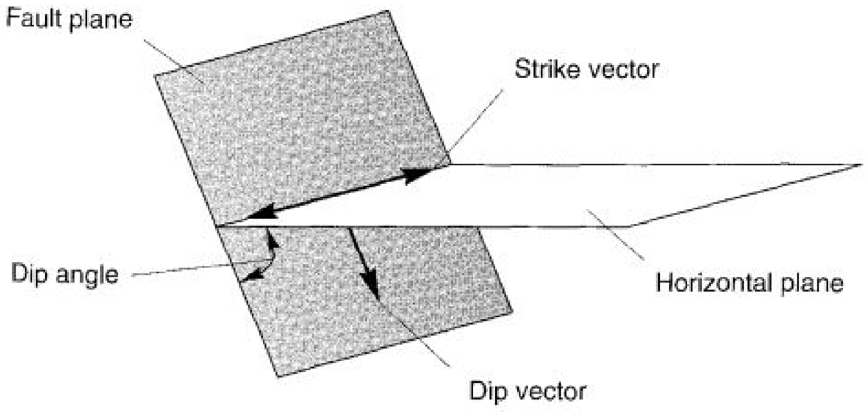
(a) *Deformation of ductile rock.* **(b)** *Fracture of brittle rock.*

Figure 2.4: *Elastic rebound theory (Kramer, 1996).*

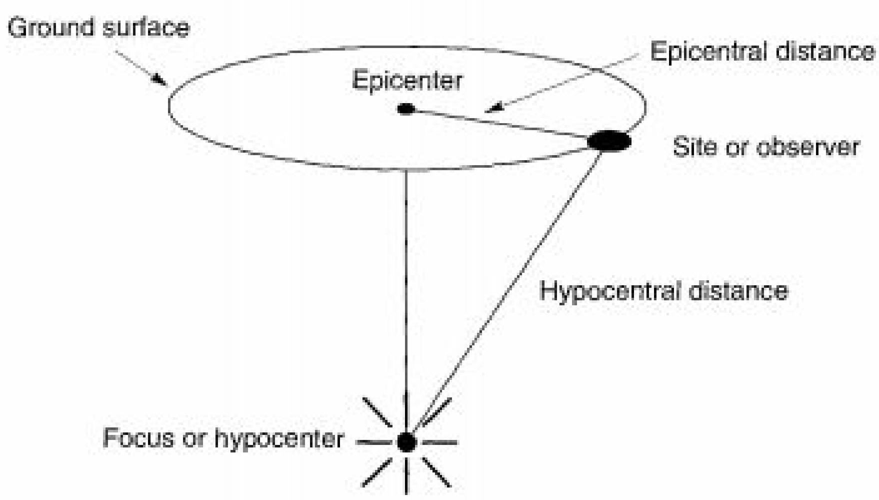
2.1.3 Faults

A fault is the movement between two portions of crust which can be the length of a few meters to hundreds of kilometers. Faults can either be detected on ground level or they can occur at depths of several kilometers. In most cases, the fault rupture does not reach the ground surface (Kramer, 1996).

The geometry of the fault is described by its *strike* and *dip*, as seen in Figure 2.5a. The *hypocenter* of the earthquake is the point at which the rupture begins and the first seismic wave propagates. The point at the ground surface above the hypocenter is called the *epicenter*, and the distance from this point to the *site*, where for instance, the earthquake vibrations for are measured, is called the *epicentral distance*. See Figure 2.5b.



(a) *Geometric notation for description of fault plane orientation.*



(b) *Notation for description of earthquake location.*

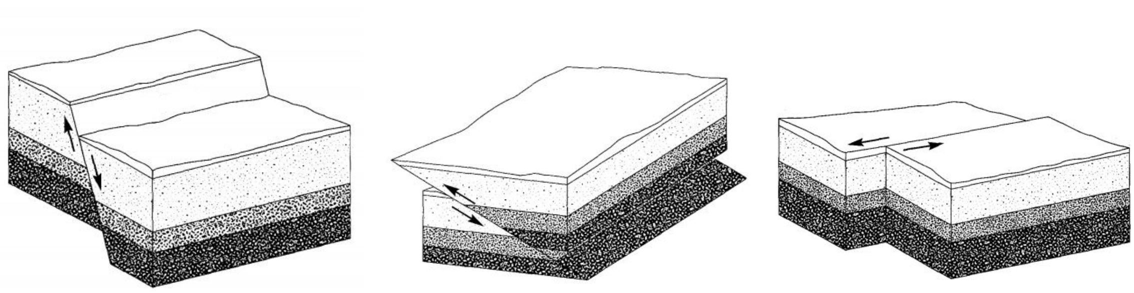
Figure 2.5: *Geometric notations for fault and location (Kramer, 1996).*

The fault movement that occurs in the direction of the dip is referred to as *dip slip* movement. *Normal fault* is considered, the fault-case where the material above the inclined fault moves downward, as seen in Figure 2.6a. This kind of fault generates mainly tensile stresses and ultimately lengthening of the crust. When the material above the inclined fault moves upwards, this is referred to as a *reverse fault*, as seen in Figure 2.6b. *Thrust fault* is a special case of reverse fault, which has a small dip angle.

This sort of fault can result in very large movements and an example of an area where it can be seen is *the European Alps.*

Strike-slip faults (Figure 2.6c) are normally nearly vertical movements and can produce large movements.

As relative movement of the plates occurs *elastic strain energy* is stored in the materials near the fault and this causes shear stresses to develop in the fault plane. The rock fails when these shear forces reach the ultimate strength of the rock and, as a result strain energy is released. Depending on the properties of the rock the strain energy will be released with different velocities. If the rock is ductile, the energy will be released quite slowly and the movement will occur aseismically (Figure 2.4a). If on the other hand, the rock is brittle a faster release of the strain energy will occur, resulting in a more explosive impact on the surrounding soil (2.4b).



(a) *Normal fault.* **(b)** *Reverse fault.* **(c)** *Left lateral strike-slip faulting.*

Figure 2.6: *Fault movement (Kramer, 1996).*

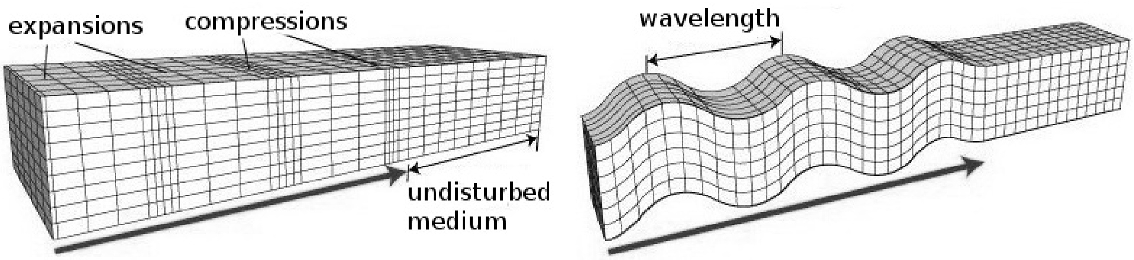
2.1.4 Seismic waves

Waves are generated when fault movement occurs. The waves that are produced are characterized according to mode that they travel through the soil. Mainly, there are two different kinds: *body waves* and *surface waves* (Kramer, 1996).

- Body waves
  - Primary waves
  - Secondary waves
- Surface waves
  - Rayleigh waves
  - Love waves

Body waves travel in the interior of the earth and can be classified as *Primary waves* (P-waves) or *Secondary waves* (S-waves). *P-waves* propagate through the soil by the alteration of the soil medium's volume or density. They are termed Primary waves because they reach the seismograph faster than the S-waves. The dissemination can occur in both solid and liquid mediums and the P-waves are not as destructive as the S-waves (illustrated in Figure 2.7a).

S-waves propagate by shear elastic deformation of the soil medium, i.e. the particles of the soil are polarized perpendicular to the direction of propagation. Since liquids are not susceptible to shear force the S-waves cannot propagate in liquid medium, which proves that in liquefied soil the wave length is significantly decreased (illustrated in Figure 2.7b).



(a) *Primary wave.* **(b)** *Secondary wave.*

Figure 2.7: *Propagation of body waves through soil medium.*

The velocities of the P- and S-waves are given as a function of the elastic moduli G_{shear} and the density ρ_{dens} of the soil medium.

$$V_{p-wave} = \sqrt{\frac{\lambda_{lame} + 2G_{shear}}{\rho_{dens}}} \tag{2.1}$$

$$V_{s-wave} = \sqrt{\frac{G_{shear}}{\rho_{dens}}} \tag{2.2}$$

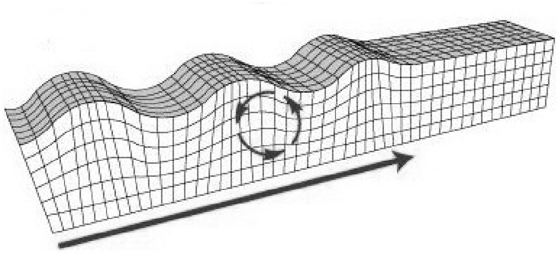
where G_{shear} is the shear modulus, ρ_{dens} is the density and λ_{lame} is the first Lamé parameter.

In general the wave propagation velocity through the ground will increase with increased pressure and vice versa, also will decrease with increasing temperature.

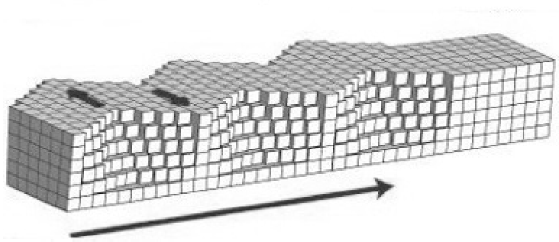
The surface waves propagate at the ground surface. Since they have low frequencies and long duration, they are particularly damaging. They are sub-categorized into *Rayleigh* and *Love waves* and the propagation velocity of these waves is the lowest in relation to the other types.

During the *Rayleigh wave's* propagation through the ground, the soil particles have an elliptic movement around the axis perpendicular to the direction of propagation (Figure 2.8a).

In order for *Love waves* to occur, it is essential for there to be a certain thickness of the layer of the half-space. During the propagation the particles are moving with horizontal oscillations perpendicular to the direction of propagation (Figure 2.8b).



(a) *Rayleigh wave.*



(b) *Love wave.*

Figure 2.8: *Propagation of surface waves through soil medium.*

Figure 2.9 presents the arrival of seismic waves from a random earthquake to the seismograph in a time-history diagram.

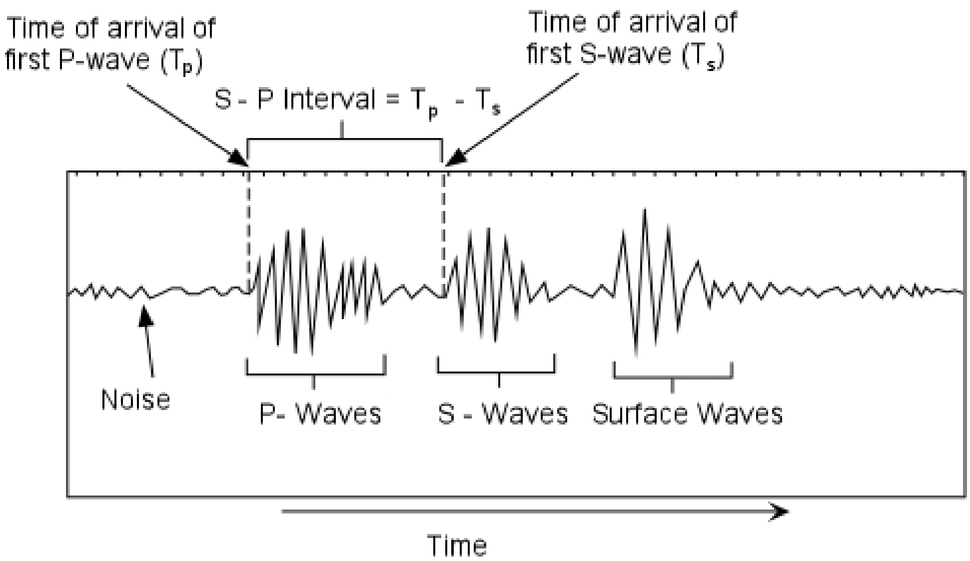


Figure 2.9: *Time-history of random earthquake (Earthsci, 2014).*

2.2 Magnitude and distance effect

The magnitude, M, describes the energy released during an earthquake incident. This energy creates the wave motion in the ground, i.e. the seismic waves. The magnitude of an earthquake is calculated by measuring different seismic parameters of the seismic waves, such as length, duration, period, etc. Due to the variation of waves, different scales of magnitude were developed.

- Local magnitude
- Surface magnitude
- Body wave magnitude
- Moment magnitude

The logarithm for the maximum width of a recorded earthquake is called the *local magnitude*.

The *Richter local magnitude*, M_L, is the best known magnitude scale today, but it is important to mention that it is not always the most appropriate scale for description of the earthquake size.

$$M_L = log_{10} A_W - log_{10} A_0(\delta) = log_{10}\left[\frac{A_W}{A_0(\delta)}\right] \tag{2.3}$$

where A_W is the maximum excursion of the *Wood-Anderson seismograph*, the empirical function A_0 depends only on the epicentral distance of the seismological station, δ.

When looking at a superstructure, a parameter that is important for the prediction of structural response is the actual *governing natural period* of the earthquake's excitation at the specific location (Gazetas, 2013). As shown in Figure 2.10, the predominant period of the earthquake is increasing further away from the rupture. If the natural periods of the soil and superstructure are close to each other, then resonance will occur, which will result in a large amplification of the vibration.

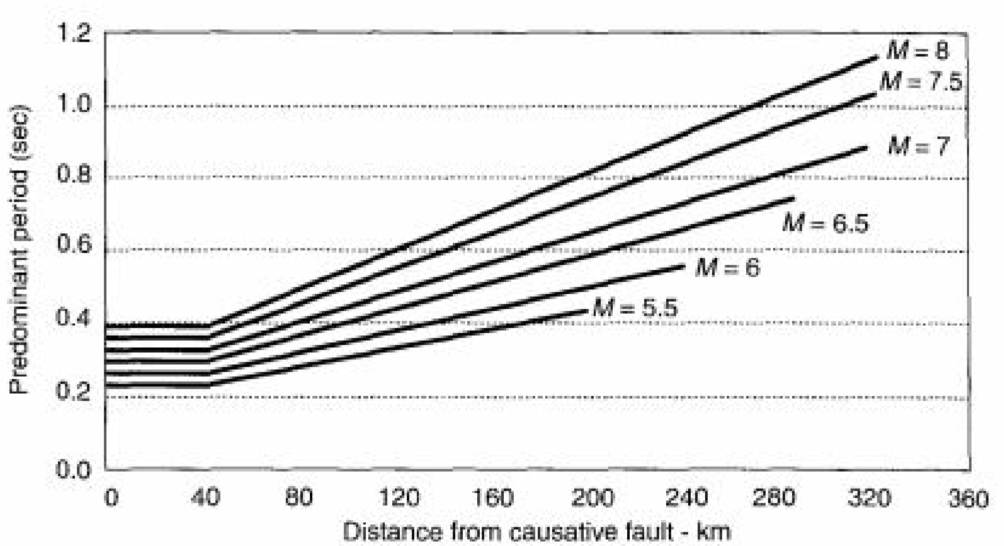


Figure 2.10: *Relation of magnitude and distance to hypocenter (Kramer, 1996).*

Table 2.1: *Richter scale (Spyrakos & Toutoudaki, 2011).*

Magnitude M_L	Description	Effects
>9.0	Great	Severe damage or collapse to all buildings.
8.0-8.9	Great	Major damage to buildings, structures likely to be destroyed.
7.0-7.9	Major	Causes damage to most buildings, some to partially or completely collapse or receive severe damage.
6.0-6.9	Strong	Damage to a moderate number of well-built structures in populated areas.
5.0-5.9	Moderate	Can cause damage of varying severity to poorly constructed buildings.
4.0-4.9	Light	Generally causes none to minimal damage.
3.0-3.9	Minor	Often felt by people, but very rarely causes damage.
2.0-2.9	Minor	No damage to buildings.
<1.9	Micro	Not felt

3
Seismic Design

3.1 Seismic behavior of structures

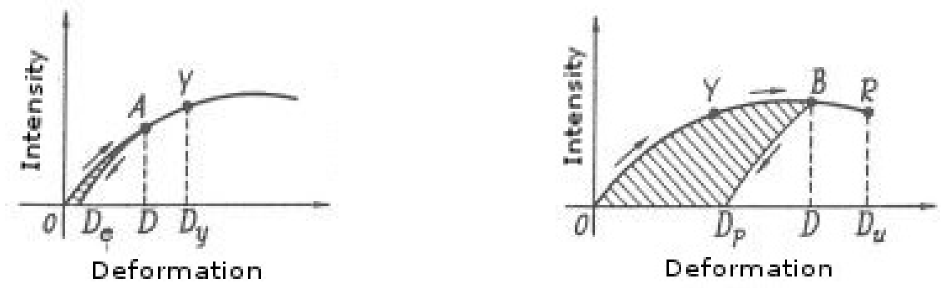
The theoretical background of seismic behavior in this section is mainly based on Anastasiadis' (1989) book about earthquake resistant structures.

The behavior of a structure during seismic impact can be thought of as an *energy balance*. An earthquake will induce energy into the structure. A part of this energy will *dissipate* due to friction, inelastic deformation etc. This is known as *damping energy* that will result in the generation of heat that ultimately dissipates from the structure. The remaining energy causes displacement and movement in the structure. This energy can be categorized as *mechanical energy* which is divided into the displacement energy, and *kinetic energy* (Anastasiadis, 1989).

This energy can be categorized as mechanical energy which is divided into the displacement energy, and kinetic energy. The larger the seismic input energy is, the larger the displacements are, where one part of the energy will be stored and one part will dissipate. The increase of the displacement stops when the output energy equals the input energy. A collapse of the structure is expected to occur if the displacement required to fulfill the energy balance, is larger than the displacement that the structural elements can withstand.

The displacements of the structure are thus important for the diffusion of large parts of the seismic energy. Figure 3.1 shows illustrations of the elastic and inelastic behavior of an arbitrary structure. The displacement due to the earthquake is denoted D. In the first case (Figure 3.1a), where elastic behavior is shown, the area under the graph, i.e. OAD, indicates the stored energy in the structure. In this case, when unloading the structure the energy is converted almost exclusively to kinetic energy, because the remaining displacement D_e is small. This means that the input acceleration that induces the structure to vibrate will barely be reduced, i.e. this is the energy that does not leave the system. Point Y indicates the limit for the elastic deformation and thus the triangle OYD_y defines the *maximum capacity of energy storage* in the system. Due to the fact that the yield displacement D_y is small, the storage of large amounts of energy during a large earthquake requires a stiff structure in order to remain elastic.

Limitations regarding design for extremely stiff structures lead to an alternative design, which allows the structural system to enter the *plastic zone*, i.e. $D > D_y$ thus providing, benefits of energy dissipation in the remaining displacement. In the second case (Figure 3.1b), the area OBD represents the maximum capacity of energy storage. In this case, one can notice that the remaining displacement D_p is large, and as a result, the energy dissipation area OBD_p is larger. In conclusion, ductile structures are desirable for their ability to enter the plastic zone without collapsing.



(a) *Elastic.* **(b)** *Elastoplastic.*

Figure 3.1: *Elastic and inelastic behaviour (Anastasiadis, 1989).*

The phenomena of damping can be described theoretically based on a certain structure with the *hysteresis loop* as a result of the alternating cyclic loading. In Figure 3.2 the hysteresis loop is illustrated based on a structural system. The area of the loop is equal to the consumption of energy which is dissipated during a full *loading and unloading cycle.* The hysteresis loop will form its shape depending on the materials of the components, and so a number of different shapes are possible. Therefore, the assumption of *linear viscous damping* is usually made, which results in an elliptic shape of the loop. The damping ratio ξ is the relation between the *absorbed energy,* i.e. of the ellipse, and the *elastic energy* (Eq. (3.1)).

$$\xi = \frac{1}{4\pi}\frac{\Delta E}{E} \tag{3.1}$$

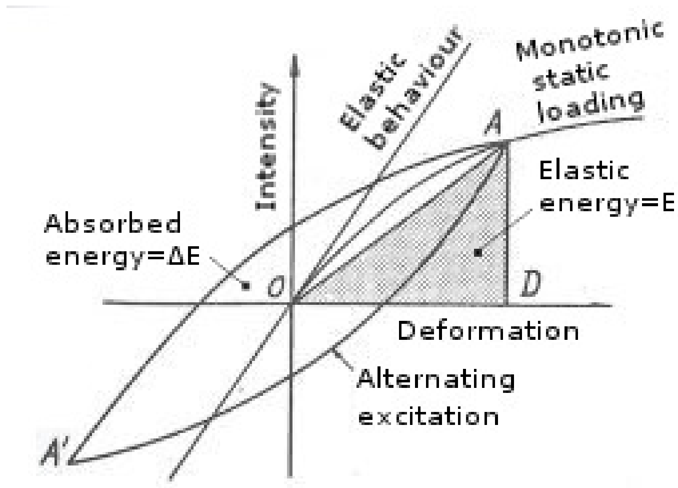


Figure 3.2: *Calculation of the damping ratio (Anastasiadis, 1989).*

The dimension of the hysteresis damping depends on the plastic deformation, i.e. the larger the plastic deformation the structure is able to undergo, the larger the damping that will occur. *Ductility* is the ability of a structural component to deform when entering the plastic zone. All things considered, the ductility is of prime importance in the seismic design of structures. Figure 3.3 illustrates a force-displacement diagram of a structural system for static loading until rupture. It exemplifies an elastoplastic system where the definition of ductility can be defined as:

$$\mu = \frac{D_u}{D_y} \tag{3.2}$$

where μ is the ductility factor, D_y is the yield displacement and D_u ultimate displacement.

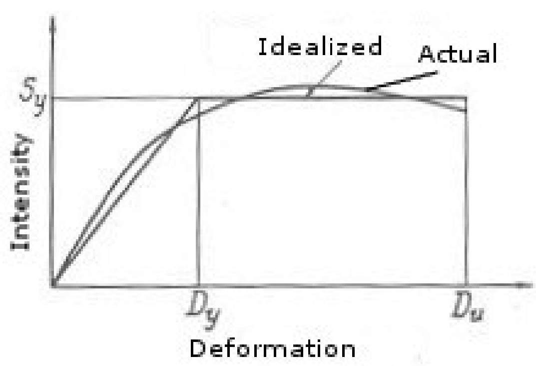


Figure 3.3: *Elastoplastic resistance model (Anastasiadis, 1989).*

The opposite of a ductile material is brittle material. If the structure is brittle, it lacks the ability to deform in the plastic zone as much as a ductile structure. In Figure 3.4 the difference in behavior between the two types are schematically depicted. R_1 is a brittle material with low ductility factor and R_2 is a ductile material with high ductility factor. The main difference regarding the seismic energy input in these two systems is that even though both systems will absorb the same amount of energy momentarily, the first one will "return" the energy to the structure in mainly kinetic energy while the second will "consume" the energy in the form of heat.

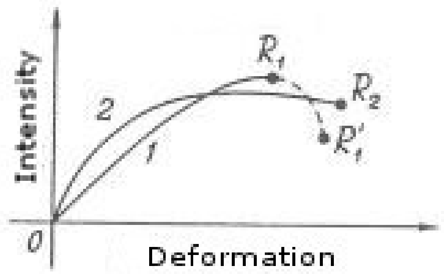
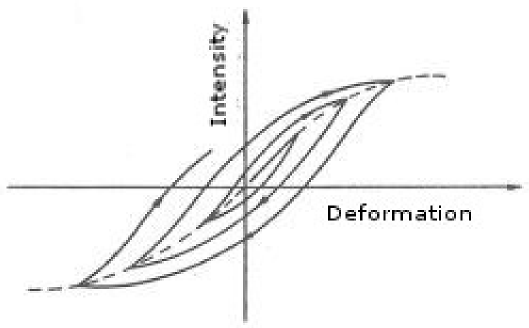


Figure 3.4: *Ductile and brittle materials (R_1:Brittle R_2:Ductile) (Anastasiadis, 1989).*

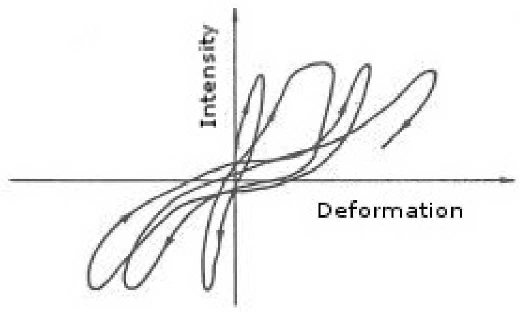
3.2 Hysteresis behavior of structures

The size of the ductility and the shape of the hysteresis loop depends on two main factors: absorption and dissipation of energy, as well as the phenomena found from empiric tests when inducing structures with seismic forces.

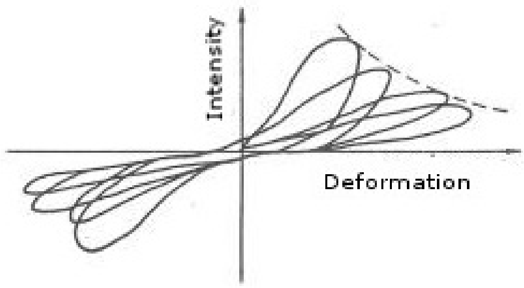
In Figure 3.5, three types of hysteresis loops are illustrated. First, Figure 3.5a depicts a hysteresis loop that remains stable when the cyclic loading is subjected. This is typical for steel structures, as well as for reinforced concrete with dense *transverse reinforcement* (stirrups/hoops). The stiffness of the structure in the second illustration (Figure 3.5b) degrades during the loading and unloading, but the resistance remains constant. This behavior is a result of the general structural components of reinforced concrete. The degradation is explained by the cracks in the concrete that reduce interaction with the steel reinforcement. This hysteresis loop can be detected in steel elements that suffer from local buckling. In the last case (Figure 3.5c) both the stiffness and the resistance degrades. This kind of shape results when the cyclic load affects wall elements and elements of reinforced concrete with large shear force.



(a) *Stable stiffness and resistance.*



(b) *Degrading stiffness.*



(c) *Degrading stiffness and resistance.*

Figure 3.5: *Characteristic shapes of hysteresis loops (Anastasiadis, 1989).*

3.2.1 Reinforced concrete structures

Non-reinforced concrete is quite brittle. If subject to cyclic compression until yielding the hysteresis loop will show a degradation of resistance capacity, as seen in Figure 3.6.

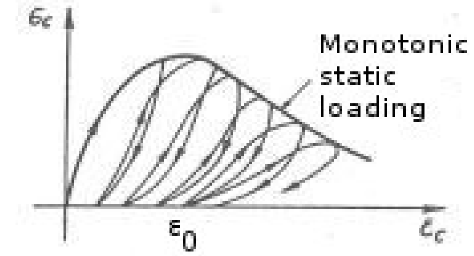


Figure 3.6: *Cyclic compression of concrete (Anastasiadis, 1989).*

The peaks of the hysteresis loop are tangent to the *curve of monotonic static loading*, which at strain $\varepsilon_0 \approx 2‰$ displays the *deceasing resistance section.* The higher the quality of the concrete, the larger the angle will be in the increasing and deceasing sections. Thus, higher quality concrete is more brittle, which is undesirable in seismic design.

Generally, to increase the capacity of the concrete element, transverse reinforcement is used to enclose the concrete and obtain a *triaxial stress state*, as seen in Figure 3.7. A more transverse reinforcement content results in a smaller angle in the decreasing part of the diagram, i.e. more ductile behavior.

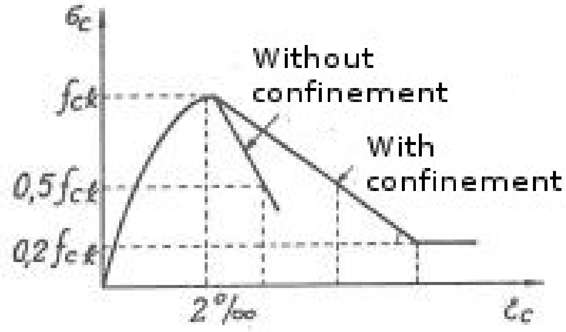


Figure 3.7: *Enclosure of concrete (Anastasiadis, 1989).*

The influence of the spacing between the transverse reinforcement is shown in Figure 3.8. One of the aspects vital for selection of spacing is the buckling of the longitudinal reinforcement.

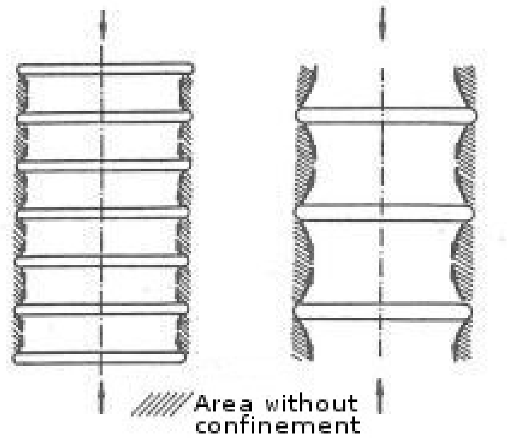


Figure 3.8: *Affection of distance of transverse reinforcement (Anastasiadis, 1989).*

The ductility factor of a reinforced concrete element can either be calculated with Eq. (3.2) or following relationship:

$$\mu = \frac{D_L}{D_y} \tag{3.3}$$

This is valid if the force S_u, which is corresponding to the ultimate deformation D_u, is smaller than $S_L = 0.8S_m$. D_L is calculated based on S_L that is determined according to Figure 3.9.

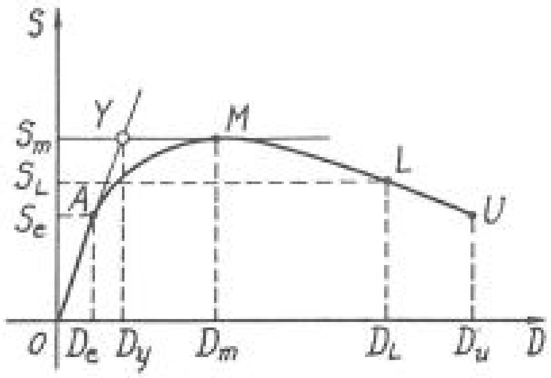


Figure 3.9: *Influence of distance of transverse reinforcement (Anastasiadis, 1989).*

3.2.2 Walls

Shear walls are the most effective structural bearing component to resist horizontal earthquake excitation. Walls can be characterized as *slender*, i.e. $h/l > 1.5$, whose behavior resembles that of beams. If the relation $h/l < 1.5$, the wall is classified as a *short wall* whose behavior incorporates special attributes. In Figure 3.10 three failure mechanisms are illustrated. One is due to bending and two are due to shear in slender walls. For small shear and normal force, the tension reinforcement fails first, which results in horizontal cracks. Moreover, failure of the concrete on the opposite side will occur, see Figure 3.10a.

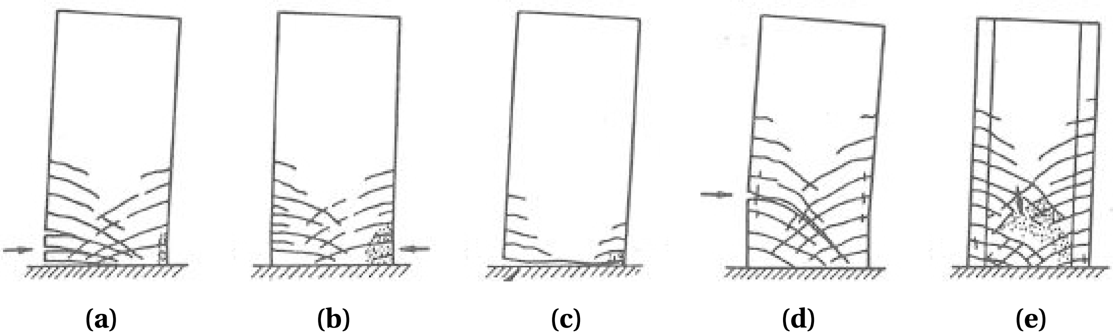


Figure 3.10: *Wall failure mechanisms.* ***(a)*** *Yielding of reinforcement in tension.* ***(b)*** *Rupture of concrete in compression.* ***(c)*** *Fracture of reinforcement.* ***(d)*** *Yielding of longitudinal and transverse reinforcement.* ***(e)*** *Rupture of body-concrete (Anastasiadis, 1989).*

The hysteresis behavior of an arbitrary wall is shown in Figure 3.11. The point $B1$ indicates that the ductility factor is in order of 30, which could be increased even more if the transverse reinforcement is more dense, as seen in $B3$. The hysteresis loop indicates that the structure is able to dissipate a high amount of energy.

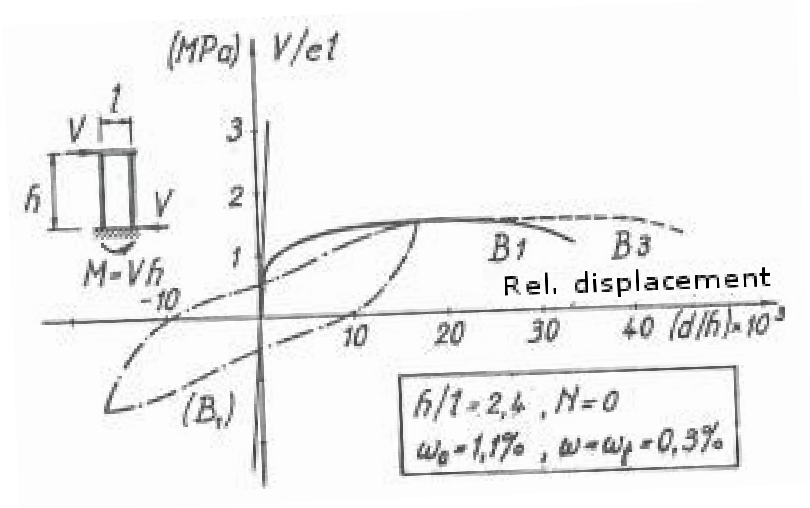


Figure 3.11: *Wall failure mechanism due to dominating bending (B1: $\mu = 30$ B2: $\mu > 30$) (Anastasiadis, 1989).*

Figure 3.10b shows what happens in a case of large normal force and strong bending reinforcement. This set-up will give failure of the concrete, recession of the ductility factor and in general, unfavorable behavior in comparison with Figure 3.10a. In Figure 3.10c the bending reinforcement fails, which occurs at locations where the bond between concrete and reinforcement happens.

In Figure 3.10d and Figure 3.10e the mechanism of failure is due to shear. The case in Figure 3.10d corresponds to the case in Figure 3.10a, i.e. failure of the bending and transverse reinforcement. This creates slanting failures due to the dominating shear force. To the contrary, in Figure 3.10e, the high shear force results in failure of the concrete in the middle of the wall, due to high resistance of the bending and transverse reinforcement. The hysteresis behavior of this case is illustrated in Figure 3.12. The corresponding hysteresis behavior is considered satisfactory. In comparison to Figure 3.11 the ductility factor is lower and the hysteresis loop has contracted. The existence of axial forces leads to higher resistance, but further regression of the ductility factor ($B7$).

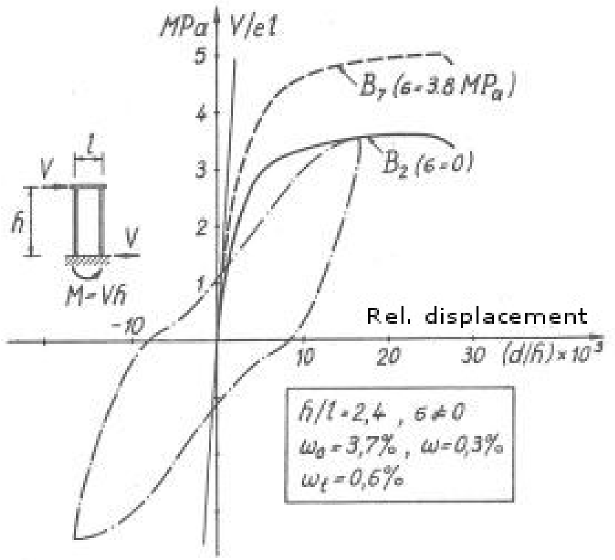
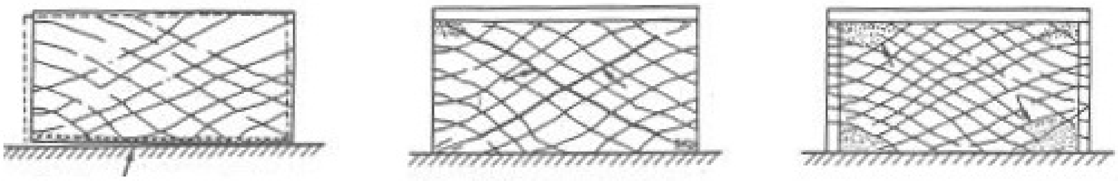


Figure 3.12: *Wall failure mechanism due to dominating shear (Anastasiadis, 1989).*



(a) *Sliding at base.* **(b)** *Crosswise fractures.* **(c)** *Fractures in diagonal compression zones.*

Figure 3.13: *Wall failure mechanism of short walls (Anastasiadis, 1989).*

Figure 3.13 illustrates the three main failure mechanisms of short walls. In Figure 3.13a, sliding occurs at the base, which could be a result of progressive plasticity of the longitudinal reinforcement due to bending and shear. In Figure 3.15, the hysteresis behavior of this failure mechanism is illustrated. The ductility factor is decreasing and the area of the loop is smaller, however, the energy dissipation capacity is still high due to a high amount of reinforcement.

In the case of Figure 3.13b, the failure mechanism of shear appears, i.e. slanting cracks, where the horizontal and vertical reinforcement have reached yielding. In Figure 3.13c, the compressed concrete in the corner fails. This occurs when the reinforcement content of the wall is large and the shear force is high. The behavior is almost exclusively elastic and a very small amount of energy can dissipate.

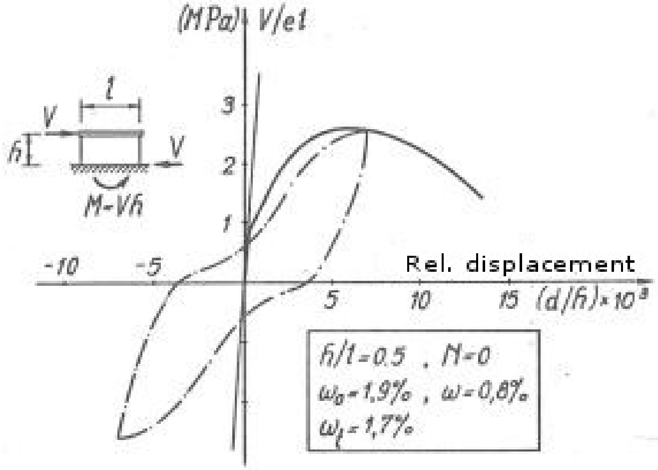


Figure 3.14: *Wall failure mechanism due to dominating shear (Anastasiadis, 1989).*

Walls have, with adequate reinforcement design, excellent plastic behavior with the ability to dissipate large amounts of energy. Of importance is the transverse reinforcement, placed at the edges of the wall, that contributes to the state of triaxial stress in the concrete element. This reinforcement should be detailed in the same way as for columns with a small distance between each other. In Figure 3.15, the hysteresis behavior of a wall is depicted, which even with sufficient eccentric axial force, shows stable hysteresis loops with decent ductility.

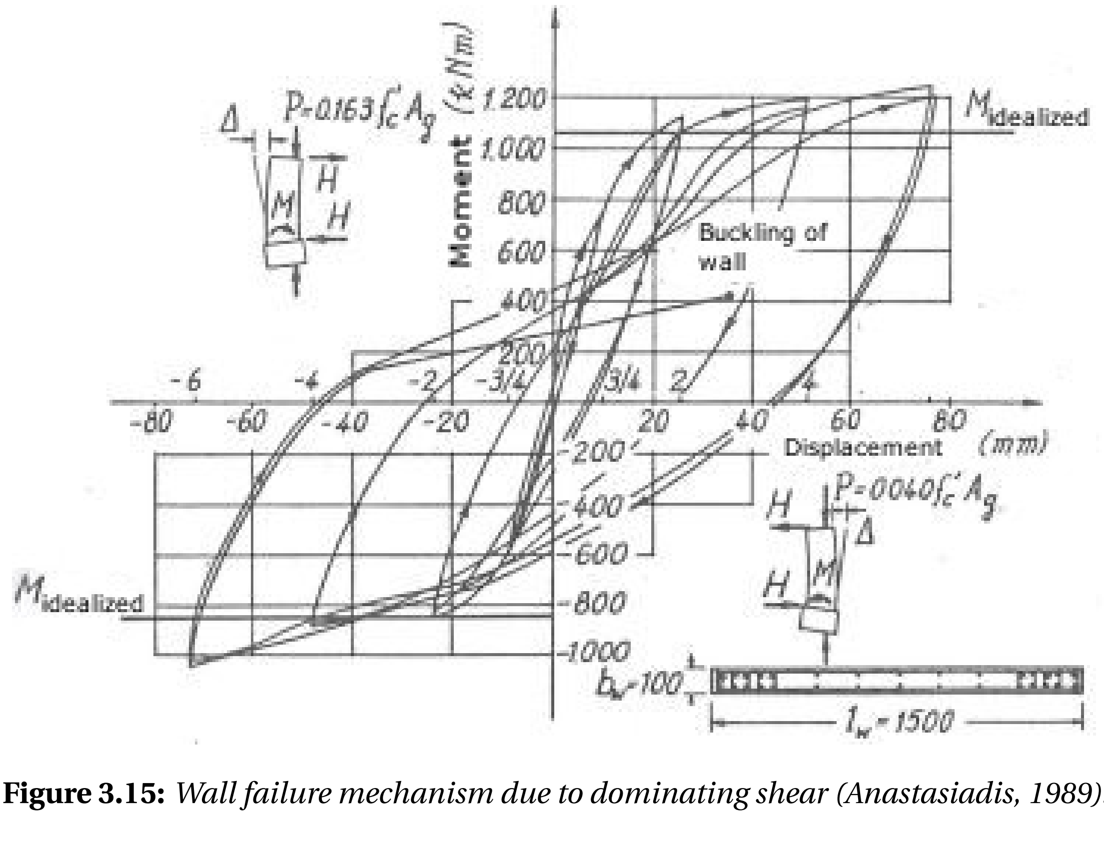


Figure 3.15: *Wall failure mechanism due to dominating shear (Anastasiadis, 1989).*

3.3 Structural dynamics

3.3.1 Dynamic forces and vibration

A dynamic force $p(t)$ changes with respect to time in contrast to a static force p that is monotonic and adopts a constant value. This means that static problems are constant in time and dynamic problems are time-dependent. Ultimately, dynamic forces can be classified depending on manner of change over time. Figure 3.16 shows examples of dynamic loads. With the *harmonic vibration*, the source could be a rotating machine in a building, while the *periodic vibration* could be the result of a rotating ship propeller. The *impulse vibration* typically a result of a blast load, while the *random vibration* in the last subfigure illustrates a time-history of an earthquake excitation (El Centro-earthquake, 1940).

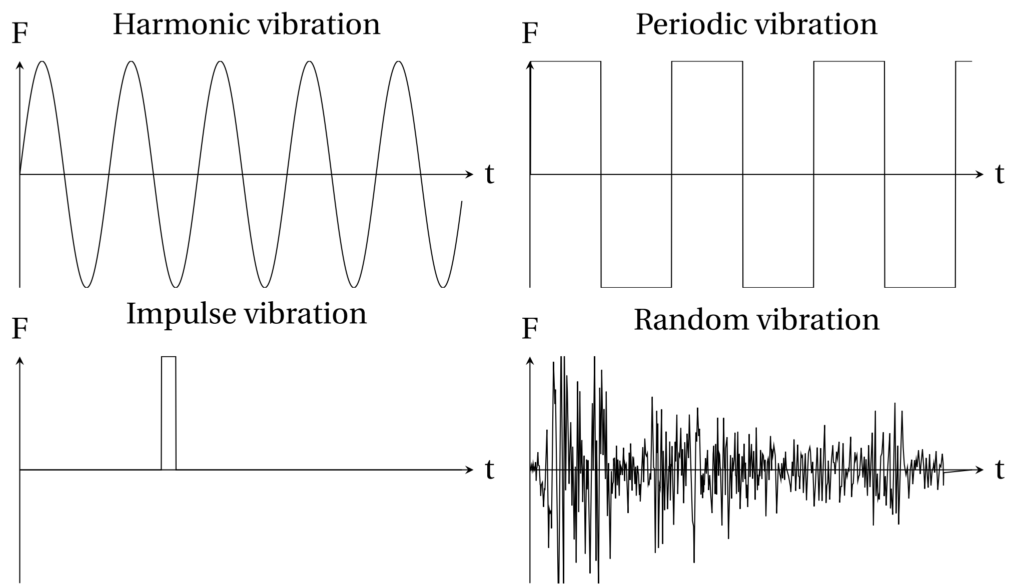
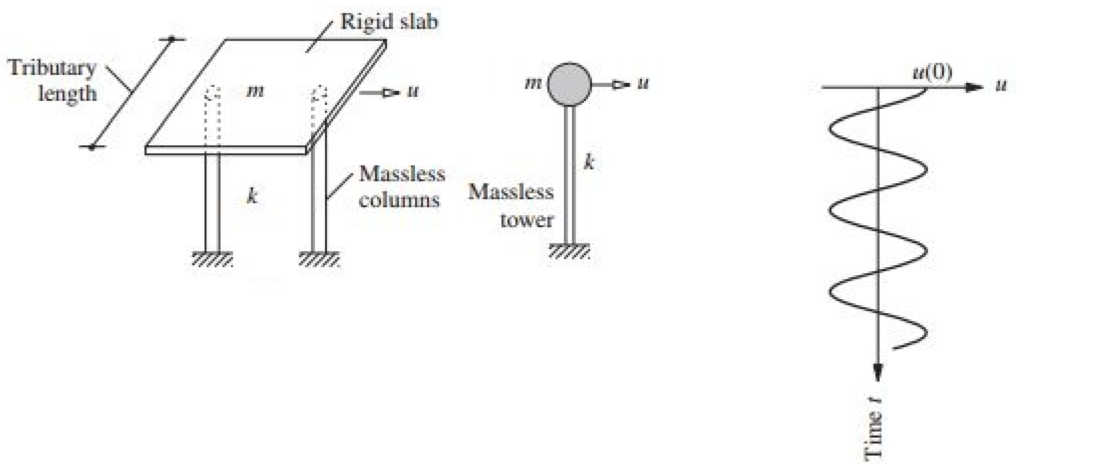


Figure 3.16: *Time histories of dynamic load types.*

3.3.2 Single-degree-of-freedom systems

Idealization

To understand the concept of dynamic problems a system can be simplified into a system with lumped mass m supported by a massless structure with stiffness k. The assumption that the supporting system can be considered massless is permissible because the lumped mass is much heavier than the weight of the system (Chopra, 2007). Examples of idealized systems are illustrated in Figure 3.17.



(a) *Idealized pergola.* **(b)** *Idealized water tank.* **(c)** *Free vibration due to initial displacement* $u(0)$.

Figure 3.17: *Idealization of single-degree-of-freedom systems (Chopra, 2007).*

A single-degree-of-freedom (SDOF) system can be modeled as a mechanical system. A idealized SDOF-system and the corresponding free body diagram is shown in Figure 3.18.

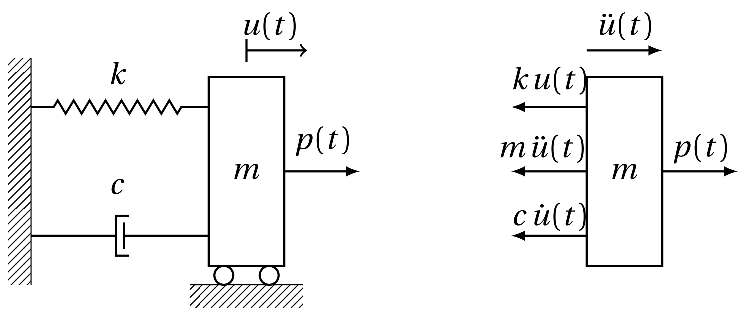


Figure 3.18: *Free body diagram of single-degree-of-freedom system.*

System characteristics

Given theses properties, the dynamic characteristics that follow can be calculated for these systems. The natural period T_n of the system:

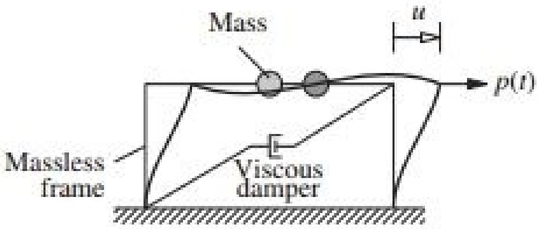
$$T_n = 2\pi\sqrt{\frac{m}{k}} \tag{3.4}$$

where m is the lumped mass and k is the stiffness of the system.

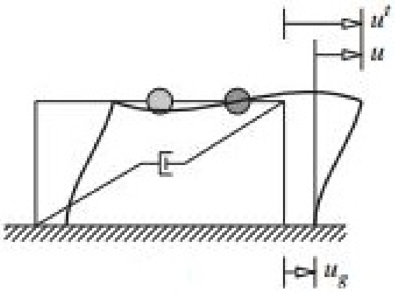
The natural frequency ω_n of the system:

$$\omega_n = \frac{2\pi}{T_n} \tag{3.5}$$

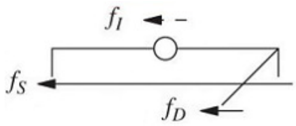
Further on a viscous damper c can be added that dissipates energy from the system. This means that three properties are defined, which are concentrated to separate system components. This is illustrated in Figure 3.19. Two different types of excitations are inducing this SDOF-system. In Figure 3.19a, an applied dynamic force $p(t)$ is vibrating the system whereas and in Figure 3.19b, earthquake ground motion is vibrating the system.



(a) *Applied force.*



(b) *Ground motion.*



(c) *Internal forces.*

Figure 3.19: *Single-degree-of-freedom system (Chopra, 2007).*

In reality, each structural member of a structure will contribute to these three components, i.e. the inertial (m), elastic (k) and energy dissipation (c) properties of the structural system (Chopra, 2007).

The damping ratio ξ is a ratio between the damping coefficient c and the critical damping coefficient $c_{cr} = 2\sqrt{km}$.

$$\xi = \frac{c}{c_{cr}} \tag{3.6}$$

System response

The main objective of dynamic analysis is to evaluate the displacement time-history of a structural system subjected to a dynamic load. The *equation of motion* of the structure can define the dynamic displacements that are sought. The rate of change of momentum of any particle, with mass m, is equal to the force acting on it, as *Newtons II law of inertia* states (Karoumi, 2013). For a SDOF-system, as seen in Figure 3.18, the following dynamic equilibrium can be expressed:

$$p(t) - ku(t) - m\ddot{u}(t) = 0 \tag{3.7}$$

where $m\ddot{u}(t)$ is the *inertial force* resisting the acceleration of the mass. *D'Alambert's principle* states that the inertial force that a mass develops is proportional to its acceleration and opposing mass.

In order for a structural system to vibrate, either an *external excitation force* is applied and/or one or more *initial conditions* are non-zero values, i.e. an initial displacement or initial velocity.

If only the second condition, mentioned above, induces the system vibration it is defined as a *free vibration*. The equation of motion is then expressed with the right hand value equivalent to zero, see Eq. (3.8).

$$m\ddot{u}(t) + c\dot{u}(t) + ku(t) = 0 \tag{3.8}$$

with initial conditions $u(t=0) = u_0$ and/or $\dot{u}(t=0) = \dot{u}_0$.

If the vibration is induced by an external dynamic force it is defined as a *forced vibration* with the right hand side of the equation of motion equivalent to the dynamic force $p(t)$, see Eq. (3.9).

$$m\ddot{u}(t) + c\dot{u}(t) + ku(t) = p(t) \tag{3.9}$$

As illustrated in Figure 3.19b the displacements of the system are defined in the manner displayed in Eq. (3.10). This means that the total horizontal displacement of the mass is a sum of the displacement of the ground and the relative displacement of the mass with respect to the ground.

$$u^t(t) = u(t) + u_g(t) \tag{3.10}$$

where $u^t(t)$ is the total displacement, $u(t)$ is the relative displacement and $u_g(t)$ is the ground motion.

The earthquake excitation is considered a *free vibration* with an initial displacement. Eq. (3.11) shows the dynamic force equilibrium of the system that is also illustrated in Figure 3.19c.

$$f_I(t) + f_D(t) + f_S(t) = 0 \tag{3.11}$$

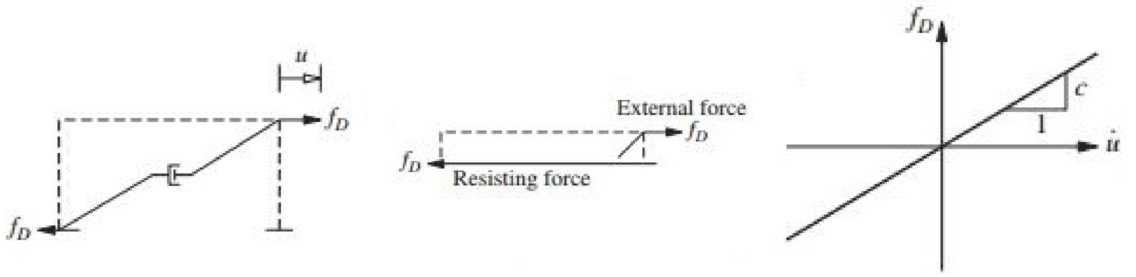
where $f_I(t)$ is the force of inertia, related to the mass of the system, $f_D(t)$ is the damping force and $f_S(t)$ is the stiffness force.

Newton's II law, $F = ma$, gives $f_I(t)$:

$$f_I(t) = m\ddot{u}^t(t) = m(\ddot{u}(t) + \ddot{u}_g(t)) = m\ddot{u}(t) + m\ddot{u}_g(t) \tag{3.12}$$

In a SDOF-system the damping force can be idealized by a linear viscous damper or dashpot. Figure 3.20 shows the damping force related $f_D(t)$ related to the velocity $\dot{u}(t)$.

$$f_D(t) = c\,\dot{u}(t) \tag{3.13}$$



(a) *Model.* **(b)** *Resisting force.* **(c)** *Force-acceleration graph.*

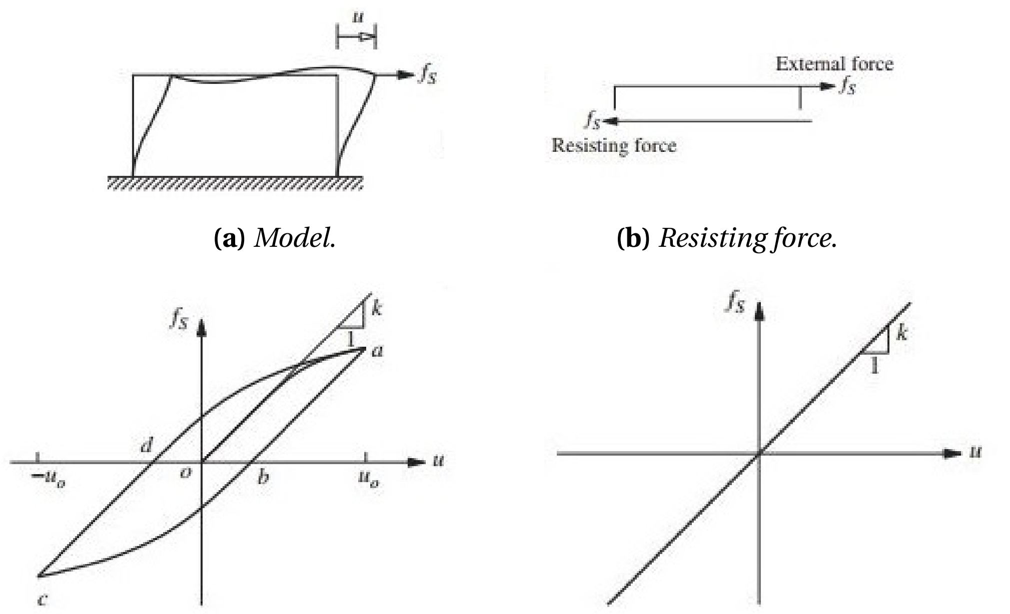
Figure 3.20: *Damping force (Chopra, 2007).*

Eq. (3.14) defines a linear system, where the relationship between the lateral force $f_S(t)$ and the displacement $u(t)$ is linear. The linear relationship indicates that the system is elastic, i.e. the loading and unloading curves are identical.

$$f_S(t) = k\,u(t) \tag{3.14}$$

The system is inelastic if the initial loading curve is non-linear at the larger deformations and the unloading an reloading curves differs. This relationship is described in Eq. (3.15).

$$f_S(t) = f_S(u(t)) \tag{3.15}$$



(a) *Model.*

(b) *Resisting force.*

(c) *Force-displacement graph (Elasto-plastic).*

(d) *Force-displacement graph (Elastic).*

Figure 3.21: *Stiffness force (Chopra, 2007).*

where m is the mass, c is the damping coefficient, k is the stiffness, $\ddot{u}(t)$ is the response acceleration, $\dot{u}(t)$ is the response velocity and $u(t)$ is the response displacement.

By inserting Eqs. (3.12 – 3.14) into Eq. (3.11) and rearranging it, the following equation results:

$$
\begin{aligned}
f_I(t) + f_D(t) + f_S(t) &= 0 \\
(m\,\ddot{u}(t) + m\,\ddot{u}_g(t)) + c\,\dot{u}(t) + k\,u(t) &= 0 \\
m\,\ddot{u}(t) + c\,\dot{u}(t) + k\,u(t) &= -m\,\ddot{u}_g(t) \qquad (3.16)
\end{aligned}
$$

When comparing Eq. (3.16) with Eq. (3.9) one can see that the right hand side can be likened to a force that induces the vibration in the degree-of-freedom in question. Thus, the following observation is made:

$$p_{eff}(t) \equiv p(t) = -m\,\ddot{u}_g(t) \qquad (3.17)$$

where $p_{eff}(t)$ is the effective earthquake force on the structure as shown in Figure 3.22.

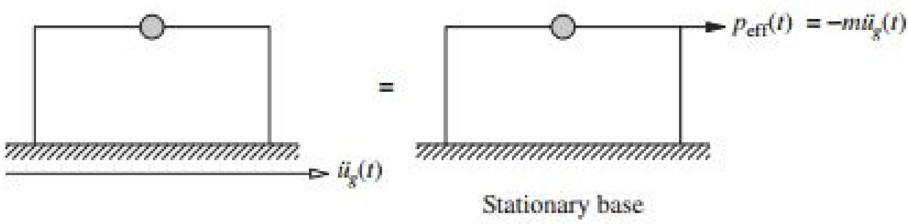


Figure 3.22: *Effective earthquake force (Chopra, 2007).*

If, for example, in an elastic one story system, as shown in Figure 3.23a, the equivalent static force $f_S(t)$ is applied and with respect to time, the computation is as follows:

$$f_S(t) = k\,u(t) \tag{3.18}$$

$$V_b(t) = f_S(t) \tag{3.19}$$

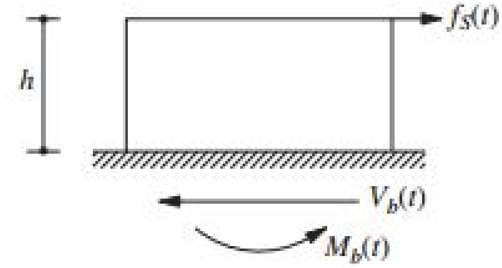
$$M_b(t) = h\,f_S(t) \tag{3.20}$$

where $V_b(t)$ is the shear force at base and $M_b(t)$ is the moment at base.

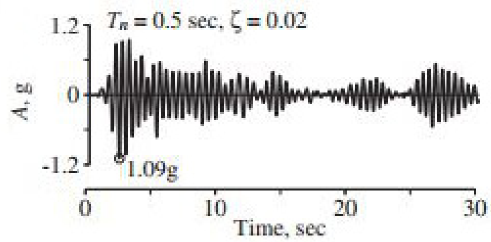
Time-history response

Ultimately the acceleration acting on the structure can be calculated with the relation shown in Eq. (3.21). The response of a certain earthquake ground motion input on the system is shown in Figure 3.23.

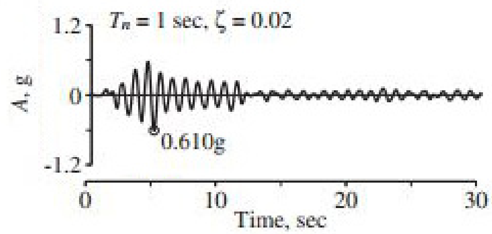
$$f_S(t) = k\,u(t) = m\omega_n^2\,u(t) = m\,a(t) \tag{3.21}$$



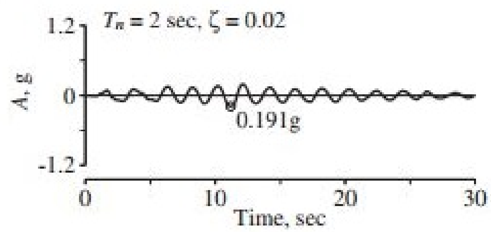
(a) *Engineering response quantities: moment at base, shear force at base and equivalent static force.*



(b) *Pseudo-acceleration response of SDOF-system to El Centro ground motion (T_n = 0.5 s and $\xi = 2\%$).*



(c) *Pseudo-acceleration response of SDOF-system to El Centro ground motion (T_n = 1 s and $\xi = 2\%$).*



(d) *Pseudo-acceleration response of SDOF-system to El Centro ground motion (T_n = 2 s and $\xi = 2\%$).*

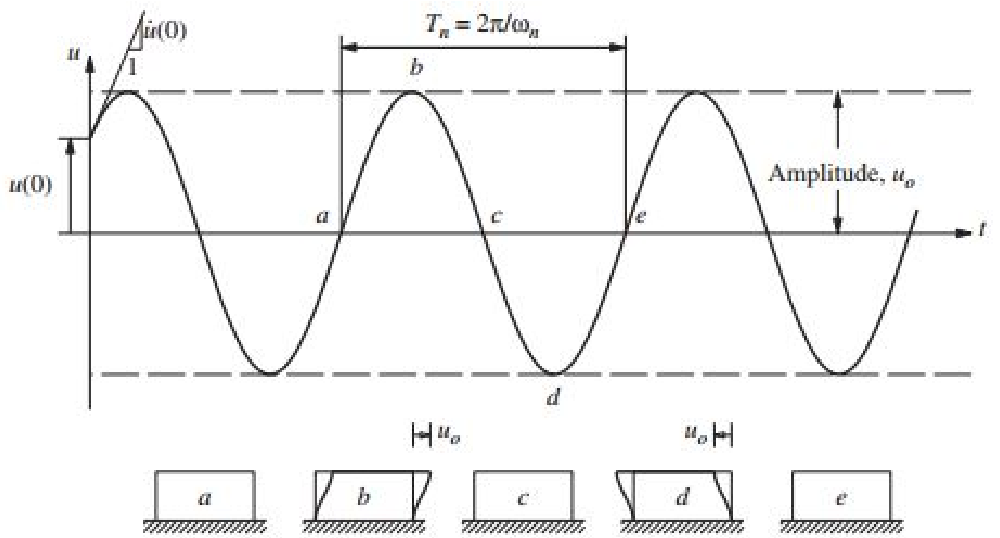
Figure 3.23: *Engineering response quantities (Chopra, 2007).*

Effect of damping

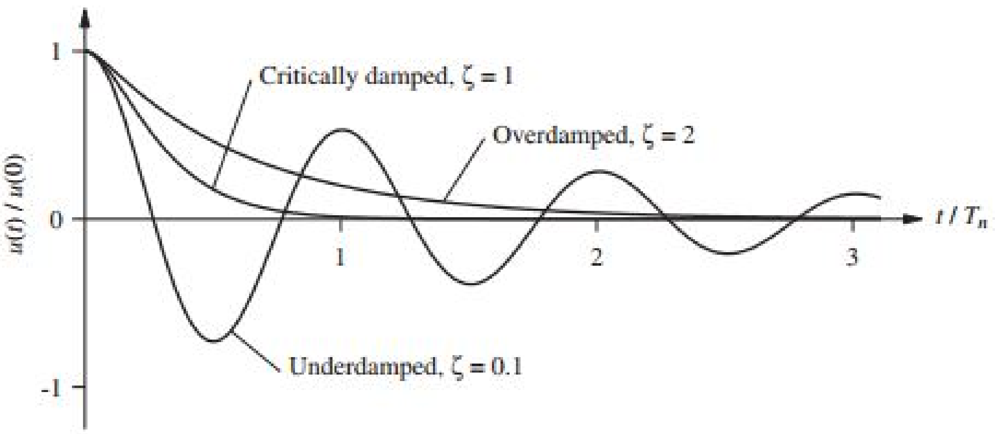
Depending on this ratio the system can be characterized in the following *damping categories.*

- *Undamped* if $\xi = 0$.
- *Underdamped* if $0 < \xi < 1$.
- *Critically damped* if $\xi = 1$.
- *Overdamped* if $\xi > 1$.

Figure 3.24 shows the time-history response of a SDOF-system excited by a initial displacement of $u(t) = u_0 = 1$, i.e. *free vibration.*



(a) *Free undamped vibration.*



(b) *Free damped vibration.*

Figure 3.24: *Free vibration (Chopra, 2007).*

Given the design of a structural system, the following properties can be calculated:

The *natural damped frequency* of the system. Thus ξ usually is small ($\xi \leq 5\%$) for structural systems $\omega_D \approx \omega$.

$$\omega_D = \omega_n \sqrt{1-\xi^2} \tag{3.22}$$

The *deformation response factor* is the ratio between the dynamic displacement $u(t)$ and the static displacement u_{st}. Shown in Figure 3.25 are different damping ratios ξ and input force excitation frequencies Ω.

$$R_d(t) = \frac{u(t)}{u_{st}} \tag{3.23}$$

Frequency ratio:

$$\beta_f = \Omega/\omega_n$$

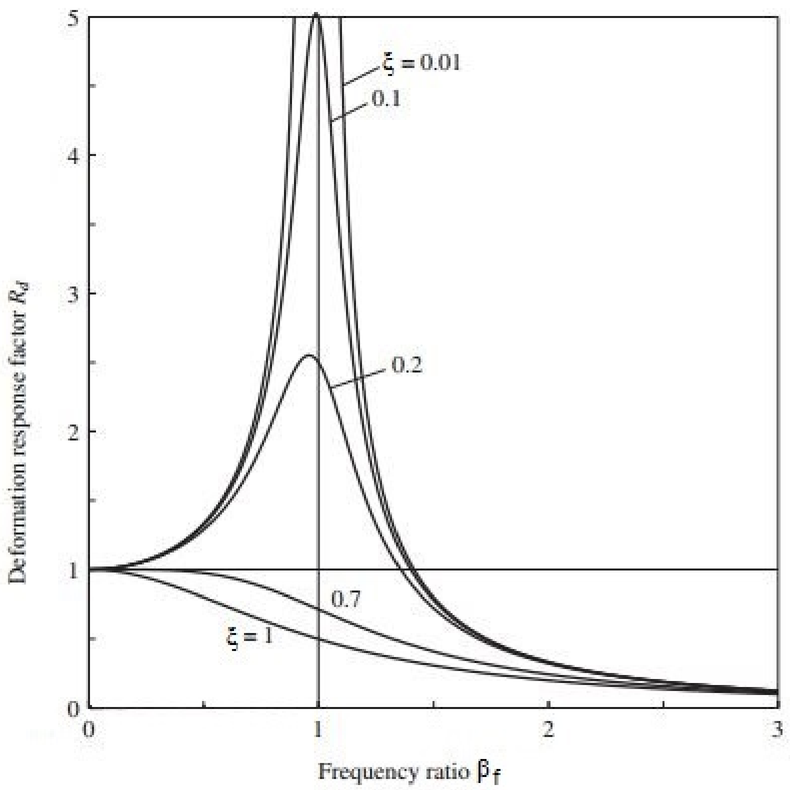


Figure 3.25: *Deformation response factor for system excited by a harmonic force with changing excitation frequency* Ω *(Chopra, 2007).*

Figure 3.25 shows that if the excitation frequency is the same as the natural frequency of the structure, resonance will occur and the amplification will be large. If the system is totally undamped, the amplification will be infinite.

3.3.3 Response spectra

Elastic response spectrum

The seismic displacement should be calculated based on the displacement at the foundation, and subsequently, the deformation and intensity considering the inelastic properties of a structure. This procedure is not possible, therefore the procedure of calculating the response occurs in reverse order.

This means that data regarding the excitation is necessary to compute the intensity and deformation. The preliminary calculation of seismic forces are done with so called *response spectra*.

The equilibrium of a SDOF-system is described with the differential equation of motion:

$$m\ddot{u}(t)+c\,\dot{u}(t)+k\,u(t)=0 \tag{3.24}$$

and subsequently with Eq. (3.10) following is computed:

$$\ddot{u}(t)+2\xi\,\dot{u}(t)+\omega^2\,u(t)=-\ddot{u}_0(t) \tag{3.25}$$

With *zero* initial conditions $u(0) = \dot{u}(0) = 0$ the following Duhamel integral is computed:

$$u(t)=-\frac{1}{\omega_d}\int_0^t \ddot{u}_0(\tau)\mathrm{e}^{-\xi\omega_n(t-\tau)}\sin\omega_d(t-\tau)\mathrm{d}\tau \tag{3.26}$$

Ultimately the functions for u, $\dot{u}$ and $\ddot{u}$ can be developed:

$$u(t)=-\rho(t)\cdot\sin[\omega_d t-\phi(t)] \tag{3.27}$$

$$\dot{u}(t)=-\omega\rho(t)\cdot\cos[\omega_d t-\phi(t)+\theta] \tag{3.28}$$

$$\ddot{u}(t)=\omega^2\rho(t)\cdot\sin[\omega_d t-\phi(t)+2\theta] \tag{3.29}$$

where $\rho(t)$ is the amplitude of the vibration, $\phi(t)$ the phase-angle and $\theta=\sin^{-1}(\xi)$.

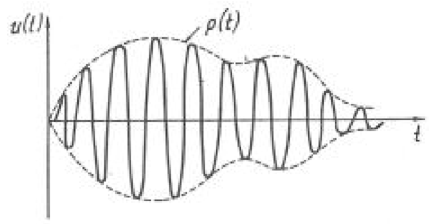


Figure 3.26: *Response of single-degree-of-freedom system (Anastasiadis, 1989).*

The same procedure is conducted for the response spectra, where a range of natural periods are represented.

Considering a number of SDOF-systems with increasing natural periods $T_i=2\pi/\omega_i$ $(i = 1,2,...,n)$ that are clamped to the same base, as seen in Figure 3.27. The base is excited by the seismic input $\ddot{u}_0(t)$ and the k^{th} system with natural period T_k will emerge the relative displacement and velocity $u_k(t)$ and $\dot{u}_k(t)$, as well as the absolute acceleration $\ddot{u}_k(t)$. These values are the time-history response of a certain system. The maximum values, of these three are of interest, and are symbolized $S_d(T_k)$, $S_v(T_k)$ and $S_a(T_k)$ for the k^{th} natural period T_k. In the spectra every structural system has the same damping ratio ξ. The spectral values of $S_d(T_k,\xi)$, $S_v(T_k,\xi)$ and $S_a(T_k,\xi)$ will decrease as the value of ξ is increasing.

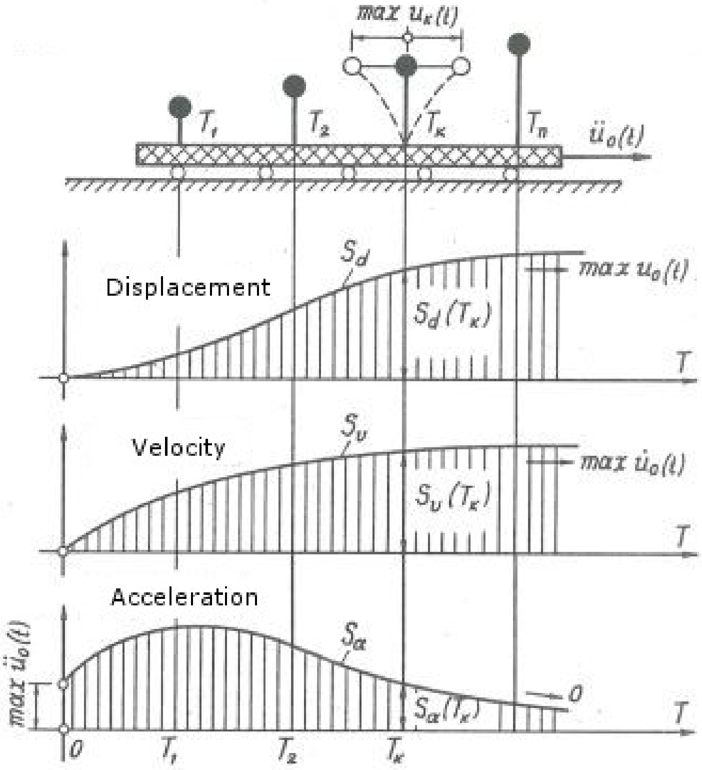


Figure 3.27: *Generation of response spectrum based on seismic excitation (Anastasiadis, 1989).*

The spectral displacement is calculated using Eq. (3.26) with for example arithmetical integration or discrete integration considering linear change of $\ddot{u}(t)$ between two points. By choosing a constant ξ_i and ultimately calculating $u_k(t)$ for T_k subsequently $\max[u_k(t)]$ gives $S_d(T_k)$. This procedure is conducted for a number of T_i in the interval of 0.001 s to 10 s which will give the response spectrum $S_d(T_i, \xi_i)$. From the spectral displacement S_d the maximum elastic force of the structure is possible to calculate as follows:

$$\max[P_s] = k \cdot S_d \tag{3.30}$$

and further the maximum deformation energy:

$$\max[U] = \frac{1}{2} k S_d^2 \tag{3.31}$$

The corresponding kinetic energy T_{kin} is zero, because $\dot{u} = 0$ when u peaks:

$$T_{kin} = \frac{1}{2} m \dot{u}^2 \tag{3.32}$$

Based on S_d the values of S_v and S_a can be calculated as below. This is an approximate calculation and therefore called *pseudo-velocity* (PSV) and *pseudo-acceleration* (PSA).

$$\max|u(t)| = S_d \approx \max[\rho(t)] \tag{3.33}$$

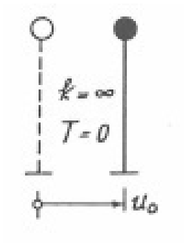
The below relationships are valid when $\xi < 20\%$, which is valid for building structures in general that usually have approximately $\xi \approx 5\%$:

$$\max|\dot{u}(t)| \approx \omega \cdot \max[\rho(t)] \approx \omega S_d = S_v \equiv PSV \tag{3.34}$$

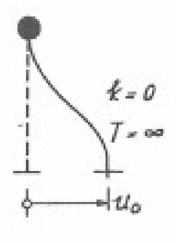
$$\max|\ddot{u}(t)| \approx \omega^2 \cdot \max[\rho(t)] \approx \omega^2 S_d = S_a \equiv PSA \tag{3.35}$$

Practically it is of interest to investigate the limit values of S_d, S_v and S_a for $T_n \to 0$ and $T_n \to \infty$.

In order to understand the properties of the structural system for the two limit states stated above $T_n = 2\pi\sqrt{\frac{m}{k}}$ is analyzed, with the mass of the structure m constant.



(a) *Very stiff system.*



(b) *Very soft system.*

Figure 3.28: *Response spectra limit states (Left: very stiff system (Anastasiadis, 1989).*

$T_n \to 0$ gives that the stiffness $k \to \infty$, i.e. a very stiff system (Figure 3.28a).

$$u(t) = 0 \qquad \dot{u}(t) = 0 \qquad \ddot{u}(t) = \ddot{u}_0(t) \tag{3.36}$$

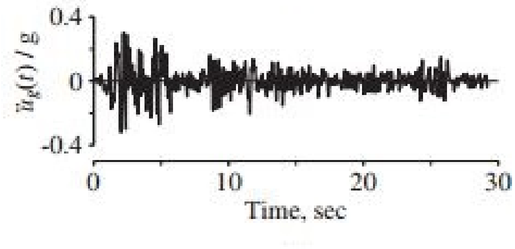
$$S_d(0) = 0 \qquad S_v(0) = 0 \qquad S_a(0) = \max(\ddot{u}_0) \tag{3.37}$$

Vice versa if $T_n \to \infty$ gives stiffness $k \to 0$, i.e. a very soft system (Figure 3.28b).

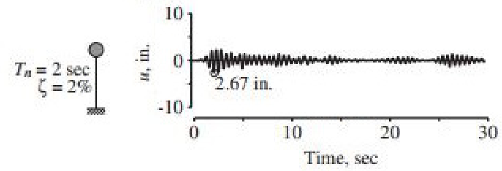
$$u(t) = -u_0(t) \qquad \dot{u}(t) = -\dot{u}_0(t) \qquad \ddot{u}(t) = 0(t) \tag{3.38}$$

$$S_d(\infty) = \max(u_0) \qquad \max(u_0) \cdot S_v(\infty) = \max \dot{u}_0 \qquad S_a(\infty) = 0 \tag{3.39}$$

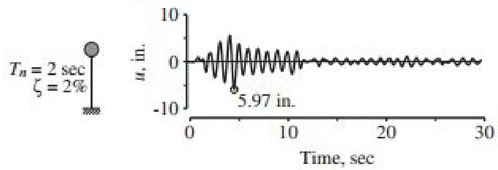
Figure 3.29 shows an arbitrary earthquake time-history acceleration and further on the response of random SDOF-systems with a constant damping ratio. Ultimately the random systems are denoted in the response spectra resulting from the excitation and system responses.



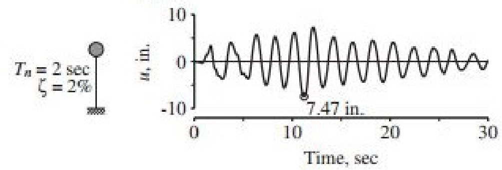
(a) *Input earthquake time-history excitation.*



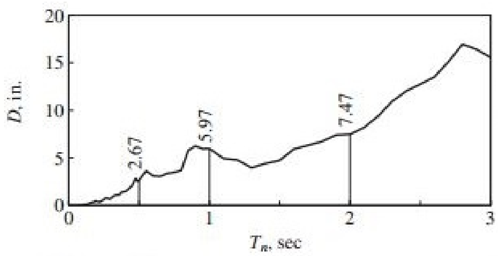
(b) *Deformation time-history response of SDOF-system with* $T_n = 0.5$ s *and* $\xi = 2\%$.



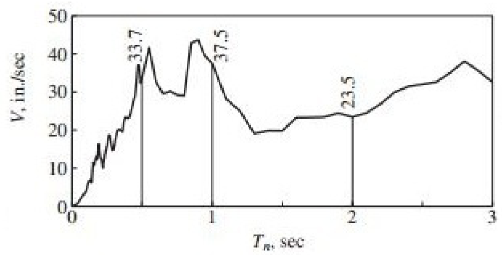
(c) *Deformation time-history response of SDOF-system with* $T_n = 1$ s *and* $\xi = 2\%$.



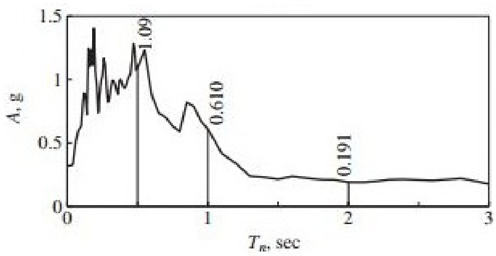
(d) *Deformation time-history response of SDOF-system with* $T_n = 2$ s *and* $\xi = 2\%$.



(e) *Displacement response spectra (*$\xi = 2\%$*).*



(f) *Pseudo-velocity response spectra (*$\xi = 2\%$*).*



(g) *Pseudo-acceleration* S_a *response spectra (*$\xi = 2\%$*).*

Figure 3.29: *Procedure to create response spectrum for certain input excitation [*1 in = 25.4 mm*] (Chopra, 2007).*

When looking at Figure 3.29g it is observed that for increasing T_n the maximum displacement S_d also increases. If looking at a system with the same load cases and dead weights, i.e. constant m, what changes the T_n is the stiffness of the system k. When

k is large the T_n is small and vice versa, as seen in Eq. (3.4). However the maximum pseudo-acceleration S_a decreases with increasing T_n, which ultimately leads to lower seismic forces on the structure, due to the law of inertia $F_s = m \cdot S_a(T_n, \xi)$.

The illustration in Figure 3.30 shows the effect of the damping ratio ξ on the spectral pseudo-acceleration:

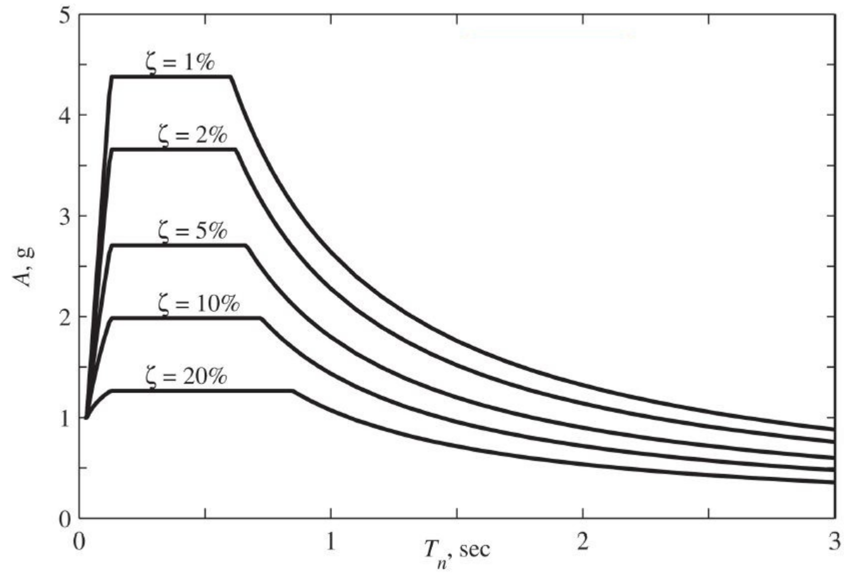


Figure 3.30: *Damping effect on spectral pseudo-acceleration (Stojadinovic, 2013).*

Inelastic response spectrum

According to the building standards, reduction is possible with the behavior factor q. The behavior factor is related to the ductility factor μ and thus the behavior factor q defines the ductility and reduction of intensity when designing a structural system. Newmark-Hall defines the relationship based on the natural period T_n of the structure, see Figure 3.31.

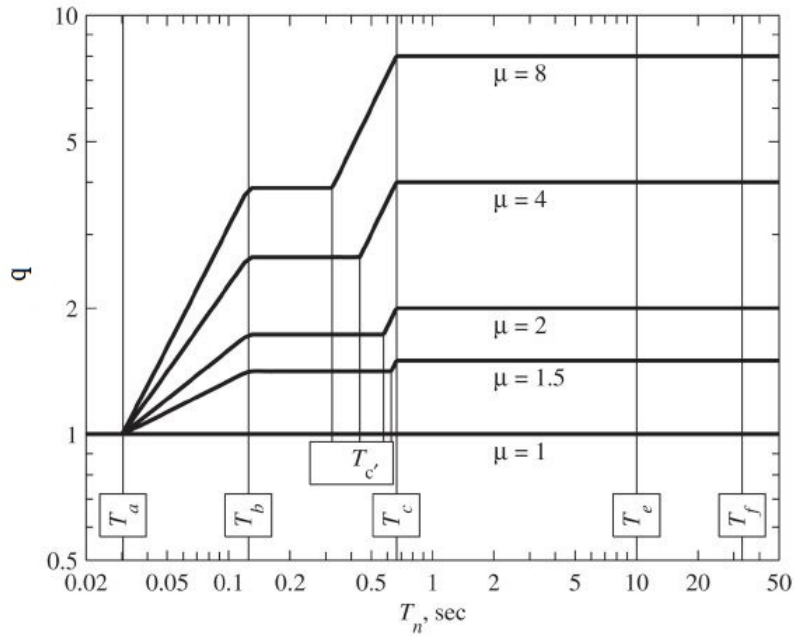


Figure 3.31: *Newmark-Hall relation $q-\mu-T_n$ (Stojadinovic, 2013).*

Three regions, where the behavior factor q and the ductility factor μ is approximately equal, can be distinguished from Figure 3.31.

- For very small natural periods ($T_n < 0.03s$) the maximum absolute accelerations are approximately equal.
- For intermediate natural periods ($0.12s < T_n < 0.5s$) the maximum energies are approximately equal.
- For intermediate natural periods ($T_n > 0.5s$) the maximum relative displacements are approximately equal.

See the following:

$$q = \begin{cases} 1 & \text{if } T_n < T_a \text{ (Acceleration principle)} \\ \sqrt{2\mu - 1} & \text{if } T_b < T_n < T_c' \text{ (Energy principle)} \\ \mu & \text{if } T_n > T_c \text{ (Displacememnt principle)} \end{cases} \tag{3.40}$$

Index p denotes the *elastoplastic* and respectively index e denotes the *elastic* values.

For $T_n < T_a$:

$$S_{a,p} = S_{a,e} \qquad S_{d,p} = \mu S_{d,e} \tag{3.41}$$

In Figure 3.32 the force-displacement diagrams are shown for structural systems. In the first diagram (Figure 3.32a) the equal displacement principle is illustrated. For $T_n > T_c$:

$$\frac{S_{a,p}}{S_{a,e}} = \frac{P_y}{P_e} = \frac{u_y}{u_{max}} = \frac{1}{\mu} \tag{3.42}$$

$$S_{a,p} = \frac{1}{\mu} S_{a,e} \qquad S_{d,p} = S_{d,e} \tag{3.43}$$

In the second diagram (Figure 3.32b) the equal energies principle is illustrated. For $T_b < T_n < T_c'$:

$$\text{Area}(OCC') = \text{Area}(OABB') \tag{3.44}$$

or

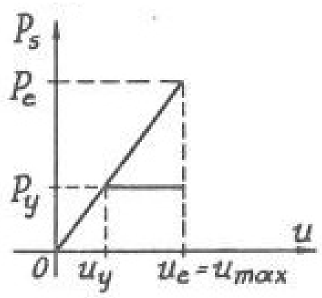
$$\frac{1}{2}(OC'')P_e = \frac{1}{2} u_y P_y + P_y(u_{max} - u_y) \tag{3.45}$$

Since $(OC') = u_y \cdot \dfrac{P_e}{P_y}$ ultimately:

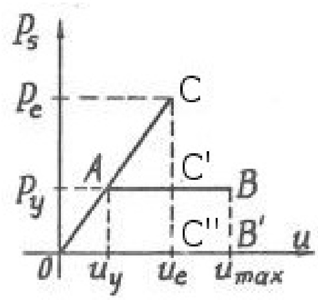
$$\frac{S_{a,p}}{S_{a,e}} = \frac{P_y}{P_e} = \frac{1}{\sqrt{2\mu - 1}} \tag{3.46}$$

$$S_{a,p} = \frac{1}{\sqrt{2\mu-1}} S_{a,e} \qquad S_{d,p} = \frac{\mu}{\sqrt{2\mu-1}} S_{d,e} \tag{3.47}$$

The following can then be observed in the force-displacement diagram of a structural system:



(a) *Displacement principle.*



(b) *Energy principle.*

Figure 3.32: *Correlation between elastic and elastoplastic structural systems (Anastasiadis, 1989).*

From the above empirical relations, the inelastic response spectra can be computed, as seen in Figure 3.33.

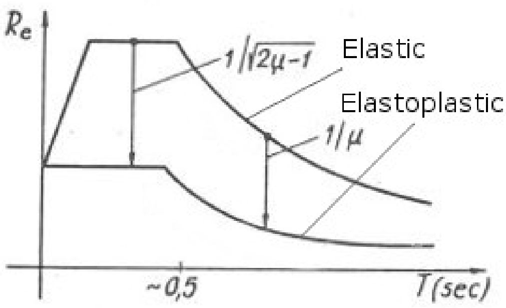


Figure 3.33: *Elastic and inelastic acceleration response spectra (Anastasiadis, 1989).*

4
European Standards

This chapter discusses the key points of *Eurocode 8* (EC8), also referred to as EN-1998-1:2004, which are relevant for detailing of precast structures. In addition, as the case study examines a building structure in Norway, it is important to recognize the applicable standards in the national application annex of Norway (NA:2014).

Up until March 2010 the national code of Norway was in effect. Starting in April 2010, EC8 was adopted and remains the current code along with the national application annex.

The EC8 is comprised of ten sections:

1. General
2. Performance Requirements and Compliance Criteria
3. Ground Conditions and Seismic Action
4. Design of Buildings
5. Specific Rules for Concrete Buildings
6. Specific Rules for Steel Buildings
7. Specific Rules for Steel-Concrete Composite Buildings
8. Specific Rules for Timber Buildings
9. Specific Rules for Masonry Buildings
10. Base Isolation

Sections 1,2,3 and 4, which refer to general seismic design and rules, will be addressed along with section 5, which provides the standards that must me observed for the design of the type of primary seismic bearing systems being analyzed in this thesis.

Apart from the assumptions of *Eurocode 0* (EC0) the assumption is that no change will occur in the lifetime or construction phase of the structure; i.e. no change will occur with the structural properties used to calculate seismic impact. Even if changes are made that increase the structural resistance, they should not be accounted for.

4.1 Performance requirements and compliance criteria

4.1.1 Requirements

One of the fundamental requirements that buildings must meet is the *no-collapse* requirement, which means that the structure is able to retain its structural integrity and a residual load bearing capacity after the seismic event has occurred, which means that both global or local collapse must be prevented. For this requirement the design seismic action has a reference return period of 475 years or 50 years probability to exceed (EN-1998-1:2004).

The second fundamental requirement is the *damage limitation*, which prescribes that the structure must be able to withstand a design seismic action that has a probability to exceed in 10 years and a return period of 95 years without the occurrence of damage and the associated limitations of use. The local conditions in Norway results in an inappreciable seismic action for this return period, therefore this criteria is neglected in analysis and design (NA:2014).

This means that for buildings in Norway only the *ultimate limit state* must be computationally checked for buildings located in Norway. This requirement is associated with the *no-collapse criteria*, because by meeting the requirement, buildings avoid collapse or other types of structural failures, which ultimately endanger the safety of people.

It is possible to differentiate the seismic action depending on the importance of the building, i.e. depending on the usage that it is designed for. *Classification of buildings, importance factor* γ_I assigned to each *importance class* are given in Table 4.1.

Table 4.1: *Classification of buildings (NA:2014).*

Importance class	γ_I	Building
I	0.7	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.
II	1.0	Ordinary buildings, not belonging in the other categories.
III	1.4	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.
IV	2.0	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.

Figure 4.1 shows the importance class of a building in terms of earthquake performance level. The objective of the importance factors in Table 4.1 is to ensure that there are additional safety measures in place in structures of importance to civil protection.

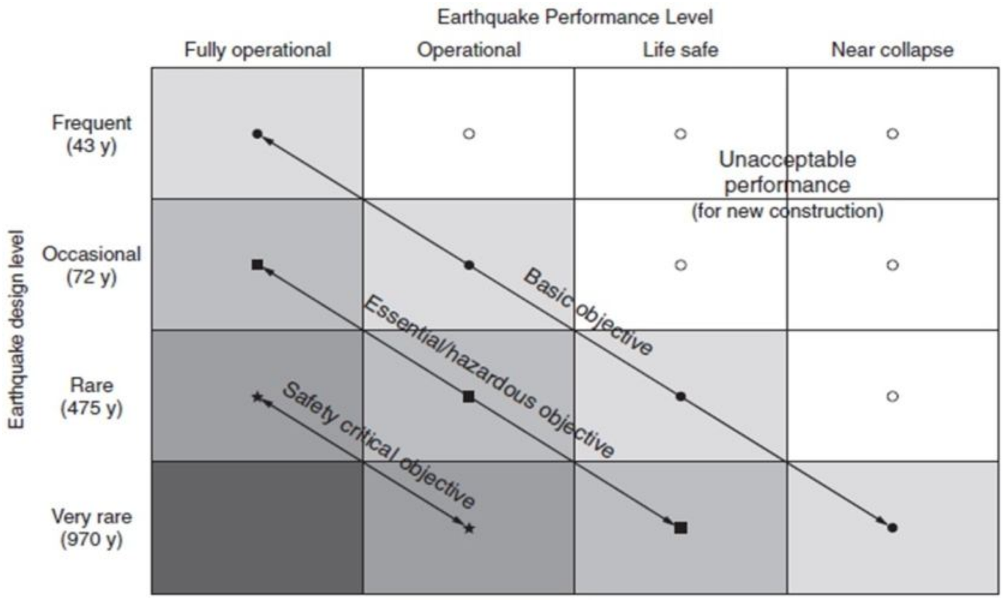
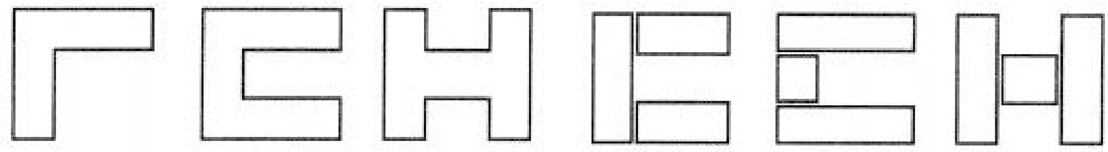


Figure 4.1: *Performance objectives (Ozcep* et al.*, 2011).*

4.1.2 Specific conceptual design measures

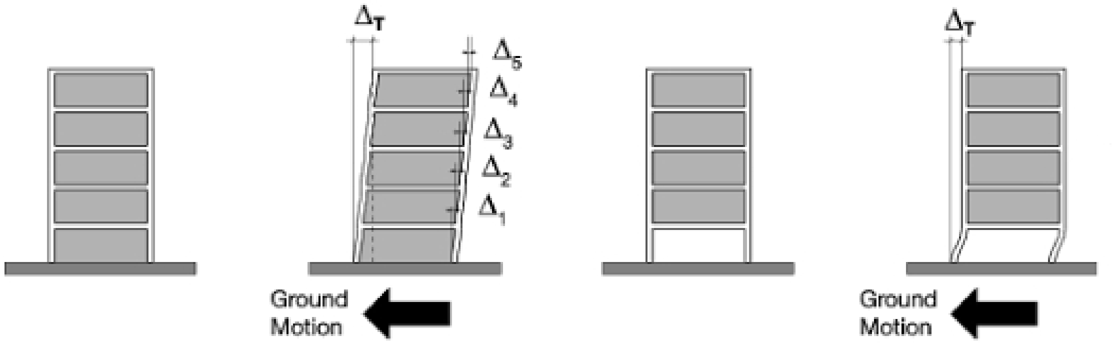
In order to limit uncertainties in the seismic behavior of the structure some specific measures must be taken. In the conceptual design phase *simple and regular* layouts should be chosen in both plan and elevation. An alternative is to divide a building into several regular parts by using *seismic joints*.



(a) *Plan of structures without joints.* **(b)** *Plan of structures with joints.*

Figure 4.2: *Division of structure into regular and simpler plans (Bachmann, 1997).*

When ductile design is the chosen approach, overall *dissipative and ductile* behavior must be ensured. Brittle failure or permanent formation of unstable mechanisms is to be avoided. An irregular layout in elevation can lead to an uneven distribution of stiffness. In a regular building where overall ductile behavior applies, the total displacement will be distributed equally between the stories, which gives small relative displacements that are favorable (Figure 4.3a). In the soft story mechanism, the plastic hinges occur partly in the structure where the stiffness is lower, which leads to larger relative displacements (Figure 4.3b).



(a) *Distributed ductile behaviour in connections.* **(b)** *Soft story mechanism.*

Figure 4.3: *Behaviour of ductile designs (Guevara-Perez, 2012).*

The seismic performance of a structure depends on the behavior of *critical regions*. Therefore, the detailing of these and shall be such as to maintain the capacity to transmit the necessary forces and to dissipate energy under cyclic loading. This means that connections between elements and in regions where non-linear behavior is predicted should receive special care in design.

An adequate model should be used to conduct the analysis, which should take into account the influence of *soil deformability* and of the *non-structural elements* and other aspects, such as the presence of *adjacent structures*. In addition, the *second order effects* of the actions should be taken into account.

With regards to building foundations, the stiffness should be adequate for transmitting the forces from the superstructure to the ground in a uniform manner. In general, only one foundation type should be used for the same structure, unless the structure contain dynamically independent units.

4.2 Ground conditions and seismic action

4.2.1 Ground conditions

In order to classify the *ground type* under the foundation appropriate investigations must be conducted. The construction site should be free from risks of *ground rupture*, *slope instability* and *permanent settlements* caused by liquefaction or densification in the event of an earthquake.

The influence of the condition of the local ground on seismic excitation is reflected in ground type categories. The investigation will result in proper *natural period thresholds* in the design spectrum and also the *amplification of the ground acceleration*. In Table 4.2 the ground types are described by the stratigraphic profiles.

Table 4.2: *Ground types (EN-1998-1:2004).*

Ground type	Description of stratigraphic profile	$v_{s,30}$ [m/s]	N_{SPT} [blows/30cm]	c_u [kPa]
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800 -		
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of meters in thickness, characterized by a gradual increase of mechanical properties with depth.	360−800	> 50	> 250
C	Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters.	180˘360	15−50	70−250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	120−180	10−15	30−70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s> 800 m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index ($PI > 40$) and high water content.	< 100 (indicative)		10−20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1.			

Table 4.3: *Values of parameters describing the elastic response spectrum (NA:2014).*

Ground type	S	T_B [s]	T_C [s]	T_D [s]
A	1.0	0.10	0.20	1.7
B	1.3	0.10	0.25	1.5
C	1.4	0.10	0.30	1.5
D	1.55	0.15	0.40	1.6
E	1.65	0.10	0.30	1.4

4.2.2 Seismic zones

In the Norwegian national annex, national territories are divided into *seismic zones* depending on the seismic hazard. In each zone the hazard is assumed to be constant. The hazard is described by the single parameter a_{g40Hz}, which is the value of the peak acceleration of the bedrock for the return period of 475 years in Norway. In Figure 4.4 the seismic zones are marked with isocurves. If the structure is located in between two isocurves, the value should be interpolated; if it is placed at the maximum-regions (*H*) a constant of 0.05 $\mathrm{m/s^2}$ is added to a_{g40Hz}. These values may only be used for buildings at the main land. In the event, the building is located on *Svalbard*, *Jan Mayen*, *Bjørnøya* or at the continental shelf, the peak accelerations are defined with other methods (NA:2014).

The reference ground acceleration a_{gR} is given by the relation:

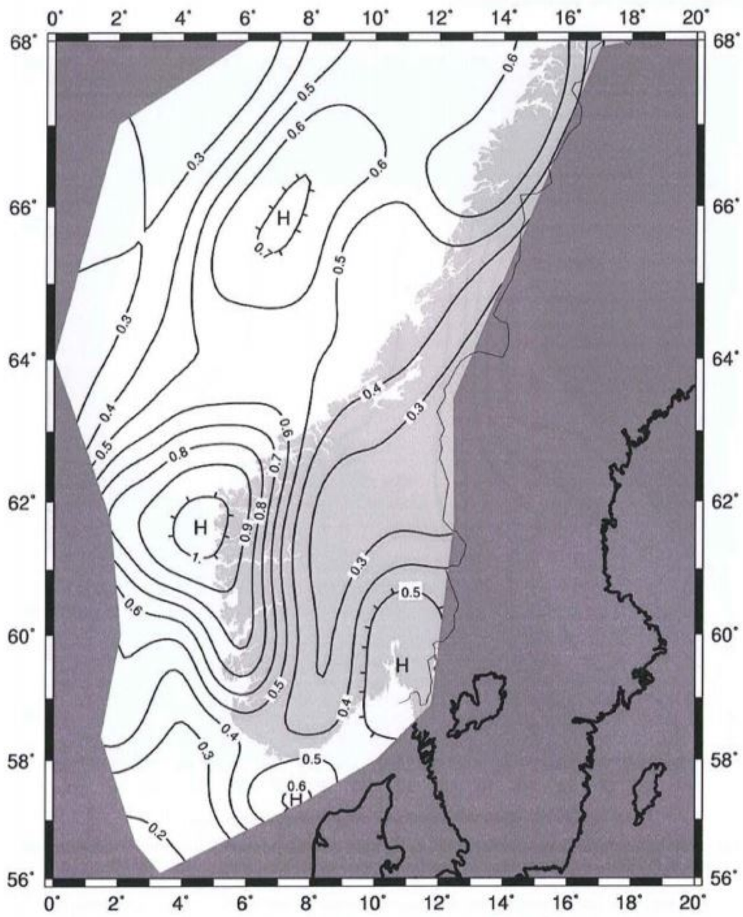
$$a_{gR} = 0.8 \cdot a_{g40Hz} \tag{4.1}$$

The importance of the structure is accounted for by modifying the peak ground acceleration a_{gR} by multiplying with the importance factor γ_I. The classification of buildings and corresponding importance factor listed in Table 4.1.

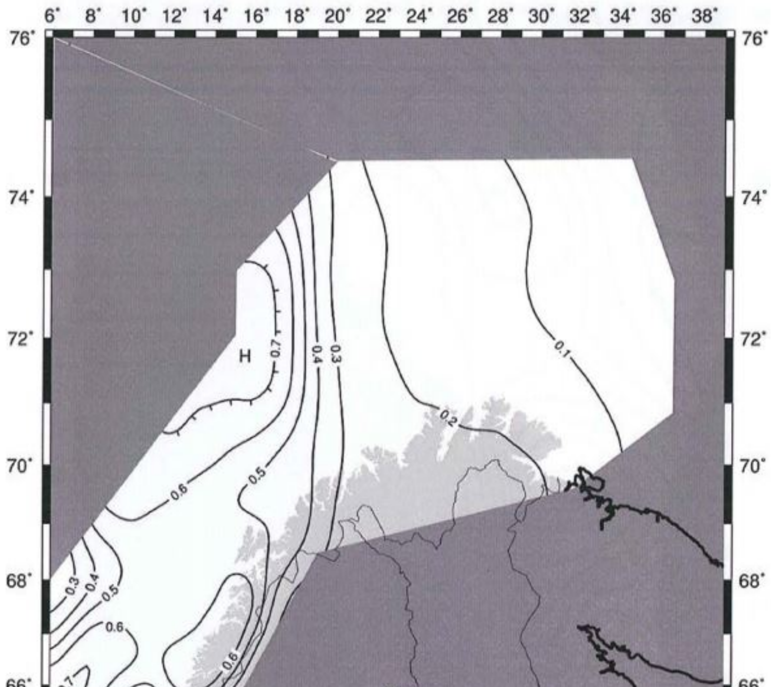
$$a_g = a_{gR} \cdot \gamma_I \tag{4.2}$$

where a_g is the design ground acceleration.

In the case of *very low seismicity* the rules of EC8 can be ignored. In Norway, is when $a_g S < 0.05$ g, where S is the amplification factor of the ground (Table 4.3), if $a_g S < 0.1$ g this is considered as *low seismicity* and the behavior factor q is maximum 1.5 . For cases where $a_g S \geq 0.1$ g, higher ductility classes can be used for design, i.e. the behavior factor q can assume values larger than 1.5 . In Tables 5.1 – 5.4 in the next chapter, the design acceleration is calculated with respect to location, ground type and importance of the building structure. The case of *non-low seismicity design acceleration* are highlighted, i.e. the cases where DCL or DCM design can be selected.



(a) *Southern Norway.*



(b) *Northern Norway.*

Figure 4.4: *Seismic zones in Norway,* a_{g40Hz} *[m/s^2] (NA:2014).*

4.2.3 Elastic response spectrum

The earthquake motion at a given point is represented by an elastic ground acceleration spectrum called *elastic response spectrum.*

The values of the natural periods T_B, T_C, T_D, T_E and soil amplification factor S describe the shape of the elastic response spectrum depending on the ground type, as shown in Figure 4.5. The figure is based on the equations in Table 4.4.

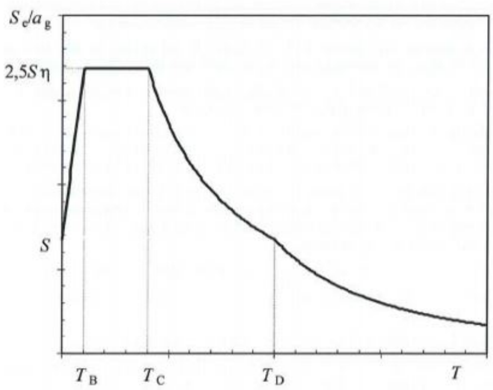
Table 4.4: *Elastic response spectrum (EN-1998-1:2004).*

Condition	Elastic response
$0 \leq T_n \leq T_B$	$S_e(T_n) = a_g \cdot S \cdot \left[1 + \frac{T_n}{T_B} \cdot (\eta \cdot 2.5 - 1)\right]$
$T_B \leq T_n \leq T_C$	$S_e(T_n) = a_g \cdot S \cdot \eta \cdot 2.5$
$T_C \leq T_n \leq T_D$	$S_e(T_n) = a_g \cdot S \cdot \eta \cdot 2.5 \left[\frac{T_C}{T_n}\right]$
$T_D \leq T_n \leq 4s$	$S_e(T_n) = a_g \cdot S \cdot \eta \cdot 2.5 \left[\frac{T_C T_D}{T_n^2}\right]$

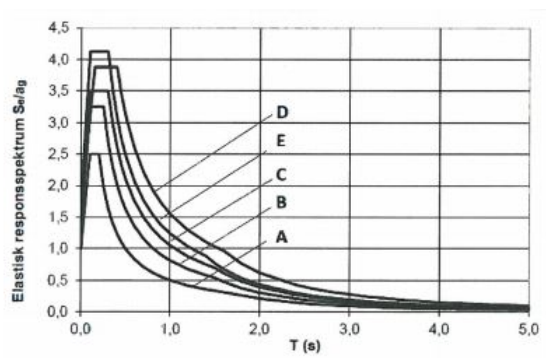
In general the damping ratio of the structure is $\xi = 5\,\%$, but in the event, another value is used for a certain structure the correction factor η may be used.

$$\eta = \sqrt{\frac{10}{5 + \xi}} \geq 0.55 \tag{4.3}$$

where ξ is the viscous damping ratio of the structure, expressed as a percentage.



(a) *Shape of the elastic response spectrum.*



(b) *Elastic response spectrum for ground types A-E.*

Figure 4.5: *Elastic response spectrum (NA:2014).*

4.2.4 Design response spectrum

The capacity of structural systems to resist seismic actions in the non-linear range generally permits their design for resistance to seismic forces smaller than those corresponding to a linear response. To avoid explicit inelastic structural analysis in design, the capacity of the structure to dissipate energy, through mainly ductile behavior of its elements and other mechanisms, is taken into account by performing an elastic analysis based on a response spectrum reduced with respect to the elastic one, henceforth called a *design spectrum.* This reduction is made by introducing the behavior factor q.

The behavior factor is an assumption of the ratio of the seismic forces that the structure would experience if its response was totally elastic using a conventional elastic analysis model. The behavior factor is given in relevant parts of EC8 depending on the material and the structural system. This value may also differ in both horizontal directions, although the classification shall be the same in all directions. The shape of the design response spectrum are given by the equations in Table 4.5. The shape are similar to the elastic response spectrum (Figure 4.5) with the difference that the elastic response $S_e(T_n)$ are divided by the behavior factor q, which gives the design response $S_d(T_n)$.

Table 4.5: *Design response spectrum (EN-1998-1:2004).*

Condition	Design response
$0 \le T_n \le T_B$	$S_d(T_n) = a_g \cdot S \cdot \left[\frac{2}{3} + \frac{T_n}{T_B} \cdot \left(\frac{2.5}{q} - \frac{2}{3}\right)\right]$
$T_B \le T_n \le T_C$	$S_d(T_n) = a_g \cdot S \cdot \frac{2.5}{q}$
$T_C \le T_n \le T_D$	$S_d(T_n) = a_g \cdot S \cdot \frac{2.5}{q} \cdot \left[\frac{T_C}{T_n}\right] \ge \beta \cdot a_g$
$T_D \le T_n \le 4s$	$S_d(T_n) = a_g \cdot S \cdot \frac{2.5}{q} \cdot \left[\frac{T_C T_D}{T_n^2}\right] \ge \beta \cdot a_g$

where $\beta = 0.2$ and is the lower bound factor for the design spectrum (NA:2014).

It should be noted that these formulas are not sufficient for design in the case of structural systems with base-isolation or special energy-dissipating systems.

4.2.5 Combination of the seismic action with other actions

The design value E_d is to be determined in accordance with EC0 and the inertial effects of the seismic action are to be accounted for by the presence of *the masses associated with all gravity loads* appearing in the following combination of actions:

$$\sum G_{k,j} \text{"}+\text{"} \sum \psi_{E,i} \cdot Q_{k,i} \tag{4.4}$$

where $G_{k,j}$ is the permanent loads, $Q_{k,i}$ is the variable loads and $\psi_{E,i}$ is the combination coefficient for variable action i. These take into account the likelihood of the variable loads $Q_{k,i}$ not being present over the entire structure during an earthquake event. They may also account for the reduced participation of masses in the motion of the structure due to non-rigid connections between them. The value of $\psi_{E,i}$ is calculated as shown in Eq. (4.5) with constants $\psi_{2,i}$ and ϕ taken from Tables 4.6 and 4.7.

$$\psi_{E,i} = \phi \cdot \psi_{2,i} \tag{4.5}$$

Table 4.6: *Values of ϕ used for the calculation of $\psi_{E,i}$ (NA:2014).*

Type of variable action	Story	ϕ
Categories A-C	Roof	1.0
Categories A-C	Stories with correlated occupancies	1.0
Categories A-C	Independently occupied stories	1.0
Categories D-F and Archives		1.0

Table 4.7: *Values of ψ factors for buildings (NA:2014).*

Action	ψ_0	ψ_1	ψ_2
Category A: domestic, residential areas	0.7	0.5	0.3
Category B: office areas	0.7	0.5	0.3
Category C: congregation areas	0.7	0.7	0.6
Category D: shopping areas	0.7	0.7	0.6
Category E: storage areas	1.0	0.9	0.8
Category F: traffic area, vehicle weight $\leq 30kN$	0.7	0.7	0.6
Category G: traffic area, $30kN <$ vehicle weight $\leq 160kN$	0.7	0.5	0.3
Category H: roofs	0	0	0
Snow loads on buildings	0.7	0.5	0.2
Wind loads on buildings	0.6	0.2	0
Temperature (non-fire) in buildings	0.6	0.5	0

4.3 Design of buildings

4.3.1 Basic principles of conceptual design

When designing for seismic actions, the following guiding principles should be taken into consideration for the conceptual design:

- Structural simplicity
- Uniformity, symmetry and redundancy
- Bi-directional resistance and stiffness
- Torsional resistance and stiffness
- Diaphragmatic behavior at story level
- Adequate foundation

Structural simplicity is characterized by clear paths of the transmission of the forces between structural members. Modeling, analysis, dimensioning, detailing and construction of simple structures are less uncertain and therefore a more reliable prediction of seismic effect on the structure can be made.

Uniformity in plan is when the structural elements are evenly distributed in plane as well as in elevation. This allows short and direct transmission of inertia forces created in the distributed masses of the building. The building may be divided in uniform parts with seismic joints that are dynamically independent.

Uniformity in plan is important because it tends to eliminate the sensitive zones where large ductility demands can occur and might cause collapse.

Designs aim for a close relationship between the mass-distribution and the distribution of resistance and stiffness, i.e. mass and stiffness-centers.

The use of evenly distributed structural elements increases redundancy and allows a more favorable redistribution of action effects and widespread energy dissipation across the entire structure.

Horizontal seismic excitation is *bi-directional* meaning the building should be able to resist actions in any direction. To satisfy this elements should be arranged in an orthogonal manner and give similar resistance and stiffness characteristics in both main horizontal directions. The choice of the stiffness characteristics of the structure should also limit the development of excessive displacements that might result in instabilities due to second order effects or excessive damages.

In order to reach adequate torsional resistance, the main structural elements resisting the seismic forces should be placed as close to the periphery of the building as possible. This is to avoid non-uniform stress development in elements that torsion tends to cause.

The floor system is important for transmission of seismic actions to the vertical bearing system. This means that the floor structure ensures that the horizontal and vertical systems act together. This is especially important when the layout of the vertical bearing system is non-uniform, e.g. in dual or mixed systems.

The floor systems, as well as roof systems, shall therefore provide adequate in-plane stiffness and resistance together with an effective connection to the vertical structural systems.

A certain number of the structural members may be designed as *secondary seismic members.* This means that they do not form part of the seismic resisting system of the structure. These members should be ignored in terms of strength and stiffness against seismic actions. However, they must be designed and detailed in order to maintain support for gravity loads when subject to the displacement caused by the most unfavorable seismic design conditions.

The total contribution to lateral stiffness of all secondary seismic members should not exceed 15% of that of all primary seismic members.

4.3.2 Structural regularity

Depending on the structure's layout plan and elevation various simplifications are allowed. These are compiled in Table 4.8.

Table 4.8: *Consequences of structural regularity on seismic analysis and design.*

Regularity			Allowed simplification		Behaviour factor*
Example	Plan	Elevation	Model	Linear-elastic analysis	(linear analysis)
Fig. 4.6a	Yes	Yes	Planar	Lateral force	Reference value
Fig. 4.6b	Yes	No	Planar	Modal	Decreased value
Fig. 4.6c	No	Yes	Spatial	Lateral force	Reference value
Fig. 4.6d	No	No	Spatial	Modal	Decreased value

* The decreased behavior factor is determined by the reference values multiplied by 0.8.

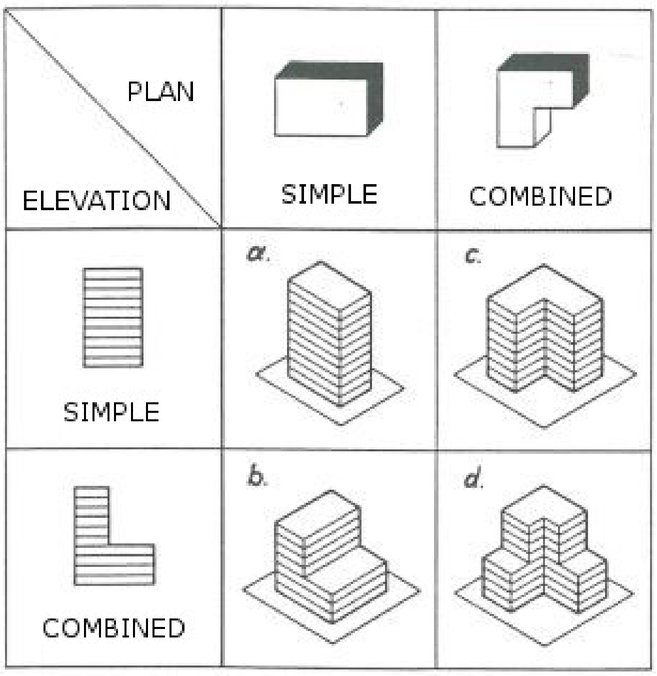


Figure 4.6: *Illustration of structural layouts (Anastasiadis, 1989).*

In order for the building to be considered regular in plan, the following expressions shall be fulfilled:

$$\lambda = \frac{L_{max}}{L_{min}} \leq 4 \tag{4.6}$$

$$e_{0x} \geq 0.30 \cdot r_x \tag{4.7}$$

$$r_x \leq l_s \tag{4.8}$$

where λ is the slenderness, L_{max} and L_{min} is the maximum respectively minimum length of the building, e_{0x} is the structural eccentricity, r_x is the torsional radius, i.e. distance between the stiffness and mass centers, and l_s is the radius of gyration of the floor mass in plan.

In Figure 4.7 the criteria for regularity in elevation are summarized.

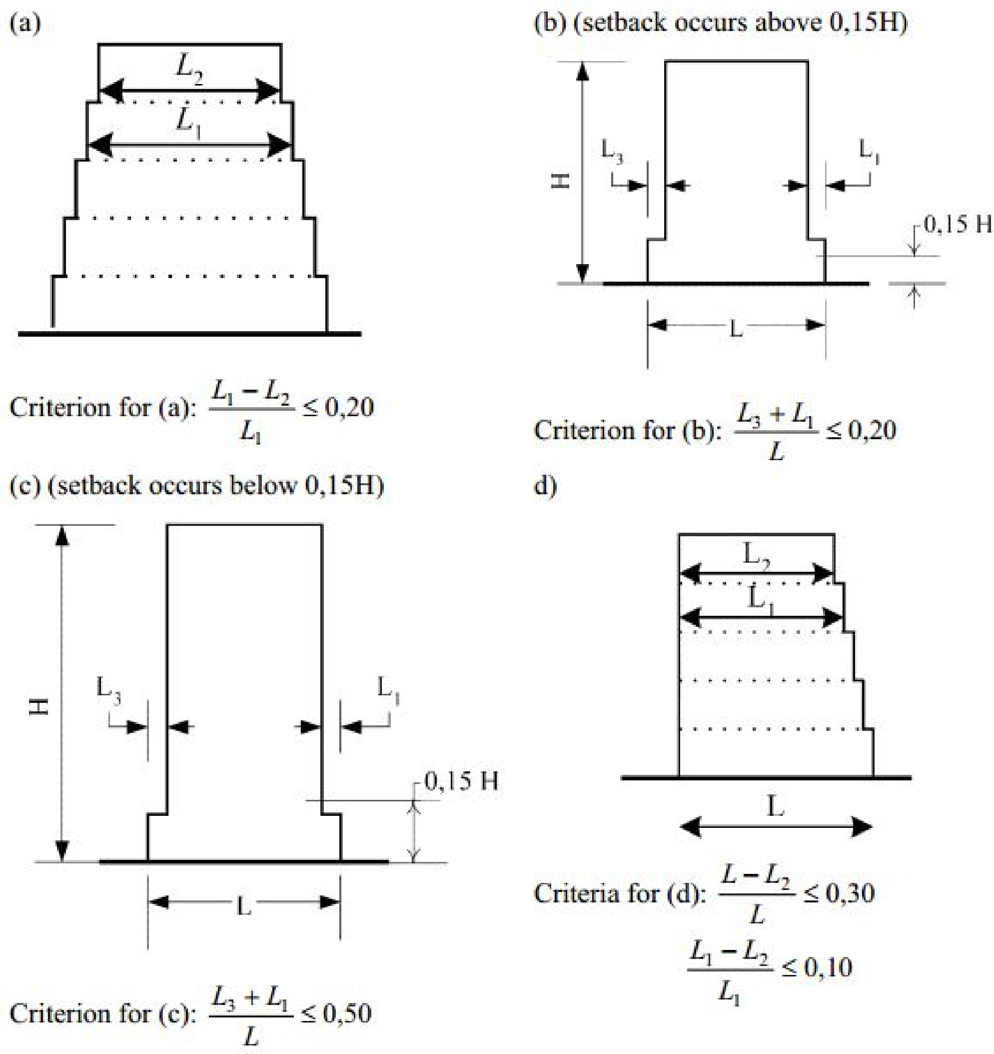


Figure 4.7: *Criteria for regularity if setbacks in elevation (EN-1998-1:2004).*

4.4 Concrete structures

For the design of precast buildings like the one in this case study, *section 5 - Specific Rules for Concrete Buildings* (EN-1998-1:2004) is of importance. Due to the layout of the structural system, the concrete walls will act to resist the seismic impact because the steel columns in the system are hinged and therefore, are important when it comes to resistance against vertical loads, i.e. dead weight, live loads etc.

4.4.1 Energy dissipation capacity and ductility classes

Design of the concrete elements shall provide an adequate capacity to dissipate energy without substitutional reduction of its overall resistance against horizontal and vertical loading. This is also mentioned in the previous section for general targets for seismic design according to EC8.

EC8 prescribes three different *ductility classes* that could be applied for design and detailing:

- Ductility class low (DCL)
- Ductility class medium (DCM)
- Ductility class high (DCH)

Concrete structures can be designed for low dissipation capacity and low ductility, termed as ductility class low (DCL). This occurs by applying the design rules of Eurocode 2 (EC2) to the seismic design, while also adding the few additions of chapter 5.3 in EC8 to the EC2 regulations to fulfill the criteria. EC8 prescribes this design for *low seismicity cases*, meaning reduced or simplified design procedures may be used for certain types or categories of structures.

In other cases the structure shall provide energy dissipation and overall ductile behavior. This is ensured by designing a large volume of the structure to be ductile. This could be classified as ductility class medium (DCM) or ductility class high (DCH), depending on the level of energy dissipation capacity. The design should ultimately provide stable mechanisms under repeated reversed loading without suffering brittle failure.

4.4.2 Structural types and behavior factors

The classifications of concrete structures are listed below. Systems are differentiated according to their behavior when subject to horizontal actions. Depending on the characteristics of the structure, classification categories are as follows:

- Frame system
- Dual system (frame or wall equivalent)
- Ductile wall system (coupled or uncoupled)
- System of large lightly reinforced walls
- Inverted pendulum system
- Torsionally flexible system

In order to account for and design a structure with ductile behavior the behavior factor q is used. The selection of q depends on the *type of structural system* and requested *ductility class.* The behavior factor is calculated according to Eq. (4.9), based on the values in Table 4.9.

$$q = q_o \cdot k_w \geq 1.5 \tag{4.9}$$

The behavior factor q shall be derived for every direction and k_w is reflecting the prevailing failure mode in structural systems with walls. The factor of k_w is calculated as follows in Eq. (4.10)

$$k_w = \begin{cases} 1.00 & \text{for frame and frame-equvalent dual systems} \\ (1+\alpha_0)/3 \leq 1 & \text{but not less than 0.5, for wall, wall-equivalent} \\ & \text{and torsionally flexible systems} \end{cases} \tag{4.10}$$

α_0 is the prevailing aspect ratio of the walls of the structural system. If the aspect ratios $h_{w,i}/l_{w,i}$ of all walls i of a structural system do not significantly differ, this ratio may be calculated using Eq. (4.11):

$$\alpha_0 = \frac{\sum h_{w,i}}{\sum l_{w,i}} \tag{4.11}$$

where $h_{w,i}$ is the height of wall i and $l_{w,i}$ is the length of the section of wall i.

Table 4.9: *Basic values of the behavior factor q_o, (EN-1998-1:2004).*

Structural type	DCM
Frame system, dual system, coupled wall system	$3.0\alpha_u/\alpha_1$
Uncoupled wall system	3.0
Torsionally flexible system	2.0
Inverted pendulum system	1.5

If the building is non-regular in elevation q_o is reduced by 20 %. The values that may be assumed for α_u/α_1 are be further defined in EC8.

4.4.3 Design criteria

Adequate design requires that overall ductile behavior is achieved and that the potential plastic hinge formation is defined. Those areas should possess high plastic rotational capacities.

Generally structural redundancy is important for the structure. If lower static indeterminacy is in question, lower behavior factors should be assigned.

Unless more precise data is available the curvature ductility factor μ_ϕ should be defined as in Eq. (4.12):

$$\mu_\phi = \begin{cases} 2q_o - 1 & \text{if } T_1 \geq T_C \\ 1 + 2(q_o - 1)\dfrac{T_C}{T_1} & \text{if } T_1 < T_C \end{cases} \tag{4.12}$$

where T_1 is the fundamental natural period.

In critical regions of primary seismic elements with longitudinal reinforcement of steel class B the curvature ductility factor μ_ϕ from Eq. (4.12) can be multiplied by at least 1.5.

4.4.4 Design for DCM

Only ribbed reinforcement bars are allowed, with the exception of closed stirrups and cross-ties. In critical regions reinforcing steel class B or C shall be used.

Geometrical constraints

For ductile wall design the thickness of the web b_{wo} should satisfy Eq. (4.13).

$$b_{wo} \geq \max(0.15, h_s/20) \tag{4.13}$$

where h_s is the clear story height in meters.

The thickness b_w of the confined parts of the wall should not be less than 200 mm. If the length of the confined part does not exceed $2b_w$ and $0.2l_w$ then b_w should not be less than $h_s/15$. Otherwise if the length of the confined part exceeds $2b_w$ and $0.2l_w$ then b_w should not be less than $h_s/10$.

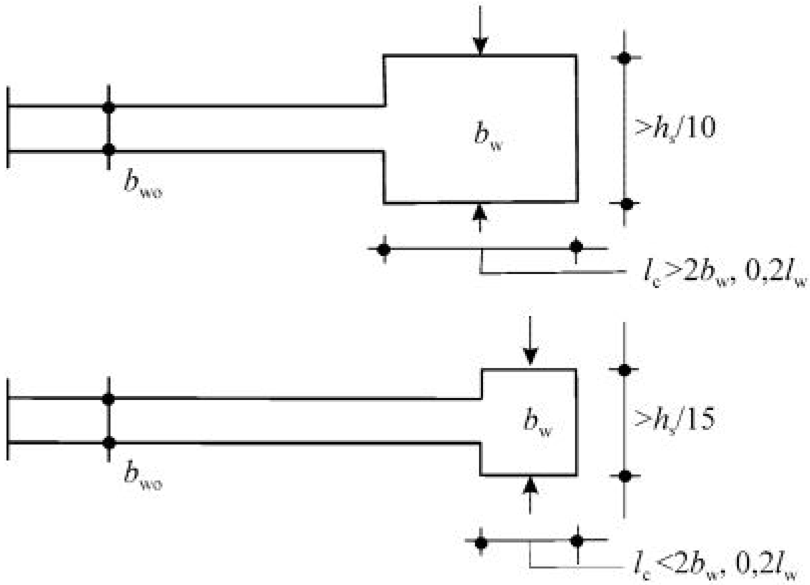


Figure 4.8: *Minimum thickness of confined boundary elements (EN-1998-1:2004).*

Provisions for ductile walls

Redistribution of the effects of seismic action between primary seismic walls of up to 30 % is allowed, without reducing the total resistance demand. Shear forces should be redistributed along with the bending moments so as to not affect the ratio between bending moments and shear forces in the individual walls.

Uncertainties of the bending moment distribution along the height of slender walls shall be covered. These uncertainties are covered by following the prescriptions in the standard as following:

In Figure 4.9 the bending moment is vertically displaced based on the values from analysis given along the height of the wall. The envelope may be assumed linear if the structure has continuous distribution of mass, stiffness or resistance over the height. The tension shift should be consistent with the strut inclination taken in ultimate limit state (ULS) and with the floor acting as ties.

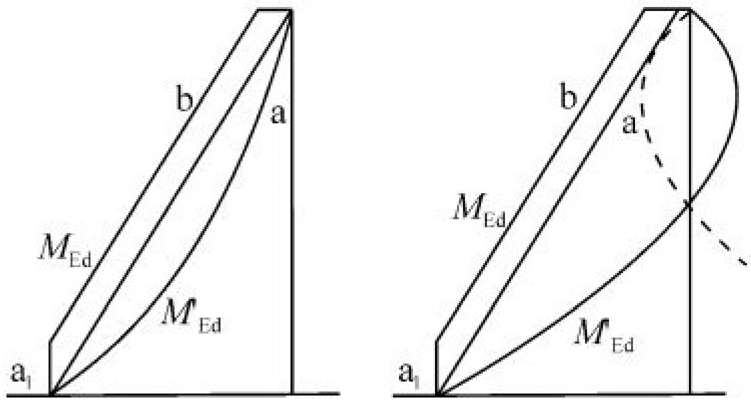


Figure 4.9: *Design envelope for bending moments in slender walls.* a *is the moment diagram from analysis,* b *is the design envelope and* a_1 *is the tension shift. (left: wall systems; right: dual systems) (EN-1998-1:2004).*

The uncertainties in higher modes in dual systems containing slender walls is accounted for by using the envelope of shear forces shown in Figure 4.10.

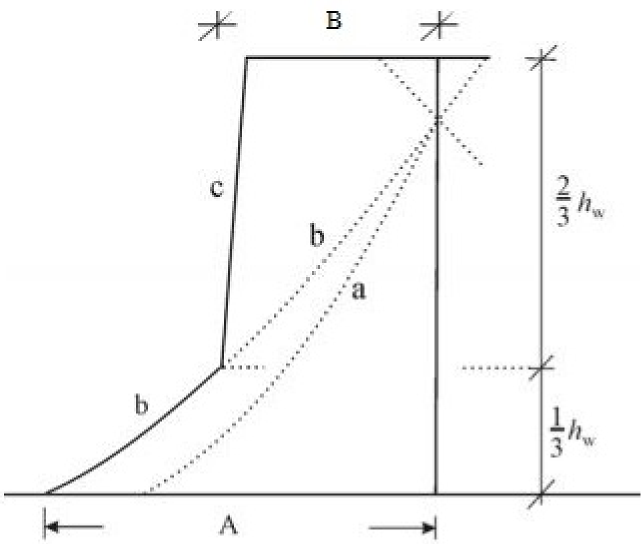


Figure 4.10: *Design envelope of the shear forces in the walls of a dual system.* a *is the shear diagram from analysis,* b *is the magnified shear diagram,* c *is the design envelope,* A: $V_{wall,base}$ *and* B: $V_{wall,top} \geq V_{wall,base}/2$ *(EN-1998-1:2004).*

Detailing for local ductility

Flexural and shear resistance are to be calculated according to EC2 using the axial forces from the seismic analysis. In primary seismic walls the value of the normalized axial force should satisfy Eq. (4.14):

$$\nu_d = \frac{N_{Ed}}{A_c f_{cd}} \leq 0.4 \tag{4.14}$$

The height of the critical region h_{cr} of the wall is estimated as follows:

$$h_{cr} = \max(l_w, h_w/6) \tag{4.15}$$

but

$$h_{cr} = \begin{cases} 2 \cdot l_w \\ \begin{cases} h_s & \text{for } n \leq 6 \text{ storeys} \\ 2 \cdot h_s & \text{for } n \geq 7 \text{ storeys} \end{cases} \end{cases} \tag{4.16}$$

where h_s is the clear story height.

At the critical regions the curvature ductility factor μ_ϕ is calculated according to Eq. (4.12) on page 60. The behavior factor q_o should be replaced by following:

$$q_o^* = q_o \cdot \frac{M_{Ed}}{M_{Rd}} \tag{4.17}$$

where the ratio $\frac{M_{Ed}}{M_{Rd}}$ is taken from the base of the wall in the seismic design situation. M_{Ed} is the design bending moment from the analysis and M_{Rd} is the design flexural resistance.

For walls with a rectangular cross-section, the mechanical volumetric ratio of the required confining reinforcement ω_{wd} in boundary elements should satisfy the relationship in Eq. (4.18):

$$\alpha\omega_{wd} \geq 30\mu_\phi(\nu_d + \omega_v)\varepsilon_{sy,d}\frac{b_c}{b_o} - 0.035 \tag{4.18}$$

where $\omega_v = \frac{\rho_v f_{yd,v}}{f_{cd}}$, which is the mechanical ratio of vertical reinforcement.

The confinement should be extended vertically over the critical height h_{cr} and horizontally along the length l_c. If no more precise data is available, the compressive strain at which spalling is expected may be taken as $\varepsilon_{cu2} = 0.0035$. The confined boundary element should be limited to a distance of $x_u(1 - \frac{\varepsilon_{cu2}}{\varepsilon_{cu2,c}})$ from the hoop centerline near the extreme compression fiber, with the depth of the confined compression zone x_u at ultimate curvature estimated from $x_u = (\nu_d + \omega_v)\frac{l_w b_c}{b_o}$ and the ultimate strain $\varepsilon_{cu2,c}$ of confined concrete estimated as $\varepsilon_{cu2,c} = 0.0035 + 0.1\alpha\omega_{wd}$.

The length l_c of the confined boundary element should not be taken smaller than $0.15l_w$ or $1.5b_w$. Further on the minimum value of ω_{wd} within the critical region should be 0.08.

The hoops should be placed in order to engage the vertical reinforcement. The hoops should at least be 6 mm in diameter and the spacing must protect the vertical bars from buckling. Vertical reinforcement bars confined by the hoops may have a maximum spacing of 200 mm. The spacing of the hoops shall not exceed the relationship in Eq. (4.19):

$$s_w \le \min(8d_{bL}, b_o/2, 175 \text{ mm}) \tag{4.19}$$

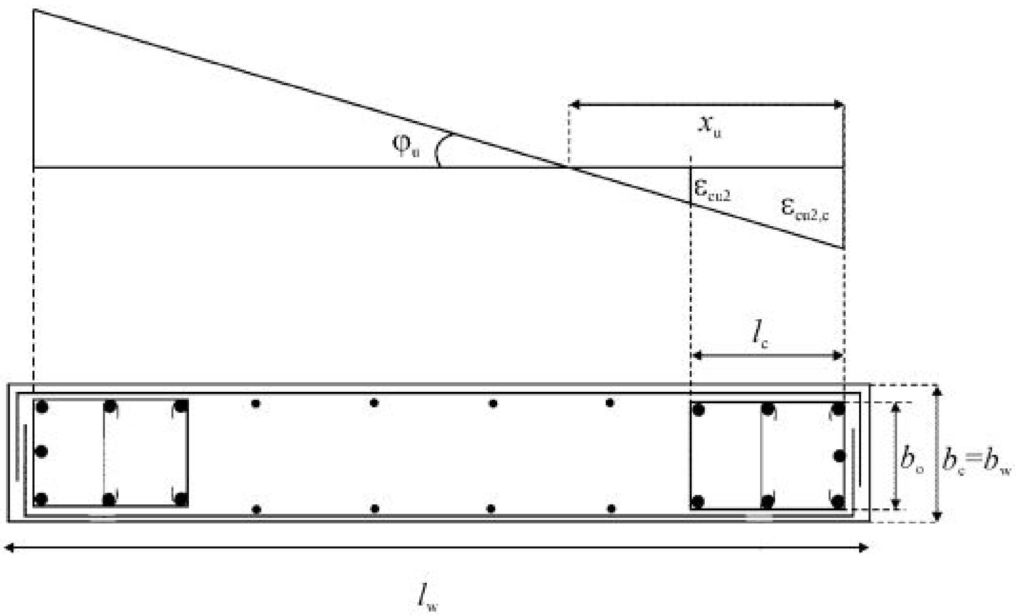


Figure 4.11: *Confined boundary element of free-edge wall end (EN-1998-1:2004).*

In the height over the critical regions relevant prescriptions in EC2 should be applied, when it comes to vertical, horizontal and transverse reinforcement. In the parts where the compressive strain, during the seismic design situation, exceeds $\varepsilon > 0.002$ the minimum vertical reinforcement ratio of 0.005 should be provided.

Regarding the transverse reinforcement of the boundary elements can be determined according to regulations in EC2 if either the value of normalized design axial force is $\nu_d \le 0.15$ or $\nu_d \le 0.20$ and the behavior factor q is reduced by 15 %.

4.4.5 Precast concrete structures

The behavior factor q_p for precast structures is calculated as following:

$$q_p = q \cdot k_p \tag{4.20}$$

k_p is further defined in relevant sections of EC8.

When modeling a precast structure it's important to evaluate the different roles of the structural system, i.e. vertical bearing system, horizontal bearing components etc.

Connections of precast elements

Connections of elements considered to be apart from the critical regions are located at least the length of the largest cross-section dimension from the critical region.

Overdesigned connections should be derived on the basis of overstrength flexural resistances at the end sections of the critical regions equal to $\gamma_{Rd} \cdot M_{Rd}$. The factor γ_{Rd} is equal to 1.20 for DCM-design. Terminating reinforcement must be fully anchored before the end sections of the critical region. In addition the reinforcement in the critical region should be fully anchored outside the overdesigned connection.

Energy dissipating connections should conform local ductility criteria or by performing tests on structure specimens that shows stable cyclic deformation of the connection.

Precast large-panel walls

A minimum confinement of concrete near the edge should be provided for all panels over a square section of side length b_w, where b_w denotes the thickness of the panel. The part of the wall between a vertical joint arranged closer than $2.5b_w$ should be detailed according to local ductility.

Horizontal joints where the entire edge is under compression can be designed without *shear keys*. If it's partly compression and shear on the edge shear keys should be provided along the entire edge. The total tensile force resulting from the axial action-effects must be taken by vertical reinforcement placed along the tensile area of the panel and fully anchored in the body of the upper and lower panels. Within the horizontal joint ductile welding secures the continuity of the reinforcement bars, alternatively special keys for this purpose can be used, see Figure 4.12. Shear resistance verification along the part under compression should be conducted in the horizontal connection which are partly in compression and partly in tension. In this case the value of the axial force N_{Ed} should be replaced by the value of total compressive force acting on the compression area F_c.

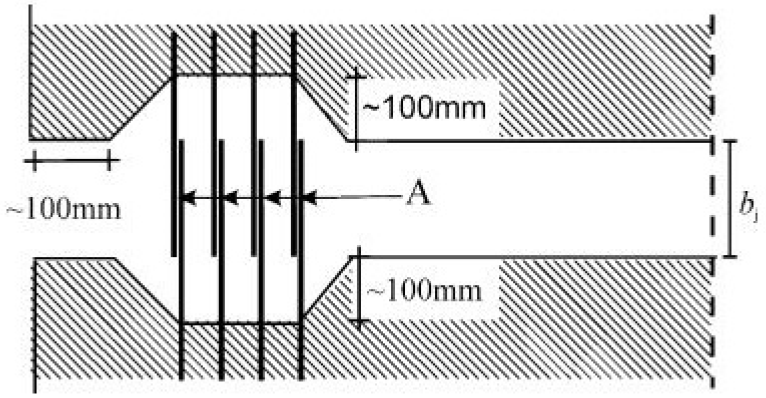


Figure 4.12: *Tensile reinforcement possibly needed at the edge of walls.* A*: lap-welding of bars (EN-1998-1:2004).*

To fulfill local ductility along vertical connections minimum reinforcement across the connection should be equal to 0.10 % (when fully compressed) and equal to 0.25 % (when partly compressed and partly in tension). In order to avoid abrupt post-peak force response softening the reinforcement ratio should not be larger than 2 %. In DCM-design the reinforcement may be concentrated in the top-, middle- and bottom-band. The reinforcement bars in the vertical connections should be anchored in the form of loops or by welding across the connection. To secure continuity along the connection after cracking, longitudinal reinforcement of minimum ratio $\rho_{c,min} = 1$ % should be provided in the grout filling space of the connections, see Figure 4.13.

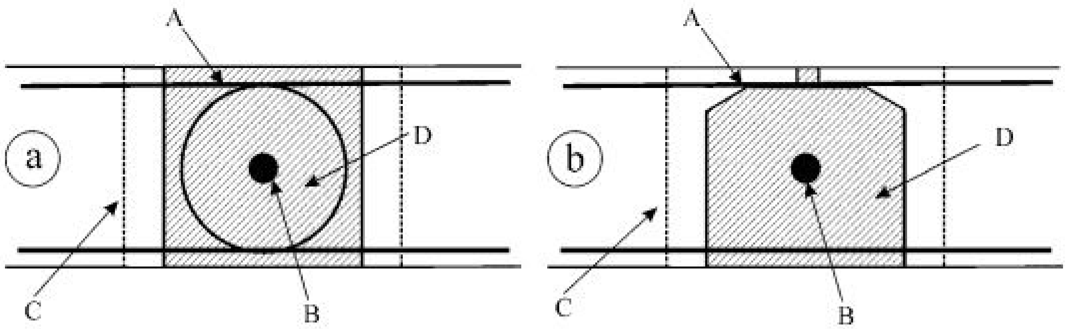


Figure 4.13: *Cross-section of vertical connections between precast large-panels.* A*: reinforcement protruding across connection,* B*: reinforcement along connection,* C*: shear keys,* D*: grout filling space between panels (EN-1998-1:2004).*

Diaphragms

In order to obtain rigid diaphragm behavior joints should be placed only over supports. Topping of in-situ reinforced concrete can improve the rigidity of the diaphragm. The topping layer should not be less than 40 mm if the span between the supports is less than 8 m, or less than 50 mm for longer spans.

The mesh reinforcement in the topping layer shall be connected to the vertical resisting elements above and below. Tensile forces should be resisted by steel ties along the perimeter of the diaphragm, as well along some joints of the precast slab elements. If cast in-situ topping is used this additional reinforcement should be located in this topping. The target of the ties is to create a continuous system of reinforcement along and across the diaphragm and should also be connected to each lateral force resisting element.

In-plane acting shear forces along connections should be calculated using an overstrength factor of 1.30. Primary seismic elements over and under the diaphragm should be satisfactory connected to the diaphragm. Any horizontal joints should always be properly reinforced and friction forced should not be relied upon.

4.5 Essential parts of Eurocode 2

4.5.1 Calculation of actions

In section 6 of the EC2 necessary information about the calculation procedure of internal actions can be found (EN-1992-1-1:2005).

4.5.2 Material

In section 3 Table 3.1 of the EC2 material properties of concrete necessary for design is found (EN-1992-1-1:2005).

4.5.3 Maximum and minimum reinforcement

Vertical reinforcement

Minimum reinforcement is recommended to:

$$A_{v,min} = 0.002 \cdot A_c \tag{4.21}$$

Maximum reinforcement is recommended to:

$$A_{v,max} = 0.04 \cdot A_c \tag{4.22}$$

where A_c is the cross-sectional area where the reinforcement is placed.

If the minimum reinforcement content is governing, the reinforcement bars should be divided and placed equally at the wall surfaces.
The spacing between the vertical bars should not be larger than 3 times the wall thickness and 400 mm.

Horizontal reinforcement

Minimum reinforcement is recommended to:

$$A_{h,min} = \max(0.25 \cdot A_v, 0.001 \cdot A_c) \tag{4.23}$$

where A_c is the cross-sectional area where the reinforcement is placed and A_v is the area of the total amount of the vertical reinforcement in the specific region.

The spacing between the horizontal bars should not be larger than 400 mm.

4.6 Summary of requirements for design

Tables 4.10, 4.11, 4.12 and 4.13 summarize the most important criteria for the detailing requirements of a wall according to DCL- and DCM-design.

Table 4.10: *Summary of detailing requirements: Geometrical (EN-1998-1:2004 & EN-1992-1-1:2005).*

Parameter			DCM	DCL
Web thickness	b_{wo}	$\geq$	max(150 mm, $h_s/20$)	-
Critical region length	h_{cr}	$\leq$	max(l_w, $h_w/6$)	-
	h_{cr}	$\leq$	min($2l_w$, h_s) if wall $\leq$ 6 stories	-
	h_{cr}	$\leq$	min($2l_w$, $2h_s$) if wall > 6 stories	-

Table 4.11: *Summary of detailing requirements: Boundary elements (EN-1998-1:2004 & EN-1992-1-1:2005).*

	Parameter			DCM	DCL
In critical region	Critical length	l_c	$\geq$	$\max(0.15 l_w, 1.5 b_w)$	-
				length over which $\varepsilon > 0.0035$	
	Thickness	b_w	$\geq$	$\max(200 \text{ m}, h_s/15)$	-
				if $l_c \leq \max(2b_w, l_w/5)$	
	Thickness	b_w	$\geq$	$\max(200 \text{ m}, h_s/10)$	
				if $l_c > \max(2b_w, l_w/5)$	
	Vertical reinforcement				
	Minimum	$\rho_{v,min}$	$=$	0.5 %	0.2 %
	Maximum	$\rho_{v,max}$	$=$	4 %	4 %
	Confining hoops				
	Diameter	d_{bw}	$\geq$	6 mm	-
	Spacing	s_w	$\leq$	$\min(8d_{bL}, b_o/2, 175 \text{ mm})$	-
	Volumetric ratio	ω_{wd}	$\geq$	0.08	-
		$\alpha\omega_{wd}$	$\geq$	$30\mu_\phi(\nu_d + \omega_v)\varepsilon_{sy,d}\frac{b_c}{b_o} - 0.035$	-

Table 4.12: *Summary of detailing requirements: Boundary elements (EN-1998-1:2004 & EN-1992-1-1:2005).*

	Parameter			DCM & DCL
In the rest of the wall	**Vertical reinforcement**			
	Minimum	$\rho_{v,min}$	$=$	0.5 % in parts where $\varepsilon > 0.2$ % elsewhere 0.2 %
	Confining hoops			
	Diameter	d_{bw}	$\geq$	$\max(6 \text{ mm}, d_{bL}/4)$
	Spacing	s_w	$\leq$	$\min(12d_{bL}, 0.6b_{wo}, 240 \text{ mm})$ up to a distance of $4b_w$ from the critical region
	Spacing	s_w	$\leq$	$\min(20d_{bL}, b_{wo}, 400 \text{ mm})$ otherwise

Table 4.13: *Summary of detailing requirements: Web (EN-1998-1:2004 & EN-1992-1-1:2005).*

Parameter			DCM	DCL
Vertical reinforcement				
Minimum	$\rho_{v,min}$	$=$	0.5 % in parts where $\varepsilon > 0.2$ % elsewhere 0.2 %	0.2 %
Maximum	$\rho_{v,max}$	$=$	4 %	4 %
Horizontal reinforcement				
Diameter	$\rho_{h,min}$	$=$	$\max(0.1\ \%, 0.25\rho_v)$	$\max(0.1\ \%, 0.25\rho_v)$
Spacing	s_h	$\leq$	400 mm	400 mm

5
Computation of the DCL- and DCM-designs

Commercial software is used in order to analyze the structure. *Robot Structural Analysis Professional 2014* (RSA2014) is used to conduct modal analysis and to perform the calculation of actions in the model.

The output from RSA2014 is used in order to design and detail the structural components of relevance.

5.1 Overview of sections

Design ground acceleration
First, the design acceleration range relevant for the analysis is defined, based on the prescribed seismic zones and properties in the standards.

Modal Analysis
Second a model of the study case is created and load cases are defined. For this study, the lateral seismic force is set as a changing parameter and is based on the values extracted from eight load combinations in the previous section with constant vertical load and variable horizontal load being used for the analysis.

Analysis results

Presentation of the modal analysis results for the two designs (DCL and DCM) and respectively the eight load combinations.

Design

In this subsection the calculation procedure for the detailing is presented with two examples of DCL-design and DCM-design, respectively.

Economical assessment and comparison

The calculation procedure in the previous section is then applied for every core wall in the shaft for all load combinations defined earlier. The costs are then compared and plotted in order to obtain the cost development in the defined range of design ground accelerations.

5.2 Design ground acceleration

In the tables that follow, the design ground acceleration for different territories is calculated, depending on the seismic zone and ground type. Tables 5.1 – 5.4 indicate the seismic ground acceleration for importance factors I — IV respectively. The highlighted values are the ones where the choice of DCL or DCM design is possible, according to the standards.

Table 5.1: *Design ground acceleration, $a_g \cdot S$, for buildings in importance class I ($\gamma_I = 0.7$). Highlighted acceleration values indicates non-low seismicity cases.*

			Ground type				
		$\gamma_I = 0.7$	A	B	C	D	E
a_{g40Hz} [m/s²]*	a_{g40Hz} [g]**	a_{gR} [g]***	$a_g \cdot S$ [g]****				
0.1	0.010	0.008	0.01	0.01	0.01	0.01	0.01
0.15	0.015	0.012	0.01	0.01	0.01	0.01	0.01
0.2	0.020	0.016	0.01	0.01	0.02	0.02	0.02
0.25	0.025	0.020	0.01	0.02	0.02	0.02	0.02
0.3	0.031	0.024	0.02	0.02	0.02	0.03	0.03
0.35	0.036	0.029	0.02	0.03	0.03	0.03	0.03
0.4	0.041	0.033	0.02	0.03	0.03	0.04	0.04
0.45	0.046	0.037	0.03	0.03	0.04	0.04	0.04
0.5	0.051	0.041	0.03	0.04	0.04	0.04	0.05
0.55	0.056	0.045	0.03	0.04	0.04	0.05	0.05
0.6	0.061	0.049	0.03	0.04	0.05	0.05	0.06
0.65	0.066	0.053	0.04	0.05	0.05	0.06	0.06
0.7	0.071	0.057	0.04	0.05	0.06	0.06	0.07
0.75	0.076	0.061	0.04	0.06	0.06	0.07	0.07
0.8	0.082	0.065	0.05	0.06	0.06	0.07	0.08
0.85	0.087	0.069	0.05	0.06	0.07	0.08	0.08
0.9	0.092	0.073	0.05	0.07	0.07	0.08	0.08
0.95	0.097	0.077	0.05	0.07	0.08	0.08	0.09
1	0.102	0.082	0.06	0.07	0.08	0.09	0.09
1.05	0.107	0.086	0.06	0.08	0.08	0.09	0.10

* Values are extracted from the map of the seismic zones (Figure 4.4 on page 50)

** Unit conversion into gravitational acceleration of the first column.

*** Computed according to Eq. (4.1) on page 49.

**** Computed according to Eq. (4.2) with the amplification factors (S) of ground types A-E defined in Table 4.3 on page 48.

Table 5.2: *Design ground acceleration, $a_g \cdot S$, for buildings in importance class II ($\gamma_I = 1.0$). Highlighted acceleration values indicates non-low seismicity cases.*

		$\gamma_I = 1.0$	Ground type A	B	C	D	E
a_{g40Hz} [m/s^2]*	a_{g40Hz} [g]**	a_{gR} [g]***	$a_g \cdot S$ [g]****				
0.1	0.010	0.008	0.01	0.01	0.01	0.01	0.01
0.15	0.015	0.012	0.01	0.02	0.02	0.02	0.02
0.2	0.020	0.016	0.02	0.02	0.02	0.03	0.03
0.25	0.025	0.020	0.02	0.03	0.03	0.03	0.03
0.3	0.031	0.024	0.02	0.03	0.03	0.04	0.04
0.35	0.036	0.029	0.03	0.04	0.04	0.04	0.05
0.4	0.041	0.033	0.03	0.04	0.05	0.05	0.05
0.45	0.046	0.037	0.04	0.05	0.05	0.06	0.06
0.5	0.051	0.041	0.04	0.05	0.06	0.06	0.07
0.55	0.056	0.045	0.04	0.06	0.06	0.07	0.07
0.6	0.061	0.049	0.05	0.06	0.07	0.08	0.08
0.65	0.066	0.053	0.05	0.07	0.07	0.08	0.09
0.7	0.071	0.057	0.06	0.07	0.08	0.09	0.09
0.75	0.076	0.061	0.06	0.08	0.09	0.09	0.10
0.8	0.082	0.065	0.07	0.08	0.09	0.10	0.11
0.85	0.087	0.069	0.07	0.09	0.10	0.11	0.11
0.9	0.092	0.073	0.07	0.10	0.10	0.11	0.12
0.95	0.097	0.077	0.08	0.10	0.11	0.12	0.13
1	0.102	0.082	0.08	0.11	0.11	0.13	0.13
1.05	0.107	0.086	0.09	0.11	0.12	0.13	0.14

* Values are extracted from the map of the seismic zones (Figure 4.4 on page 50)

** Unit conversion into gravitational acceleration of the first column.

*** Computed according to Eq. (4.1) on page 49.

**** Computed according to Eq. 4.2 with the amplification factors (S) of ground types A-E defined in Table 4.3 on page 48.

Table 5.3: *Design ground acceleration, $a_g \cdot S$, for buildings in importance class III ($\gamma_I = 1.4$). Highlighted acceleration values indicates non-low seismicity cases.*

			Ground type				
		$\gamma_I = 1.4$	A	B	C	D	E
a_{g40Hz} [m/s^2]*	a_{g40Hz} [g]**	a_{gR} [g]***	$a_g \cdot S$ [g]****				
0.1	0.010	0.008	0.01	0.01	0.02	0.02	0.02
0.15	0.015	0.012	0.02	0.02	0.02	0.03	0.03
0.2	0.020	0.016	0.02	0.03	0.03	0.04	0.04
0.25	0.025	0.020	0.03	0.04	0.04	0.04	0.05
0.3	0.031	0.024	0.03	0.04	0.05	0.05	0.06
0.35	0.036	0.029	0.04	0.05	0.06	0.06	0.07
0.4	0.041	0.033	0.05	0.06	0.06	0.07	0.08
0.45	0.046	0.037	0.05	0.07	0.07	0.08	0.08
0.5	0.051	0.041	0.06	0.07	0.08	0.09	0.09
0.55	0.056	0.045	0.06	0.08	0.09	0.10	0.10
0.6	0.061	0.049	0.07	0.09	0.10	0.11	0.11
0.65	0.066	0.053	0.07	0.10	0.10	0.12	0.12
0.7	0.071	0.057	0.08	0.10	0.11	0.12	0.13
0.75	0.076	0.061	0.09	0.11	0.12	0.13	0.14
0.8	0.082	0.065	0.09	0.12	0.13	0.14	0.15
0.85	0.087	0.069	0.10	0.13	0.14	0.15	0.16
0.9	0.092	0.073	0.10	0.13	0.14	0.16	0.17
0.95	0.097	0.077	0.11	0.14	0.15	0.17	0.18
1	0.102	0.082	0.11	0.15	0.16	0.18	0.19
1.05	0.107	0.086	0.12	0.16	0.17	0.19	0.20

* Values are extracted from the map of the seismic zones (Figure 4.4 on page 50)

** Unit conversion into gravitational acceleration of the first column.

*** Computed according to Eq. (4.1) on page 49.

**** Computed according to Eq. (4.2) with the amplification factors (S) of ground types A-E defined in Table 4.3 on page 48.

Table 5.4: *Design ground acceleration, $a_g \cdot S$, for buildings in importance class IV ($\gamma_I = 2.0$). Highlighted acceleration values indicates non-low seismicity cases.*

		$\gamma_I = 2.0$	Ground type A	B	C	D	E
a_{g40Hz} [m/s^2]*	a_{g40Hz} [g]**	a_{gR} [g]***	$a_g \cdot S$ [g]****				
0.1	0.010	0.008	0.02	0.02	0.02	0.03	0.03
0.15	0.015	0.012	0.02	0.03	0.03	0.04	0.04
0.2	0.020	0.016	0.03	0.04	0.05	0.05	0.05
0.25	0.025	0.020	0.04	0.05	0.06	0.06	0.07
0.3	0.031	0.024	0.05	0.06	0.07	0.08	0.08
0.35	0.036	0.029	0.06	0.07	0.08	0.09	0.09
0.4	0.041	0.033	0.07	0.08	0.09	0.10	0.11
0.45	0.046	0.037	0.07	0.10	0.10	0.11	0.12
0.5	0.051	0.041	0.08	0.11	0.11	0.13	0.13
0.55	0.056	0.045	0.09	0.12	0.13	0.14	0.15
0.6	0.061	0.049	0.10	0.13	0.14	0.15	0.16
0.65	0.066	0.053	0.11	0.14	0.15	0.16	0.17
0.7	0.071	0.057	0.11	0.15	0.16	0.18	0.19
0.75	0.076	0.061	0.12	0.16	0.17	0.19	0.20
0.8	0.082	0.065	0.13	0.17	0.18	0.20	0.22
0.85	0.087	0.069	0.14	0.18	0.19	0.21	0.23
0.9	0.092	0.073	0.15	0.19	0.21	0.23	0.24
0.95	0.097	0.077	0.15	0.20	0.22	0.24	0.26
1	0.102	0.082	0.16	0.21	0.23	0.25	0.27
1.05	0.107	0.086	0.17	0.22	0.24	0.27	0.28

* Values are extracted from the map of the seismic zones (Figure 4.4 on page 50)

** Unit conversion into gravitational acceleration of the first column.

*** Computed according to Eq. (4.1) on page 49.

**** Computed according to Eq. (4.2) with the amplification factors (S) of ground types A-E defined in Table 4.3 on page 48.

5.3 Modal analysis

Modal analysis is computed in order to find the dynamic response of the structure.

5.3.1 Model

The arbitrary structure is modeled in RSA2014 for the case study, as shown in Figure 5.1. The structure is regular in elevation, but irregular in plan. In total the building contains 7 stories where the height of each story is 3.2 m.

Linear releases are introduced in the connections between the wall elements in order to simulate independent precast wall segments. In Figure 5.2 the linear releases are marked with dotted lines.

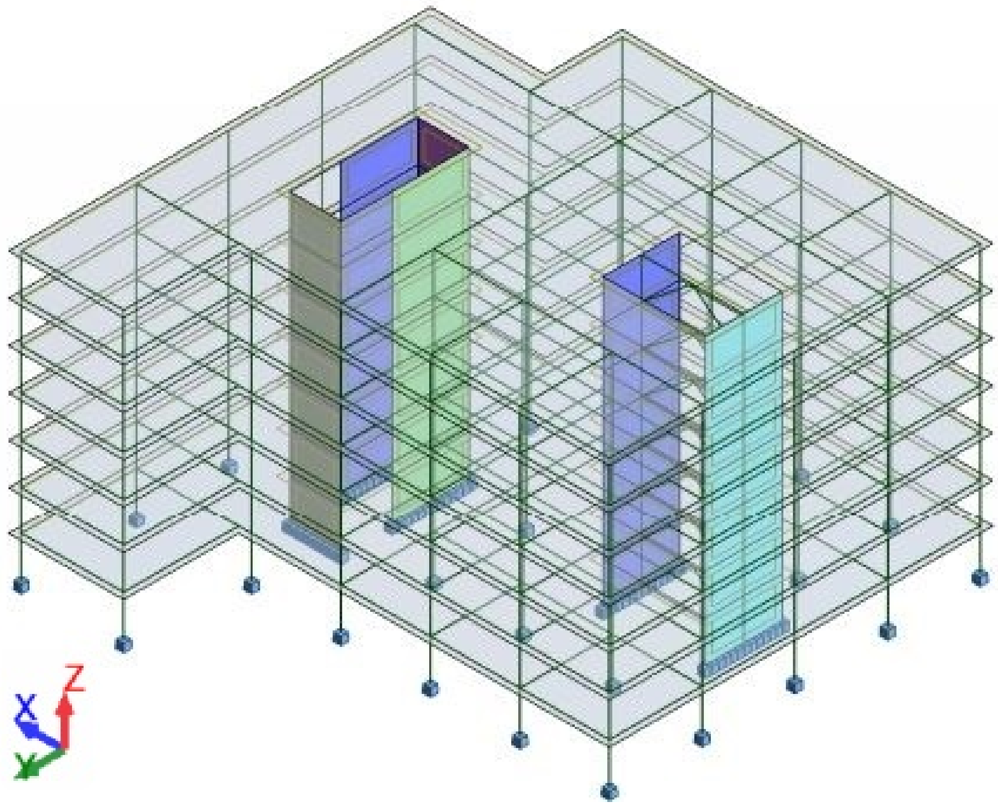


Figure 5.1: *3D-view of the structural model.*

For the case study the lateral bearing system is of importance and in this case the shaft, shown in Figure 5.2, is further examined. The shaft contains four core walls (CW3, CW4, CW5 and CW6) which are precast concrete walls with a thickness of 200 mm. The vertical columns are modeled to resist mainly the vertical loads, i.e. secondary seismic members, and therefore their boundaries are modeled as pinned-pinned connections.

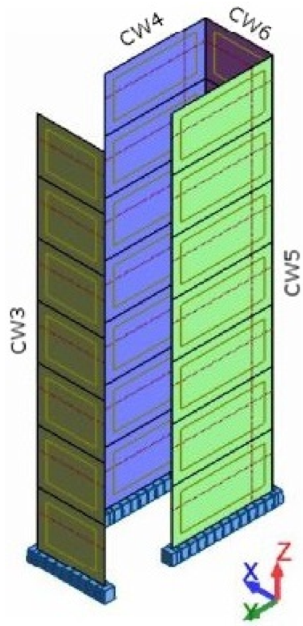


Figure 5.2: *Core walls (linear releases marked with dotted lines).*

5.3.2 Actions

Theory states that the lateral seismic vibration is related to the masses the structure contains. In the modal analysis, these masses are extracted from the vertical loads on each story's level. Table 5.5 lists the actions that are converted into masses for the seismic analysis.

Table 5.5: *Vertical static loads acting on the structure.*

Type	Index	Elements	Distributed force [kN/m^2]
Self weight	DL1	All structural components	N/A
Dead load	DL2	All floor levels	2.00
Dead load	DL2	Roof	2.00
Snow load	SN1	Roof	2.50
Live load	LL2	All floor levels	5.00

For the analysis the actions are combined with the quadratic combination method (CQC), which means the following:

$$G_{load} + \psi_{2i} Q_{load} + \psi_{2i} S_{load} \pm E_{load} \tag{5.1}$$

where G_{load} is the gravity load, Q_{load} is the live load, S_{load} is the snow load, E_{load} is the seismic load. $\psi_{2Q} = 0.3$ (Category A — B) and $\psi_{2S} = 0.2$ according to Table 4.7

on page 54. The seismic load assumes both negative and positive signs in the load combination in order to determine the governing load combination.

The inertial effects of the design seismic action are evaluated by taking into account the presence of the masses associated with all gravity loads appearing in the combination of actions that follow. In order to account for the mass of the building Eq. (5.2) is used for the analysis:

$$m_j = \sum G_{kj} + \sum \psi_{Ei} \cdot Q_{ki} \tag{5.2}$$

Computation of ψ_{Ei} is found in previous chapter, see Eq. (4.5) on page 53.

Four cases for each design (DCL and DCM) are defined (Case 1, Case 2, Case 3 and Case 4) with ranging design ground acceleration. Table 5.6 lists the input values used in RSA2014 in order to define the design spectra (Figure 5.3) that RSA2014 uses to compute the structural response. Note that the behavior factors for DCL- and DCM-design are set to $q = 1.5$ respectively $q = 3.0$, i.e. the values given and allowed in the standards for the structure in question.

Table 5.6: *Seismic acceleration input in RSA2014.*

						DCL		DCM	
Case	S	β	T_B [s]	T_C [s]	T_D [s]	q	a_g [g]	q	a_g [g]
1	1.0	0.2	0.10	0.30	1.40	1.5	0.10	3.0	0.10
2	1.0	0.2	0.10	0.30	1.40	1.5	0.16	3.0	0.16
3	1.0	0.2	0.10	0.30	1.40	1.5	0.22	3.0	0.22
4	1.0	0.2	0.10	0.30	1.40	1.5	0.28	3.0	0.28

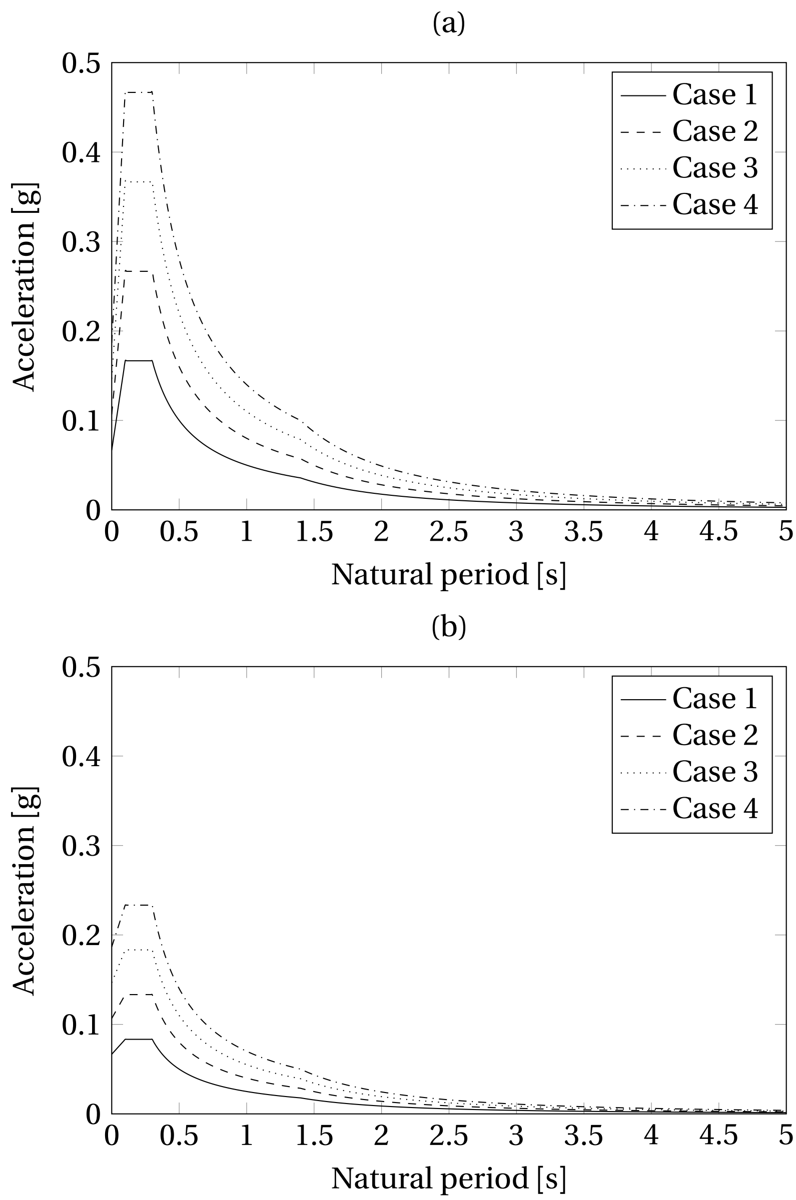
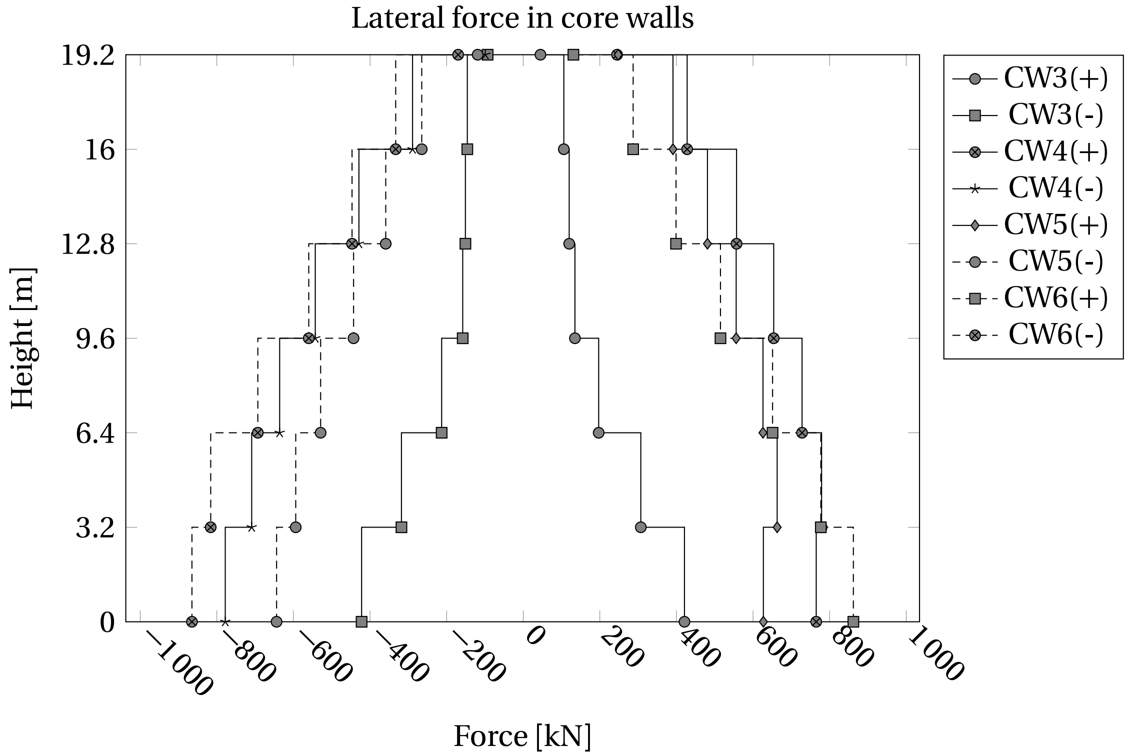


Figure 5.3: *Design response spectra for analysis in RSA2014:* ***(a)*** *DCM-design* ***(b)*** *DCL-design (see Table 5.6).*

5.4 Analysis results

In this section the results of the structural analysis in RSA2014 are presented. The lateral forces and moments in the core walls are plotted with changing design acceleration and behavior factor, i.e. DCL- and DCM-design. Note that the positive and negative seismic impact in the load combination are denoted with (+) respectively (-) in the diagram legend.

The results from the analysis for the lateral force and the moment are plotted for each acceleration case in DCL-design and DCM-design (Figures 5.4 – 5.11). The vertical static force for all cases is shown in Figure 5.12. The values plotted in the graphs are specified in Appendix A.2.



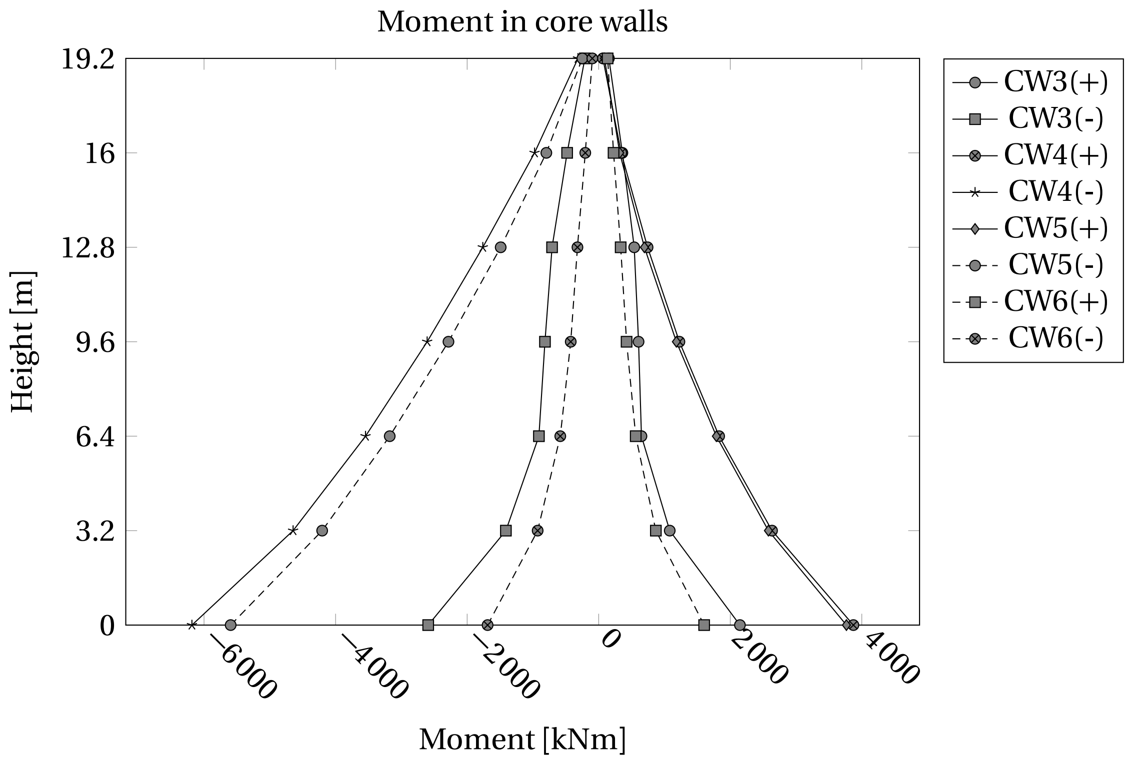


Figure 5.4: *[Case 1: $a_g \cdot S = 0.10$ g, DCL: $q = 1.5$] Lateral force and moment in the core walls from analysis in RSA2014 ((+): positive sign seismic action, (-): negative sign seismic action).*

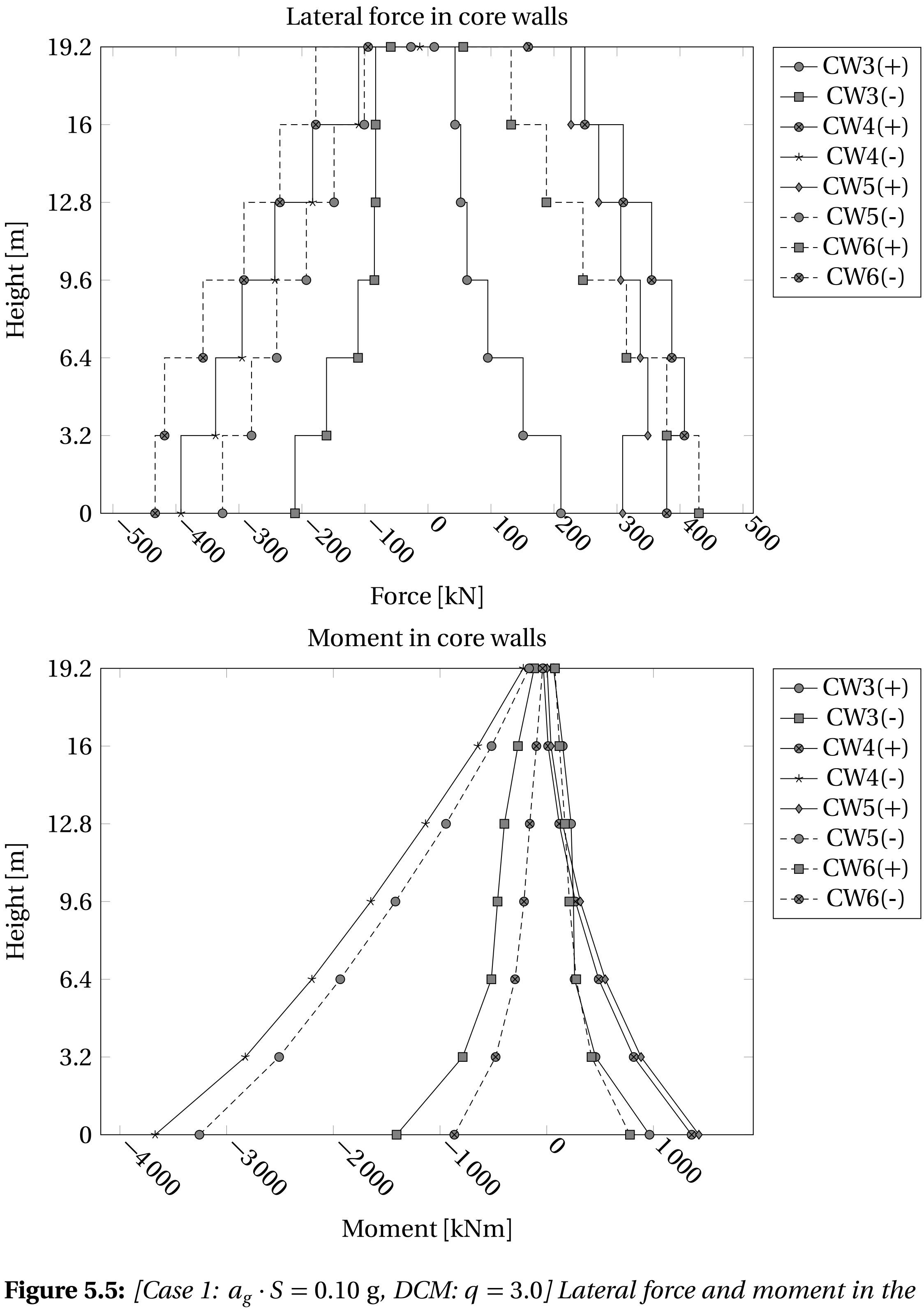
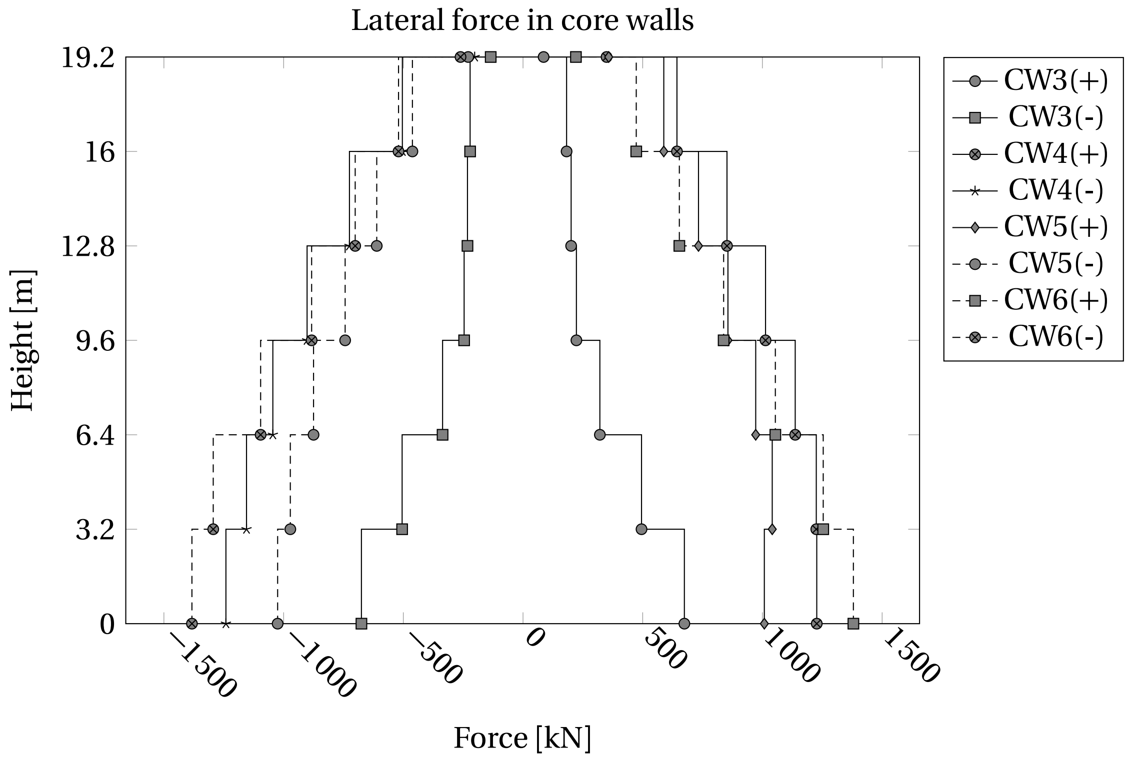


Figure 5.5: *[Case 1: $a_g \cdot S = 0.10$ g, DCM: $q = 3.0$] Lateral force and moment in the core walls from analysis in RSA2014 ((+): positive sign seismic action, (-): negative sign seismic action).*



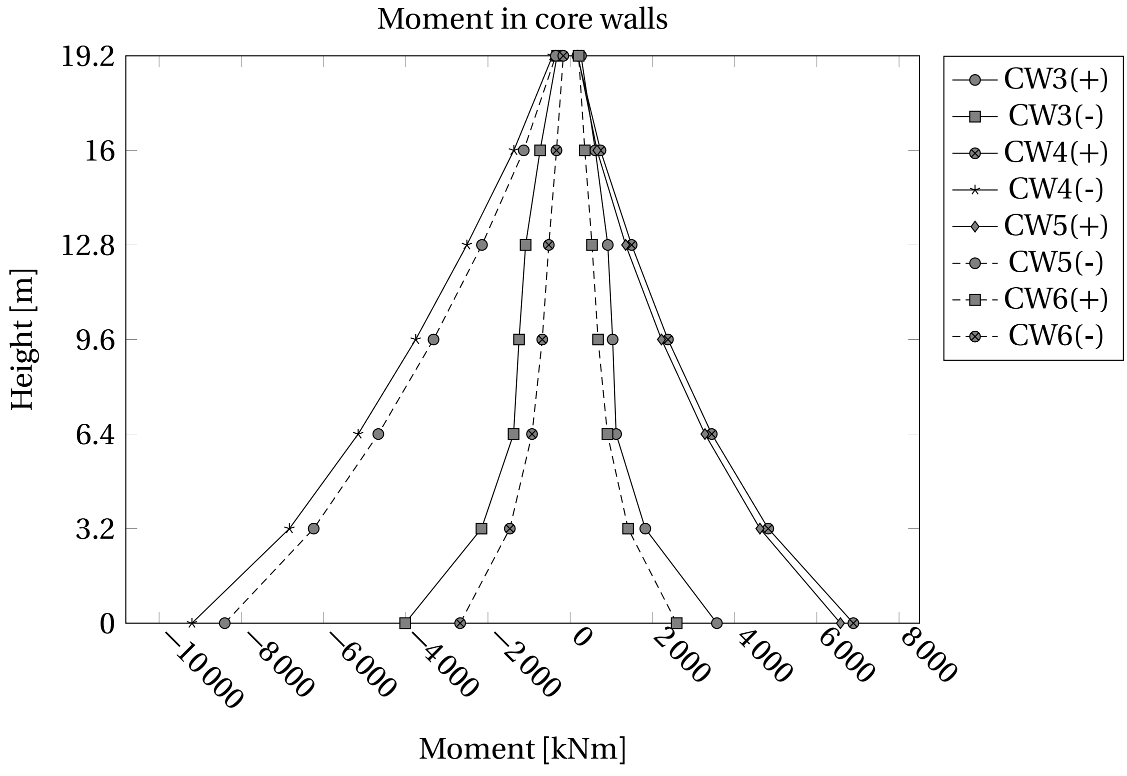
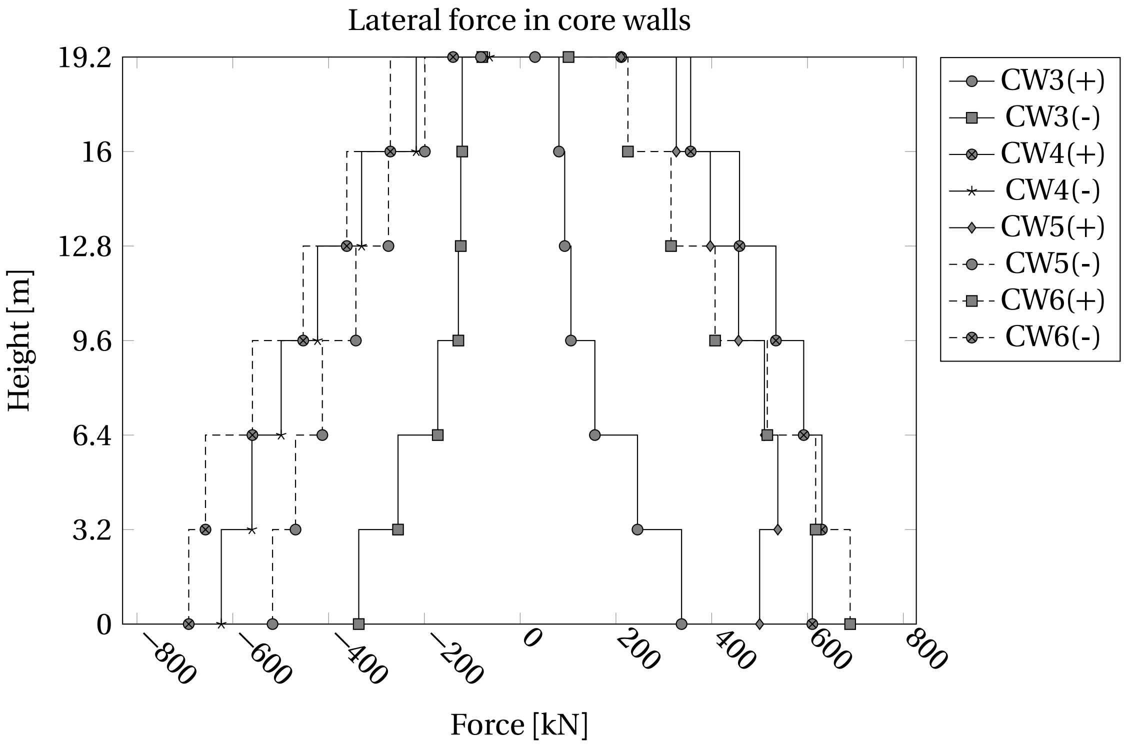


Figure 5.6: *[Case 2: $a_g \cdot S = 0.16$ g, DCL: $q = 1.5$] Lateral force and moment in the core walls from analysis in RSA2014 ((+): positive sign seismic action, (-): negative sign seismic action).*



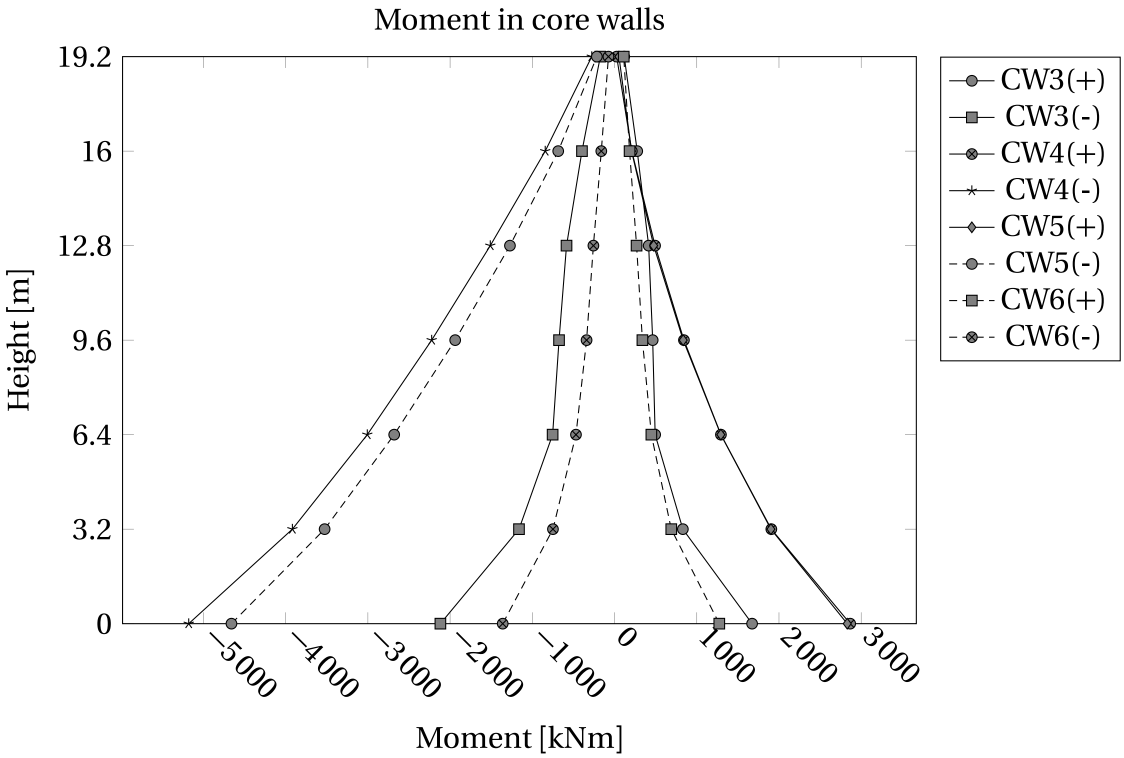


Figure 5.7: *[Case 2: $a_g \cdot S = 0.16$ g, DCM: $q = 3.0$] Lateral force and moment in the core walls from analysis in RSA2014 ((+): positive sign seismic action, (-): negative sign seismic action).*

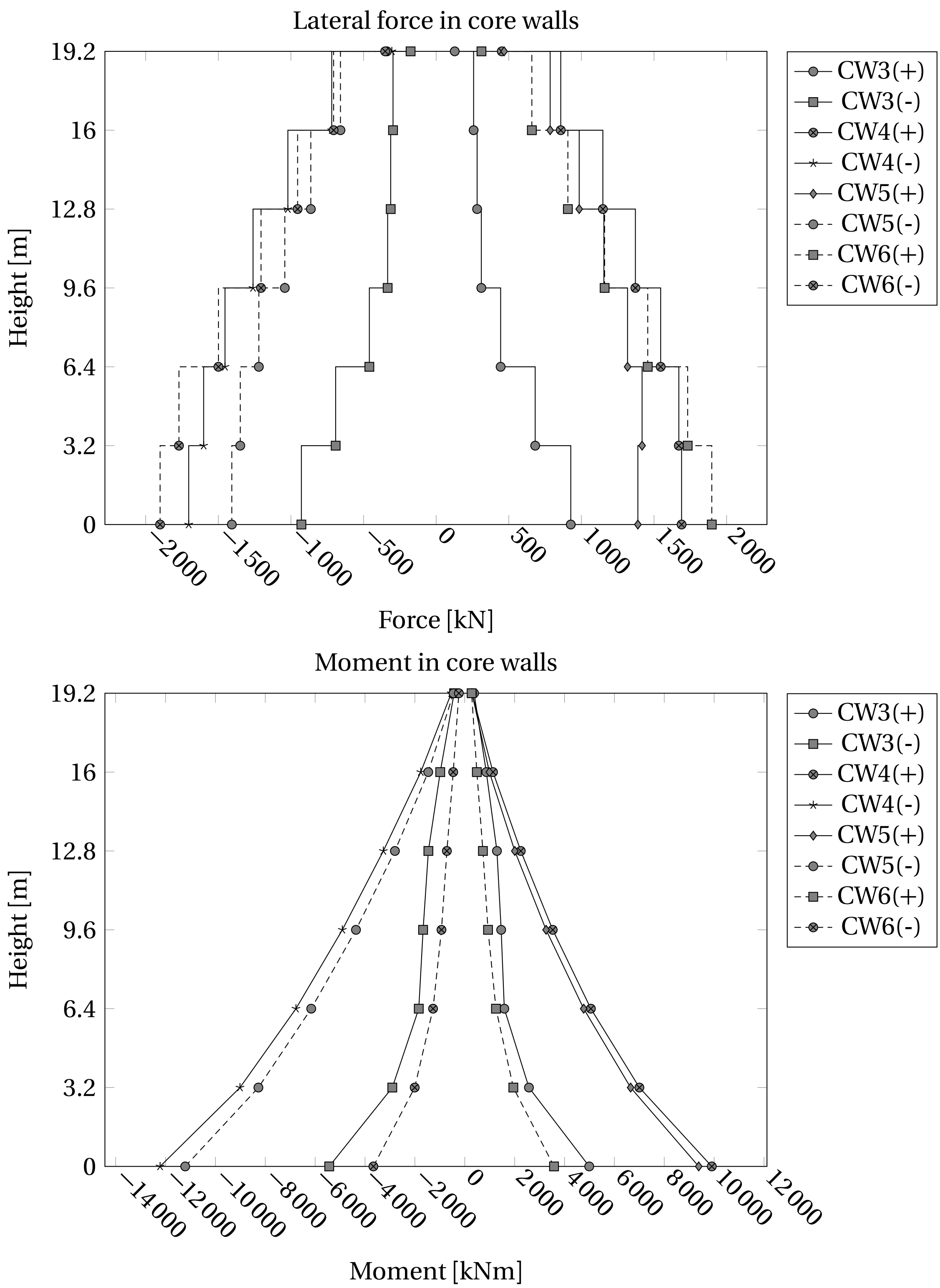
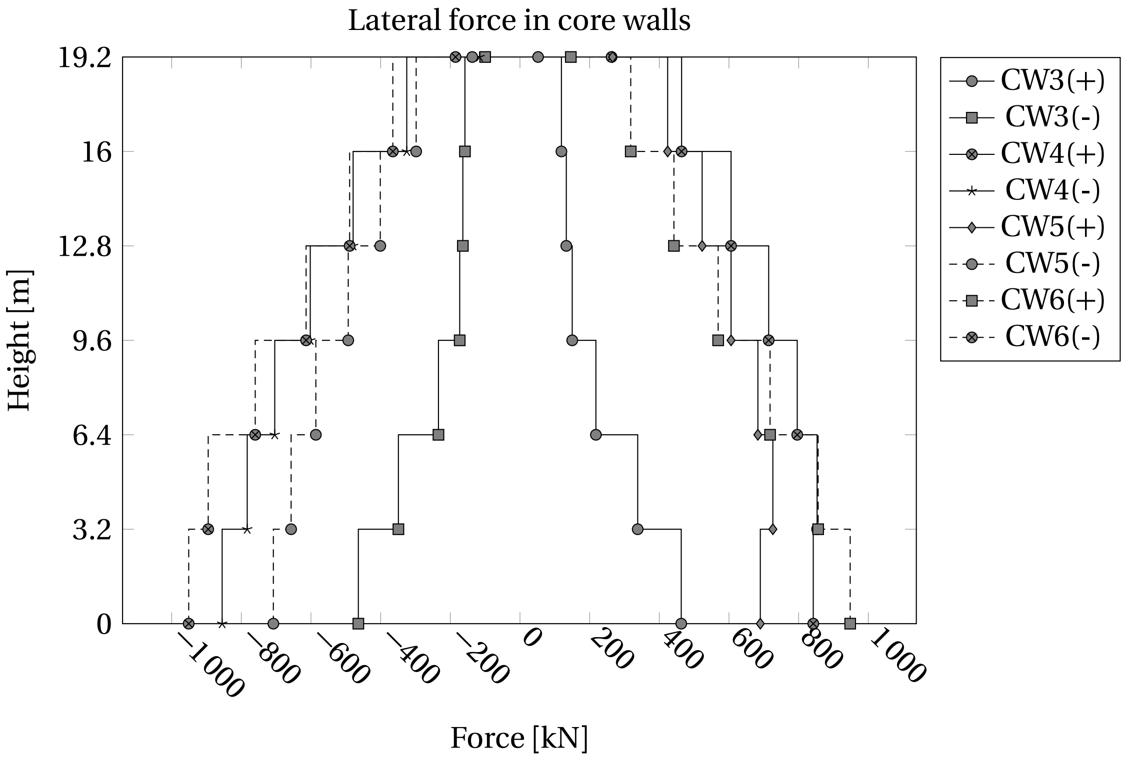


Figure 5.8: *[Case 3: $a_g \cdot S = 0.22$ g, DCL: $q = 1.5$] Lateral force and moment in the core walls from analysis in RSA2014 ((+): positive sign seismic action, (-): negative sign seismic action).*



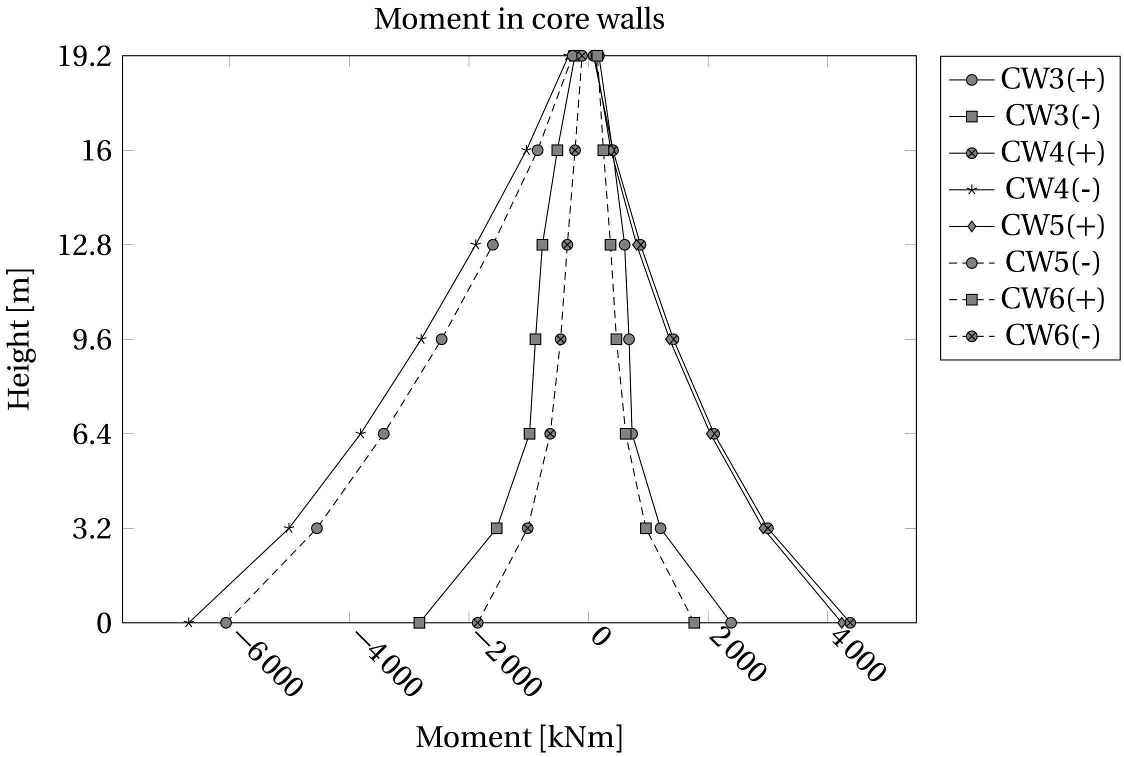


Figure 5.9: *[Case 3: $a_g \cdot S = 0.22$ g, DCM: $q = 3.0$] Lateral force and moment in the core walls from analysis in RSA2014 ((+): positive sign seismic action, (-): negative sign seismic action).*

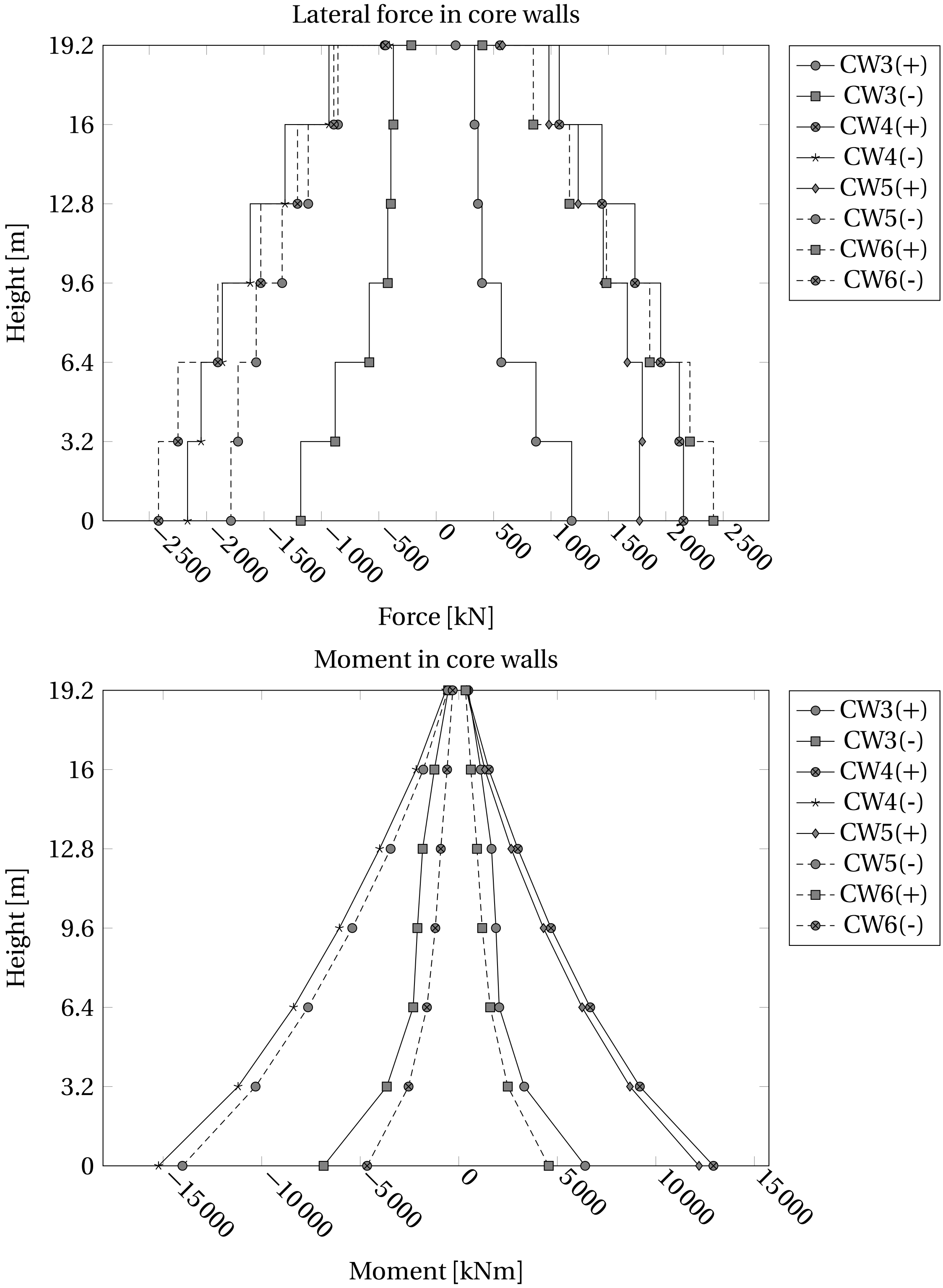
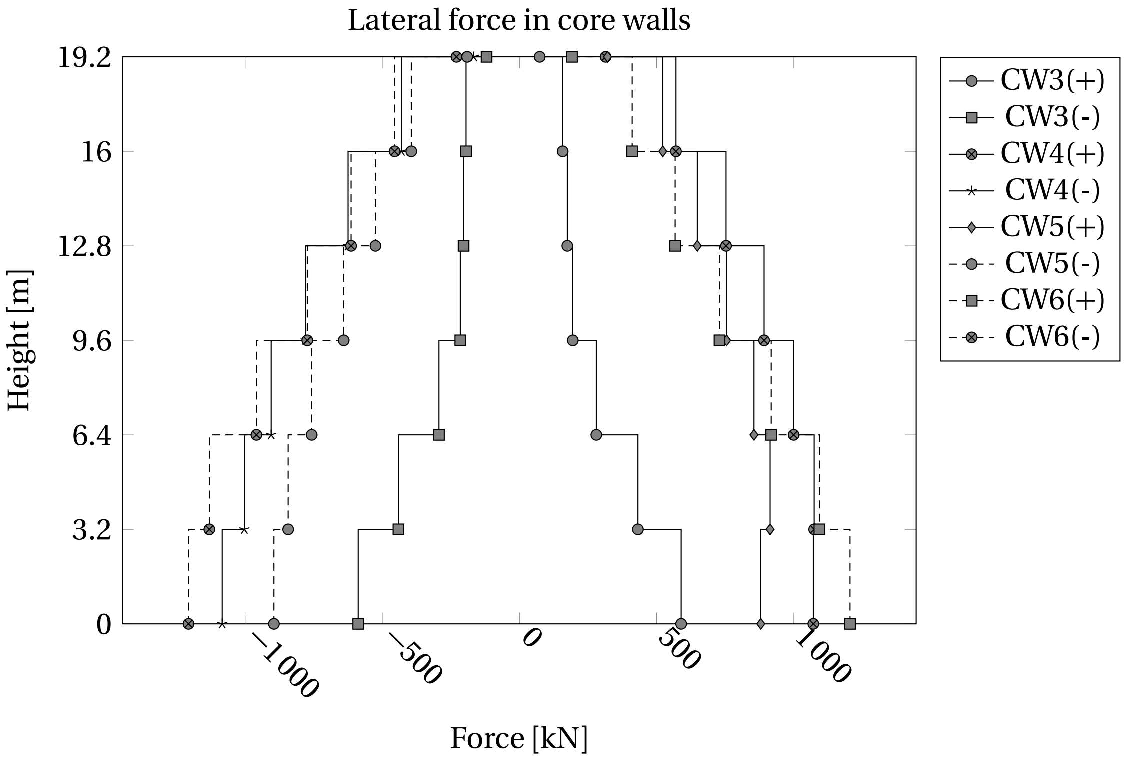


Figure 5.10: *[Case 4: $a_g \cdot S = 0.28$ g, DCL: $q = 1.5$] Lateral force and moment in the core walls from analysis in RSA2014 ((+): positive sign seismic action, (-): negative sign seismic action).*



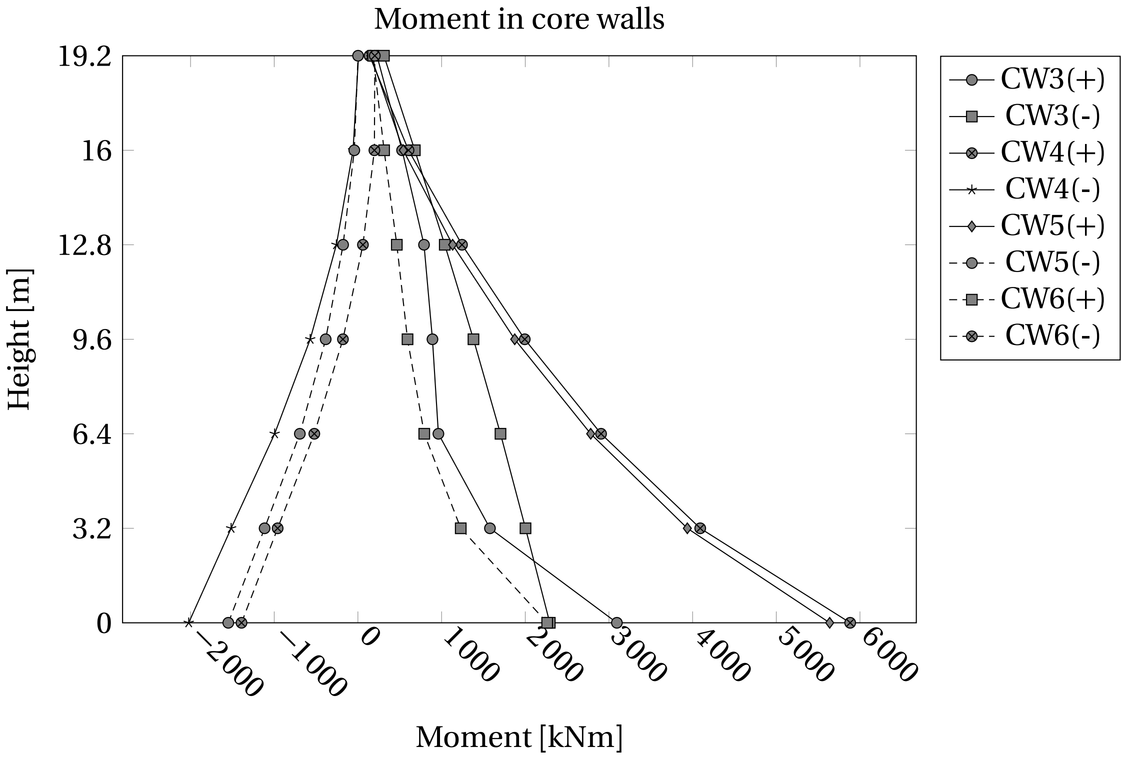


Figure 5.11: *[Case 4: $a_g \cdot S = 0.28$ g, DCM: $q = 3.0$] Lateral force and moment in the core walls from analysis in RSA2014 ((+): positive sign seismic action, (-): negative sign seismic action).*

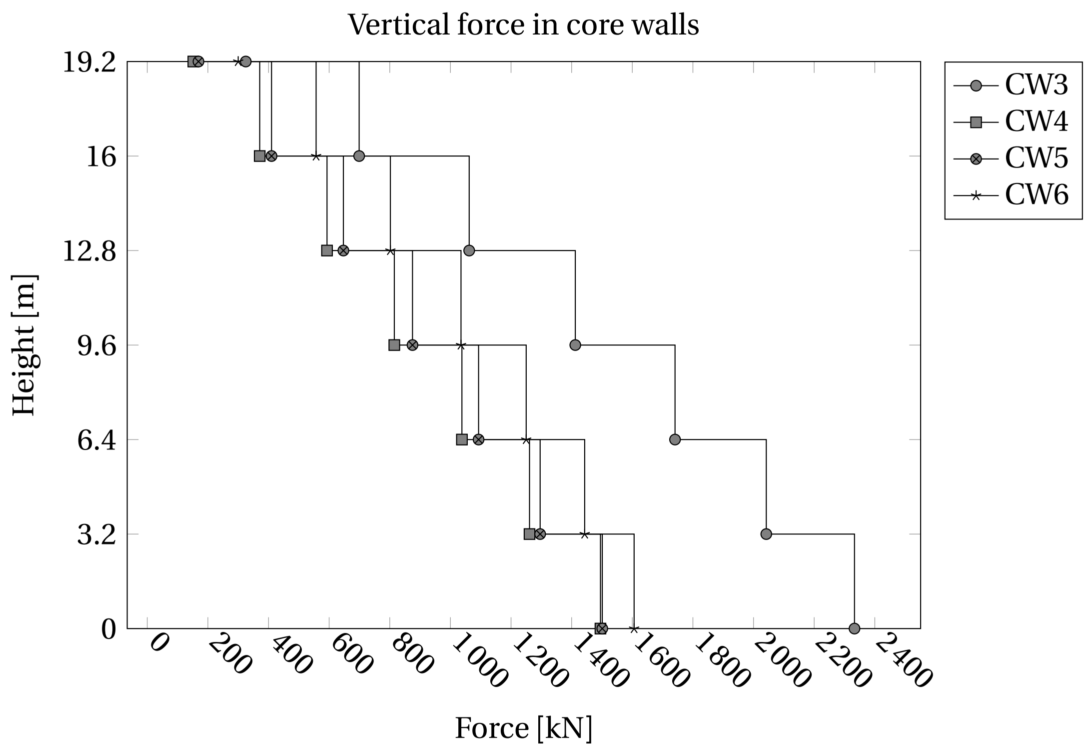


Figure 5.12: *Vertical static force from analysis in RSA2014.*

5.5 Design

Design calculations are computed for all cases (both negative and positive seismic action in the load combinations) for DCL and DCM. The full presentation results of the computations are attached in Appendix B.1.

The following subsections presents dimensioning examples of core wall 3 (CW3) within the critical height, i.e. at the foundation level.

5.5.1 Calculation procedure for DCL-design

The fundamental period is extracted from the modal analysis in RSA2014, shown in Table A.1.

$$T_1 = 0.79 \text{ s}$$

Materials

The quality of concrete is chosen and corresponding partial factors are defined:

$$\gamma_{c,DCL} = 1.2$$

$$\gamma_{s,DCL} = 1.0$$

Characteristic steel strength (B500C) is set to:

$$f_{yk} = 500 \text{ MPa}$$

Dimensioning steel strength is then given as:

$$\begin{aligned} f_{yd,DCL} &= \frac{f_{yk}}{\gamma_{s,DCL}} \\ &= \frac{500 \text{ MPa}}{1.0} \\ &= 500 \text{ MPa} \end{aligned}$$

Characteristic concrete strength (B30) is set to:

$$f_{ck} = 30 \text{ MPa}$$

Dimensioning concrete strength is then given as:

$$\begin{aligned} f_{cd,DCL} &= \frac{f_{ck}}{\gamma_{c,DCL}} \\ &= \frac{30 \text{ MPa}}{1.2} \\ &= 25 \text{ MPa} \end{aligned}$$

Characteristic concrete tensile strength:

$$f_{ctk,0.05} = 2 \text{ MPa}$$

Dimensioning concrete tensile strength is then given as:

$$\begin{aligned} f_{ctd,DCL} &= \frac{f_{ctk,0.05}}{\gamma_{c,DCL}} \\ &= \frac{2 \text{ MPa}}{1.2} \\ &= 1.7 \text{ MPa} \end{aligned}$$

Geometry

Table 5.7: *Geometry properties of core walls.*

Core wall	l_w [m]	b_w [m]	c_{conf} [m]
CW3	4	0.200	0.030
CW4	6	0.200	0.030
CW5	6	0.200	0.030
CW6	4	0.200	0.030

Depth to center of reinforcement is then given as:

$$\begin{aligned} d &= b_w - c_{conf} - \frac{\phi_v}{2} \\ &= 0.200\ \text{m} - 0.030\ \text{m} - \frac{0.012\ \text{m}}{2} \\ &= 0.164\ \text{m} \end{aligned}$$

Tension and compression

In this phase the "section-quantities" are extracted from RSA2014. See the results from RSA2014 in section 5.4. For this specific example, the first level of core wall 3 (CW3) is designed with case 4 (positive load combination) as governing seismic action. The analysis results are depicted in Figure 5.10. The vertical static load is extracted from Figure 5.12.

$$V_{RSA} = 1180\ \text{kN} \qquad M_{RSA} = 6413\ \text{kNm} \qquad N_{RSA} = 2333\ \text{kN}$$

In DCL-design the moment, M_{RSA}, and shear force, V_{RSA} is not increased by any partial factor, see Figure 5.13.

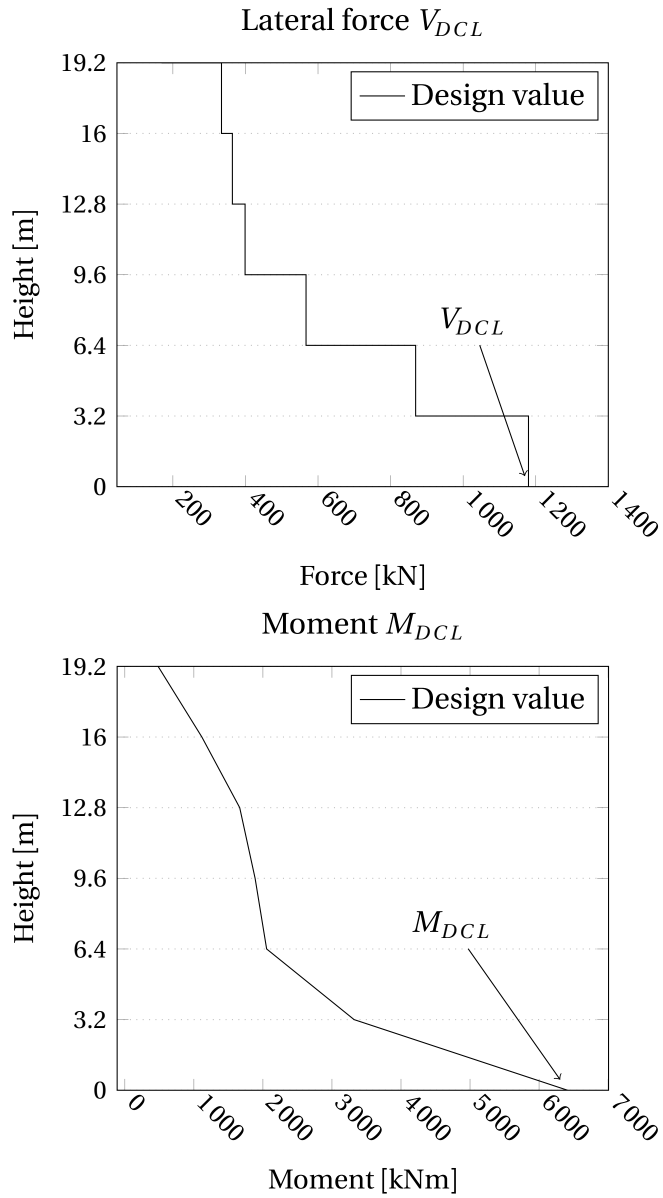


Figure 5.13: *Lateral force and the moment in core wall 3 for DCL-design (Case 4: $a_g = 0.28$ g).*

$$V_{DCL} = V_{RSA} = 1180 \text{ kN}$$

$$M_{DCL} = M_{RSA} = 6413 \text{ kNm}$$

$$N_{DCL} = N_{RSA} = 2333 \text{ kN}$$

These quantities are used to calculate the internal tension, $T_{computed}$, and compression by iteration, which will give the tension force that will be governing for the vertical reinforcement in the tension part of the cross-section.

It is assumed that the internal tension $T_{assumed}$ gives the internal compression resultant. For the calculation example the internal tension is assumed to $T_{assumed} =$ 1080 kN, i.e. tension.

Compression resultant:

$$\begin{aligned} N_c &= N_{DCL} + T_{assumed} \\ &= 2333 \text{ kN} + 1080 \text{ kN} \\ &= 3350 \text{ kN} \end{aligned}$$

For the capacity of the concrete, the following is computed:

$$\begin{aligned} \sigma_c &= 0.75 \cdot f_{cd} \\ &= 0.75 \cdot 25 \text{ MPa} \\ &= 18.75 \text{ MPa} \end{aligned}$$

The length of the compression zone:

$$\begin{aligned} x &= \frac{N_c}{0.584 \cdot \sigma_c \cdot b_w} \\ &= \frac{3350 \text{ kN}}{0.584 \cdot 18.75 \text{ MPa} \cdot 0.200 \text{ m}} \\ &= 1.53 \text{ m} \end{aligned}$$

Location of the compression resultant:

$$\begin{aligned} c_2 &= 0.354 \cdot x \\ &= 0.354 \cdot 1.53 \text{ m} \\ &= 0.542 \text{ m} \end{aligned}$$

The location of the tension resultant is assumed as $c_1 = 500$ mm. Ultimately this gives the internal lever arm:

$$\begin{aligned} z &= l_w - c_2 - c_1 \\ &= 4 \text{ m} - 0.542 \text{ m} - 0.500 \text{ m} \\ &= 2.958 \text{ m} \end{aligned}$$

Finally the internal tension is calculated, and compared to the initial assumed value of $T_{assumed}$. If the margin of error is small the computation is considered successful and no more iterations are required.

$$
\begin{aligned}
T_{computed} &= \frac{M_{DCL}}{z} - \frac{N_{DCL} \cdot (0.5 l_w - c_2)}{z} \\
&= \frac{6413 \text{ kNm}}{2.958 \text{ m}} - \frac{2333 \text{ kN} \cdot (0.5 \cdot 4 \text{ m} - 0.542 \text{ m}}{3.065 \text{ m}} \\
&= 1018 \text{ kN}
\end{aligned}
$$

If $T_{assumed} \approx T_{computed}$ no more iterations are necessary, otherwise $T_{assumed}$ is modified and more iterations are computed.

Depending on the eventual tension or compression the required reinforcement area is computed, and a sufficient amount of reinforcement is selected.

At this point, a selection of reinforcement diameter, ϕ_v, is made. The area required, $A_{s,v}$, is then compared to the reinforcement area of single bar, $A_{\phi,v}$, which ultimately gives the required number of reinforcement bars, n. It is important to keep in mind that in the case of compression (in the entire section) or low tension force, the minimum requirement, $\rho_{v,min,DCL}$ for vertical reinforcement is used. If this is the case then $A_{s,v}$ is replaced by $A_{v,min} = \rho_{v,min} \cdot A_c$. Where $A_c = b_w \cdot l_c$ is the critical concrete area and l_c is the length of the critical area.

$$\rho_{v,min,DCL} = 0.2 \text{ \%}$$

Required total reinforcement area is then given by:

$$
\begin{aligned}
A_{s,v} &= \frac{T_{computed}}{f_{yd}} \\
&= \frac{1018 \text{ kN}}{500 \text{ MPa}} \\
&= 2036 \text{ mm}^2
\end{aligned}
$$

Length of critical zone:

$$
\begin{aligned}
l_c &= \max(0.15 \cdot l_w, 1.5 \cdot b_w) \\
&= \max(0.15 \cdot 4 \text{ m}, 1.5 \cdot 0.200 \text{ m}) \\
&= 0.600 \text{ m}
\end{aligned}
$$

The vertical reinforcement diameter is selected to $\phi_v = 20$ mm. The area of the selected rebar is:

$$
\begin{aligned}
A_{\phi,v} &= \frac{\pi \cdot \phi_v^2}{4} \\
&= \frac{\pi \cdot (20\ \text{mm})^2}{4} \\
&= 314\ \text{mm}^2
\end{aligned}
$$

Amount of ϕ_v needed in order to fulfill the requirement:

$$
\begin{aligned}
n &= \frac{A_{s,v}}{A_{\phi,v}} \\
&= \frac{2036\ \text{mm}^2}{314\ \text{mm}^2} \\
&\approx 8
\end{aligned}
$$

Total vertical reinforcement in wall boundary

$$
\begin{aligned}
A_{\phi,v,boundary,tot} &= n \cdot A_{\phi,v} \\
&= 8 \cdot 314\ \text{mm}^2 \\
&= 2513\ \text{mm}^2
\end{aligned}
$$

Spacing of the vertical reinforcement in the critical zone:

$$
\begin{aligned}
s_v &= \frac{l_c - c}{n} \\
&= \frac{0.600\ \text{m} - 0.030\ \text{m}}{8/2} \\
&= 140\ \text{mm}
\end{aligned}
$$

This means that in each boundary of the wall $2 \times 4\phi 20 s 140$ is used for the detailing.

The connection of the dowels to the wall with the foundation is detailed the same as for the vertical boundary reinforcement, i.e. $2 \times 4\phi 20 s 140$ with dowel length set to 850 mm.

Shear reinforcement between separate walls

The resistance of of an non-reinforced connections is given by:

$$
\begin{aligned}
V_{Rd,i} &= 0.03 \cdot f_{ctd} \cdot A_i + 0.5 \cdot f_{yd} \cdot A_s + 0.5 \cdot N_{DCL} \\
&= 0.03 \cdot 1.7\ \text{MPa} \cdot (4\ \text{m} \cdot 0.200\ \text{m}) + 0.5 \cdot 500\ \text{MPa} \cdot 0\ \text{mm}^2 + 0.5 \cdot 2333\ \text{kN} \\
&= 1206\ \text{kN}
\end{aligned}
$$

where A_i is the cross-section area of the connection.

This gives the shear force needed to be taken by the reinforcement (dowels):

$$V_s = \begin{cases} 0 & \text{if } V_{DCL} \leq V_{Rd,i} \\ V_{DCL} - V_{Rd,i} & \text{if } V_{DCL} \geq V_{Rd,i} \end{cases}$$

$$V_{DCL} < V_{Rd,i} \Rightarrow V_s = 0\ \text{kN}$$

In this case enough resistance is already provided and no extra shear reinforcement is required.

Otherwise, the selection of the dowels is made by the required reinforcement area:

$$A_s = \frac{V_s}{0.5 \cdot f_{yd}}$$

In seismic load situations, reinforcement that takes shear force is placed in mid-span of the wall and required tension/compression reinforcement are placed in the boundaries.

Strain check

In order to control if the reinforcement develops full strength capacity the strain is checked.

Compressive strain in the concrete from EC2:

$$\varepsilon_{cu2} = 0.2\%$$

$$\begin{aligned} \varepsilon_c &= \varepsilon_{cu2} \cdot (1 - \sqrt{1 - 0.75}) \\ &= 0.002 \cdot (1 - \sqrt{1 - 0.75}) \\ &= 0.001 = 0.1\ \% \end{aligned}$$

Strain in tensional reinforcement is given by:

$$\begin{aligned} \varepsilon_s &= \frac{\varepsilon_c (l_w - c_1 - x)}{x} \\ &= \frac{0.001 \cdot (4\ \text{m} - 0.500\ \text{m} - 1.53\ \text{m})}{1.53\ \text{m}} \\ &= 0.0013 = 1.3\ \text{‰} \end{aligned}$$

This value is then compared to $\varepsilon_{sy,d} = 2.5$ ‰ and $\varepsilon_{cu} = 30$ ‰. In order for the reinforcement to develop full strength capacity:

$$\begin{array}{ccc} \varepsilon_{sy,d} < & \varepsilon_s < & \varepsilon_{cu} \\ 2.5‰ < & 1.3‰ < & 30‰ \Rightarrow \text{OK!} \end{array}$$

Control of plastic hinge

Control of the reinforcement amount in the critical zone of the wall is managed by comparing the reinforcement amount, $A_{\phi,v,boundary,tot}$, and the balanced reinforced cross-section, $A_{s,b}$.

Length from outermost fiber in the compression zone to the center of the reinforcement:

$$\begin{aligned} d_l &= l_w - 0.500 \text{ m} \\ &= 4 \text{ m} - 0.500 \text{ m} \\ &= 3.5 \text{ m} \end{aligned}$$

Ultimate compressive strain in the concrete from EC2:

$$\varepsilon_{cu} = 3.5 ‰$$

Strain in reinforcement steel:

$$\varepsilon_{sy,d} = 2.5 ‰$$

In order to get balanced reinforced compression zone:

$$\begin{aligned} \alpha_b &= \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{sy,d}} \\ &= \frac{0.0035}{0.0035 + 0.0025} \\ &= 0.583 = 58.3 \% \end{aligned}$$

The following equation for balanced reinforced cross-section is valid for concrete qualities $f_{ck} < 50$ MPa. Factor $\lambda_d = 0.8$.

$$\begin{aligned} A_{s,b} &= \lambda_d \cdot \frac{f_{cd}}{f_{yd}} \cdot b_w \cdot d_l \cdot \alpha_b \\ &= 0.8 \cdot \frac{25 \text{ MPa}}{500 \text{ MPa}} \cdot 0.200 \text{ m} \cdot 3.5 \text{ m} \cdot 0.583 \\ &= 16333 \text{ mm}^2 \end{aligned}$$

In order for plastic hinge to occur in the critical zone:

$$\begin{aligned} A_{s,b} &\gg A_{\phi,v,boundary,tot} \\ 16333\ \text{mm}^2 &\gg 2513\ \text{mm}^2 \Rightarrow \text{OK!} \end{aligned}$$

The relationship indicates that the wall is strongly under reinforced, which is necessary in order for plastic hinges to occur.

The reinforcement in compression must be secured against buckling and this is done by introducing confinement hoops in the critical zone.

Vertical reinforcement in the web of the wall

The vertical reinforcement is designed according to the standards in EC2, which in this case means that the minimum reinforcement applies. The reason is that the vertical reinforcement in the boundaries are designed to take the moment effect on the wall.

Minimum vertical reinforcement criteria:

$$\rho_{v,min} = 0.2\ \% \tag{5.3}$$

This gives the minimum required vertical reinforcement in the wall:

$$\begin{aligned} A_{v,min} &= \rho_{v,min} \cdot (l_w - 2 \cdot l_c) \cdot b_w \\ &= 0.002 \cdot (4\ \text{m} - 2 \cdot 0.600\ \text{m}) \cdot 0.200\ \text{m} \\ &= 1120\ \text{mm}^2 \end{aligned}$$

Selection of the vertical reinforcement in the web is $\phi_v = 10$ mm. The area of a single bar is:

$$\begin{aligned} A_{\phi,v} &= \frac{\pi \cdot \phi_v^2}{4} \\ &= \frac{\pi \cdot (10\ \text{mm})^2}{4} \\ &= 79\ \text{mm}^2 \end{aligned}$$

Amount of rebars in the web of the wall:

$$
\begin{aligned}
n &= \frac{A_{v,min}}{A_{\phi,v}} \\
&= \frac{1120\ \mathrm{mm}^2}{79\ \mathrm{mm}^2} \\
&\approx 16
\end{aligned}
$$

Total area of vertical reinforcement in the web of the wall:

$$
\begin{aligned}
A_{\phi,v,web,tot} &= n \cdot A_{\phi,v} \\
&= 16 \cdot 79\ \mathrm{mm}^2 \\
&= 1206\ \mathrm{mm}^2
\end{aligned}
$$

EC2 also prescribes the maximum spacing for the vertical bars to:

$$
\begin{aligned}
s_{v,max} &\leq \min(3 \cdot b_w, 400\ \mathrm{mm}) \\
&\leq \min(3 \cdot 0.200\ \mathrm{mm}, 400\ \mathrm{mm}) \\
&\leq 400\ \mathrm{mm}
\end{aligned}
$$

Spacing between the vertical reinforcement bars in the web of the wall:

$$
\begin{aligned}
s_v &= \frac{l_w - 2 \cdot l_c}{n/2} \\
&= \frac{4\ \mathrm{m} - 2 \cdot 0.600\ \mathrm{m}}{16/2} \\
&\approx 350\ \mathrm{mm} < 400\ \mathrm{mm} \Rightarrow \mathrm{OK!}
\end{aligned}
$$

This means that in the web of the wall $2 \times 8\phi 10 s 350$ is used for the detailing.

Horizontal reinforcement in wall (shear reinforcement)

Recommended amount of horizontal reinforcement is designed and selected to:

Minimum horizontal reinforcement:

$$
\begin{aligned}
\rho_{h,min} &= \max(0.1\ \%, 0.25 \cdot \rho_v) \\
&= \max(0.1\ \%, 0.25 \cdot 0.002) \\
&= 0.001 = 0.1\ \%
\end{aligned}
$$

Minimum area of the horizontal reinforcement:

$$\begin{aligned} A_{h,min} &= \rho_{h,min} \cdot h_s \cdot b_w \\ &= 0.001 \cdot 3.2 \text{ m} \cdot 0.200 \text{ m} \\ &= 640 \text{ mm}^2 \end{aligned}$$

EC2 also prescribes the maximum spacing for the horizontal bars to:

$$s_h \leq 400 \text{ mm}$$

The horizontal reinforcement must consider the actions acting on the wall, i.e. the lateral forces. This means that more reinforcement than the minimum might be needed.

According to EC8:

$$\begin{aligned} k &= \min\left(1 + \sqrt{\frac{200 \text{ mm}}{d}}, 2\right) \\ &= \min\left(1 + \sqrt{\frac{200 \text{ mm}}{164 \text{ mm}}}, 2\right) \\ &= 2 \end{aligned}$$

Vertical reinforcement content:

$$\begin{aligned} \rho_v &= \min\left(\frac{A_{\phi,v,boundary,tot} + A_{\phi,v,web,tot}}{l_w \cdot d}, 0.002\right) \\ &= \min\left(\frac{2513 \text{ mm}^2 + 1257 \text{ mm}^2}{4 \text{ m} \cdot 0.164 \text{ m}}, 0.002\right) \\ &= 0.002 = 0.2\ \% \end{aligned}$$

Factors according to EC8:

$$k_1 = 0.15 \qquad k_2 = 0.15$$

$$\begin{aligned} C_{Rd,c} &= \frac{k_2}{\gamma_c} \\ &= \frac{0.15}{1.2} \\ &= 0.125 \end{aligned}$$

$$\begin{aligned} v_{min} &= 0.035 \cdot k^{(3/2)} \cdot \sqrt{f_{ck}} \\ &= 0.035 \cdot 2^{(3/2)} \cdot \sqrt{30 \text{ MPa}} \\ &= 0.542 \end{aligned}$$

Limitation of compression strain:

$$
\begin{aligned}
\sigma_{cp} &= \min\left(\frac{N_{DCL}}{b_w \cdot l_w}, 0.2 \cdot f_{cd}\right) \\
&= \min\left(\frac{2333\ \text{kN}}{0.200\ \text{m} \cdot 4\ \text{m}}, 0.2 \cdot 25\ \text{MPa}\right) \\
&= 2.92\ \text{MPa}
\end{aligned}
$$

Shear resistance of the wall:

$$
\begin{aligned}
V_{Rd,c,V} &= C_{Rd,c} \cdot k \cdot (100 \cdot \rho_v \cdot f_{ck})^{(1/3)} \cdot l_w \cdot d \\
&= 0.125 \cdot 2 \cdot (100 \cdot 0.002 \cdot 30\ \text{MPa})^{(1/3)} \cdot 4\ \text{m} \cdot 0.164\ \text{m} \\
&= 18.48\ \text{kN}
\end{aligned}
$$

Resistance contribution of axial force:

$$
\begin{aligned}
V_{Rd,c,N} &= k_1 \cdot \sigma_{cp} \cdot l_w \cdot d \\
&= 0.15 \cdot 2.92\ \text{MPa} \cdot 4\ \text{m} \cdot 0.164\ \text{m} \\
&= 286.9\ \text{kN}
\end{aligned}
$$

Lower limit for the shear resistance:

$$
\begin{aligned}
V_{Rd,c,min} &= v_{min} \cdot l_w \cdot d \\
&= 0.542 \cdot 4\ \text{m} \cdot 0.164\ \text{m} \\
&= 355.7\ \text{kN}
\end{aligned}
$$

Lateral resistance without horizontal reinforcement:

$$
\begin{aligned}
V_{Rd,c} &= \max(V_{Rd,c,V} + V_{Rd,c,N}, V_{Rd,c,min} \cdot d \\
&= \max(18.48\ \text{kN} + 286.9\ \text{kN}, 355.7\ \text{kN} \cdot 0.164\ \text{m} \\
&= 355.7\ \text{kN}
\end{aligned}
$$

Force that has to be horizontal reinforcement:

$$
V_s = \begin{cases} 0 & \text{if } V_{DCL} \leq V_{Rd,c} \\ V_{DCL} - V_{Rd,c} & \text{if } V_{DCL} \geq V_{Rd,c} \end{cases}
$$

$$\begin{aligned} V_{DCL} &\geq V_{Rd,c} \\ 1180 \text{ kN} &> 355.7 \text{ kN} \\ \Rightarrow V_s &= V_{DCL} - V_{Rd,c} \\ &= 1180 \text{ kN} - 355.7 \text{ kN} \\ &= 824.2 \text{ kN} \end{aligned}$$

Required area of the horizontal reinforcement:

$$\begin{aligned} A_{s,h} &= \sqrt{3} \cdot \frac{V_s}{f_y d} \\ &= \sqrt{3} \cdot \frac{824.2 \text{ kN}}{500 \text{ MPa}} \\ &= 2855 \text{ mm}^2 \end{aligned}$$

Selection of the horizontal reinforcement in the web and boundary is $\phi_h = 10$ mm. The area of a single bar is:

$$\begin{aligned} A_{\phi,h} &= \frac{\pi \cdot \phi_h^2}{4} \\ &= \frac{\pi \cdot (10 \text{ mm})^2}{4} \\ &= 79 \text{ mm}^2 \end{aligned}$$

Amount of rebars in the web and boundary of the wall:

$$\begin{aligned} n &= \frac{A_{s,h}}{A_{\phi,h}} \\ &= \frac{2855 \text{ mm}^2}{79 \text{ mm}^2} \\ &\approx 38 \end{aligned}$$

Total area of the horizontal reinforcement bars:

$$\begin{aligned} A_{\phi,h,tot} &= n \cdot A_{\phi,h} \\ &= 38 \cdot 79 \text{ mm}^2 \\ &= 2985 \text{ mm}^2 \end{aligned}$$

Spacing of the horizontal reinforcement:

$$
\begin{aligned}
s_h &= \frac{h_s}{n/2} \\
&= \frac{3.2 \text{ m}}{38/2} \\
&\approx 170 \text{ mm} < 400 \text{ mm} \Rightarrow \text{OK!}
\end{aligned}
$$

This means that for each side of the wall $2 \times 19\phi 10s170$ is used for the detailing.

Table 5.8: *Compilation of data for detailing of wall in DCL.*

Type	Location	ϕ [mm]	Amount	s [mm]	Anchorage [mm]
Vertical	Boundary	20	8	140	
Dowel	Boundary	20	8	140	850
Vertical	Web	10	16	350	
Shear dowel	Mid-span	-	-	-	-
Horizontal	Boundary + web	10	37	170	

5.5.2 Calculation procedure for DCM-design

The fundamental period is extracted from the modal analysis in RSA2014, shown in Table A.1.

$$T_1 = 0.79 \text{ s}$$

Materials

The quality of concrete is chosen and corresponding partial factors are defined:

$$\gamma_{c,DCM} = 1.5$$

$$\gamma_{s,DCM} = 1.15$$

Characteristic steel strength (B500C) is set to:

$$f_{yk} = 500 \text{ MPa}$$

Dimensioning steel strength is then given as:

$$\begin{aligned} f_{yd,DCM} &= \frac{f_{yk}}{\gamma_{s,DCM}} \\ &= \frac{500\text{ MPa}}{1.15} \\ &= 435\text{ MPa} \end{aligned}$$

Characteristic concrete strength (B30) is set to:

$$f_{ck} = 30\text{ MPa}$$

Dimensioning concrete strength is then given as:

$$\begin{aligned} f_{cd,DCM} &= \frac{f_{ck}}{\gamma_{c,DCM}} \\ &= \frac{30\text{ MPa}}{1.5} \\ &= 20\text{ MPa} \end{aligned}$$

Characteristic concrete tensile strength:

$$f_{ctk,0.05} = 2\text{ MPa}$$

Dimensioning concrete tensile strength is then given as:

$$\begin{aligned} f_{ctd,DCM} &= \frac{f_{ctk,0.05}}{\gamma_{c,DCM}} \\ &= \frac{2\text{ MPa}}{1.5} \\ &= 1.3\text{ MPa} \end{aligned}$$

Geometry

Table 5.9: *Geometry properties of core walls.*

Core wall	l_w [m]	b_w [m]	c_{conf} [m]
CW3	4	0.200	0.030
CW4	6	0.200	0.030
CW5	6	0.200	0.030
CW6	4	0.200	0.030

Depth to center of reinforcement is then given as:

$$\begin{aligned} d &= b_w - c_{conf} - \frac{\phi_v}{2} \\ &= 0.200\ \text{m} - 0.030\ \text{m} - \frac{0.012\ \text{m}}{2} \\ &= 0.164\ \text{m} \end{aligned}$$

The condition for ductile wall design must be fulfilled:

$$\begin{aligned} \frac{l_w}{b_w} &> 4 \\ \frac{4\ \text{m}}{0.200\ \text{m}} = 20 &> 4 \Rightarrow \text{OK!} \end{aligned}$$

The critical height is calculated as below:

$$\begin{aligned} h_{cr} &= \max(l_w, \frac{h_w}{6}) \\ &= \max(4\ \text{m}, \frac{3.2\ \text{m}}{6}) \\ &= 4\ \text{m} \end{aligned}$$

with upper limit set to:

$$\begin{aligned} h_{cr,limit} &= \min(h_{cr}, 2l_w, h_s) \\ &= \min(4\ \text{m}, 2 \cdot 4\ \text{m}, 3.2\ \text{m}) \\ &= 3.2\ \text{m} \end{aligned}$$

Tension and compression

In this phase the "section-quantities" are extracted from RSA2014. See the results from RSA2014 in section 5.4. For this specific example the first level part of core wall 3 (CW3) is designed with case 4 (positive load combination) as governing seismic action. The analysis results are shown in Figure 5.11. The vertical static load is extracted from Figure 5.12.

$$V_{RSA} = 590\ \text{kN} \qquad M_{RSA} = 3095\ \text{kNm} \qquad N_{RSA} = 2333\ \text{kN}$$

In DCM-design the moment, M_{DCM}, is redistributed according to Figure 4.9 at page 62. The shear force, V_{DCM}, is increased by $\gamma_{Rd,DCM} = 1.5$ within the critical height, $\gamma_{Rd,DCM} = 1.2$ in the region closer than two times the thickness of the wall and $\gamma_{Rd,DCM} =$

1.1 in the rest of the wall. The redistribution of the actions in this case is shown in Figure 5.14.

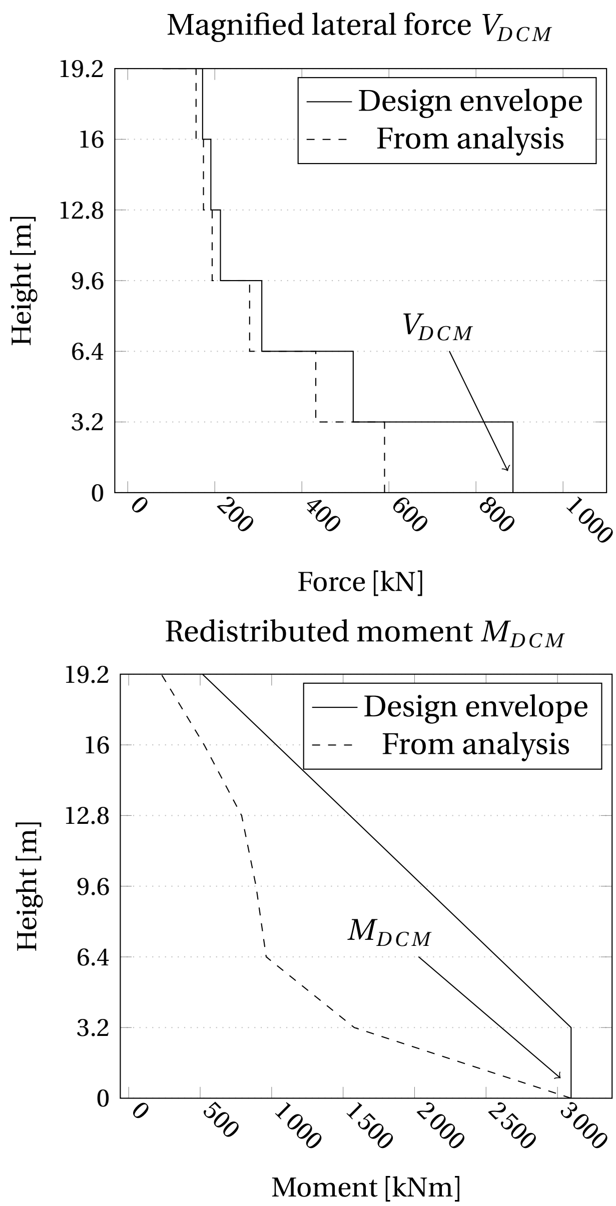


Figure 5.14: *Design envelope of the lateral force and the moment in core wall 3 for DCM-design (Case 4: $a_g = 0.28$ g).*

$$
\begin{aligned}
V_{DCM} &= V_{RSA} \cdot \gamma_{Rd,DCM} \\
&= 590\ \text{kN} \cdot 1.5 \\
&= 885\ \text{kN}
\end{aligned}
$$

$$M_{DCM} = M_{RSA} = 3095\ \text{kNm}$$

$$N_{DCM} = N_{RSA} 2333 \text{ kN}$$

These quantities are used to calculate the internal tension, $T_{computed}$, and compression by iteration, which will give the tension force that will be designing for the vertical reinforcement in the tension part of the cross-section.

The assumption is made that the internal tension $T_{assumed}$ gives the internal compression resultant. For the example calculation, the internal tension is assumed to $T_{assumed} = -181$ kN, i.e. compression.

Compression resultant:

$$\begin{aligned} N_c &= N_{DCM} + T_{assumed} \\ &= 2333 \text{ kN} + (-181 \text{ kN}) \\ &= 2151 \text{ kN} \end{aligned}$$

For the capacity of the concrete following is computed:

$$\begin{aligned} \sigma_c &= 0.75 \cdot f_{cd} \\ &= 0.75 \cdot 20 \text{ MPa} \\ &= 15 \text{ MPa} \end{aligned}$$

The length of the compression zone:

$$\begin{aligned} x &= \frac{N_c}{0.584 \cdot \sigma_c \cdot b_w} \\ &= \frac{2151 \text{ kN}}{0.584 \cdot 15 \text{ MPa} \cdot 0.200 \text{ m}} \\ &= 1.23 \text{ m} \end{aligned}$$

Location of the compression resultant:

$$\begin{aligned} c_2 &= 0.354 \cdot x \\ &= 0.354 \cdot 1.23 \text{ m} \\ &= 0.435 \text{ m} \end{aligned}$$

The location of the tension resultant is assumed to $c_1 = 500$ mm. Ultimately this gives the internal lever arm:

$$\begin{aligned} z &= l_w - c_2 - c_1 \\ &= 4\ \mathrm{m} - 0.435\ \mathrm{m} - 0.500\ \mathrm{m} \\ &= 3.065\ \mathrm{m} \end{aligned}$$

Finally the internal tension is calculated and compared to the initial assumed value of $T_{assumed}$. If the margin of error is small the computation is considered successful and no more iterations are required.

$$\begin{aligned} T_{computed} &= \frac{M_{DCM}}{z} - \frac{N_{DCM} \cdot (0.5 l_w - c_2)}{z} \\ &= \frac{3095\ \mathrm{kNm}}{3.065\ \mathrm{m}} - \frac{2333\ \mathrm{kN} \cdot (0.5 \cdot 4\ \mathrm{m} - 0.435\ \mathrm{m}}{3.065\ \mathrm{m}} \\ &= -181\ \mathrm{kN} \end{aligned}$$

If $T_{assumed} \approx T_{computed}$ no more iterations are necessary, otherwise $T_{assumed}$ is modified and more iterations are computed.

Depending on the eventual tension or compression, the required reinforcement area is computed and a sufficient amount of reinforcement is selected.

At this point a selection of reinforcement diameter, ϕ_v, is made. The area required, $A_{s,v}$, is then compared to the reinforcement area, $A_{\phi,v}$, which ultimately give the required number of reinforcement bars, n. It is important to bear in mind that in the case of compression (in the entire section) or low tension force, the minimum requirement, ρ_v for vertical reinforcement is used. If this is the case then $A_{s,v}$ is replaced by $A_{v,min} = \rho_v \cdot A_c$. Where $A_c = b_w \cdot l_c$ is the confined concrete area, where l_c is the length of the critical area.

$$\rho_{v,DCM} = 0.5\ \%$$

Required total reinforcement area is then given by:

$$A_{s,v} = \frac{T_{computed}}{f_{yd}}$$

In this specific case the entire section is subjected to compression, this means that the boundary of the wall must be designed for the minimum reinforcement requirement according to the standards.

Length of critical zone:

$$\begin{aligned} l_c &= \max(0.15 \cdot l_w, 1.5 \cdot b_w) \\ &= \max(0.15 \cdot 4\text{ m}, 1.5 \cdot 0.200\text{ m}) \\ &= 0.600\text{ m} \end{aligned}$$

$$\begin{aligned} A_{v,min} &= \rho_{v,DCM} \cdot (b_w \cdot l_c) \\ &= 0.5\ \% \cdot (0.200\text{ m} \cdot 0.600\text{ m}) \\ &= 600\text{ mm}^2 \end{aligned}$$

The vertical reinforcement diameter is selected to $\phi_v = 12$ mm. The area of the selected rebar is:

$$\begin{aligned} A_{\phi,v} &= \frac{\pi \cdot \phi_v^2}{4} \\ &= \frac{\pi \cdot (12\text{ mm})^2}{4} \\ &= 113\text{ mm}^2 \end{aligned}$$

Amount of ϕ_v needed in order to fulfil the requirement:

$$\begin{aligned} n &= \frac{A_{v,min}}{A_{\phi,v}} \\ &= \frac{600\text{ mm}^2}{113\text{ mm}^2} \\ &\approx 6 \end{aligned}$$

Total vertical reinforcement in wall boundary

$$\begin{aligned} A_{\phi,v,boundary,tot} &= n \cdot A_{\phi,v} \\ &= 6 \cdot 113\text{ mm}^2 \\ &= 679\text{ mm}^2 \end{aligned}$$

Spacing of the vertical reinforcement in the critical zone:

$$\begin{aligned} s_v &= \frac{l_c - c}{n/2} \\ &= \frac{0.600\text{ m} - 0.030\text{ m}}{6/2} \\ &= 190\text{ mm} \end{aligned}$$

This means that in each boundary of the wall $2 \times 3\phi 12$ is used for the detailing.

The connection of the dowels with the wall with the foundation is detailed the same as for the vertical boundary reinforcement, i.e. $2{\times}3\phi 12$ with dowel length set to 850 mm.

Shear reinforcement between separate walls

The resistance of a non-reinforced connections is given by:

$$\begin{aligned} V_{Rd,i} &= 0.03 \cdot f_{ctd} \cdot A_i + 0.5 \cdot f_{yd} \cdot A_s + 0.5 \cdot N_{DCM} \\ &= 0.03 \cdot 1.3 \text{ MPa} \cdot (4 \text{ m} \cdot 0.200 \text{ m}) + 0.5 \cdot 435 \text{ MPa} \cdot 0 \text{ mm}^2 + 0.5 \cdot 2333 \text{ kN} \\ &= 1198 \text{ kN} \end{aligned}$$

where A_i is the cross-section area of the connection.

This gives the shear force needed to be taken by the reinforcement (dowels):

$$V_s = \begin{cases} 0 & \text{if } V_{DCM} \leq V_{Rd,i} \\ V_{DCM} - V_{Rd,i} & \text{if } V_{DCM} \geq V_{Rd,i} \end{cases}$$

$$V_{DCM} < V_{Rd,i} \Rightarrow V_s = 0 \text{ kN}$$

In this case enough resistance is already provided and no extra shear reinforcement is required.

Otherwise, the selection of the dowels is made by the required reinforcement area:

$$A_s = \frac{V_s}{0.5 \cdot f_{yd}}$$

In seismic load situations, reinforcement that takes shear force is placed in the web of the wall and required tension/compression reinforcement are placed in the boundaries.

Strain check

In order to control if the reinforcement develops full strength capacity the strain is checked.

Compressive strain in the concrete from EC2:

$$\varepsilon_{c,2} = 0.2\ \%$$

$$\begin{aligned}\varepsilon_c &= \varepsilon_{c,2} \cdot (1 - \sqrt{1 - 0.75}) \\ &= 0.002 \cdot (1 - \sqrt{1 - 0.75}) \\ &= 0.001 = 0.1\ \%\end{aligned}$$

Strain in tensional reinforcement is given by:

$$\begin{aligned}\varepsilon_s &= \frac{\varepsilon_c (l_w - c_1 - x)}{x} \\ &= \frac{0.001 \cdot (4\ \text{m} - 0.500\ \text{m} - 1.23\ \text{m})}{1.23\ \text{m}} \\ &= 0.0019 = 1.9\ \text{‰}\end{aligned}$$

This value is then compared to $\varepsilon_{sy,d} = 2.5$ ‰ and $\varepsilon_{cu} = 30$ ‰. In order for the reinforcement to develop full strength capacity:

$$\begin{array}{ccc}\varepsilon_{sy,d} < & \varepsilon_s < & \varepsilon_{cu} \\ 2.5\ \text{‰} < & 1.9\ \text{‰} < & 30\ \text{‰} \Rightarrow \text{OK!}\end{array}$$

Control of plastic hinge

Control of the reinforcement amount in the critical zone of the wall is established by comparing the reinforcement amount, $A_{\phi,v,boundary,tot}$, and the balanced reinforced cross-section, $A_{s,b}$.

Length from outermost fiber in the compression zone to the center of the reinforcement:

$$\begin{aligned}d_l &= l_w - 0.500\ \text{m} \\ &= 4\ \text{m} - 0.500\ \text{m} \\ &= 3.5\ \text{m}\end{aligned}$$

Material values for concrete and steel from the standards. Ultimate compressive strain in the concrete from EC2:

$$\varepsilon_{cu} = 3.5\ ‰$$

Strain in reinforcement steel:

$$\varepsilon_{sy,d} = 2.5\ ‰$$

In order to get balanced reinforced compression zone:

$$\begin{aligned}
\alpha_b &= \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{sy,d}} \\
&= \frac{0.0035}{0.0035 + 0.0025} \\
&= 0.583 = 58.3\ \%
\end{aligned}$$

The following equation for balanced reinforced cross-section is valid for concrete qualities $f_{ck} < 50$ MPa. Factor $\lambda_d = 0.8$.

$$\begin{aligned}
A_{s,b} &= \lambda_d \cdot \frac{f_{cd}}{f_{yd}} \cdot b_w \cdot d_l \cdot \alpha_b \\
&= 0.8 \cdot \frac{20\ \text{MPa}}{435\ \text{MPa}} \cdot 0.200\ \text{m} \cdot 3.5\ \text{m} \cdot 0.583 \\
&= 15027\ \text{mm}^2
\end{aligned}$$

In order for plastic hinge to occur in the critical zone:

$$\begin{aligned}
A_{s,b} &\gg A_{\phi,v,boundary,tot} \\
15027\ \text{mm}^2 &\gg 679\ \text{mm}^2 \Rightarrow \text{OK!}
\end{aligned}$$

The relationship indicates that the wall is strongly under reinforced, which is necessary in order for plastic hinges to occur.

The reinforcement in compression must be secured against buckling and this is done by introducing confinement hoops in the critical zone.

Vertical reinforcement in the web of the wall

The vertical reinforcement is designed according to the standards in EC2, which in this case means that the minimum reinforcement applies. The reason is that the vertical reinforcement in the boundaries are designed to take the moment effect on the wall.

Minimum vertical reinforcement criteria:

$$\rho_{v,min} = 0.2\ \%$$

This gives the minimum required vertical reinforcement in the wall:

$$\begin{aligned} A_{v,min} &= \rho_{v,min} \cdot (l_w - 2 \cdot l_c) \cdot b_w \\ &= 0.002 \cdot (4\ \text{m} - 2 \cdot 0.600\ \text{m}) \cdot 0.200\ \text{m} \\ &= 1120\ \text{mm}^2 \end{aligned}$$

Selection of the vertical reinforcement in the web is $\phi_v = 10$ mm. The area of a single bar is:

$$\begin{aligned} A_{\phi,v} &= \frac{\pi \cdot \phi_v^2}{4} \\ &= \frac{\pi \cdot (10\ \text{mm})^2}{4} \\ &= 79\ \text{mm}^2 \end{aligned}$$

Amount of rebars in the web of the wall:

$$\begin{aligned} n &= \frac{A_{v,min}}{A_{\phi,v,web}} \\ &= \frac{1120\ \text{mm}^2}{79\ \text{mm}^2} \\ &\approx 16 \end{aligned}$$

EC2 also prescribes the maximum spacing for the vertical bars to:

$$\begin{aligned} s_{v,max} &\leq \min(3 \cdot b_w, 400\ \text{mm}) \\ &= \min(3 \cdot 0.200\ \text{mm}, 400\ \text{mm}) \\ &= 400\ \text{mm} \end{aligned}$$

Spacing between the vertical rebars in the web of the wall:

$$\begin{aligned} s_v &= \frac{l_w - 2 \cdot l_c}{n/2} \\ &= \frac{4\ \text{m} - 2 \cdot 0.600\ \text{m}}{16/2} \\ &\approx 350\ \text{mm} < 400\ \text{mm} \Rightarrow \text{OK!} \end{aligned}$$

This means that in the web of the wall $2 \times 8\phi 10 s 350$ is used for the detailing.

Horizontal reinforcement in wall (shear reinforcement)

The recommended amount of horizontal reinforcement is designed and selected to be:

Minimum horizontal reinforcement:

$$\begin{aligned}\rho_{h,min} &= \max(0.1\ \%, 0.25 \cdot \rho_v) \\ &= \max(0.1\ \%, 0.25 \cdot 0.002) \\ &= 0.001 = 0.1\ \%\end{aligned}$$

$$\begin{aligned}A_{h,min} &= \rho_{h,min} \cdot h_s \cdot b_w \\ &= 0.001 \cdot 3.2\ \mathrm{m} \cdot 0.200\ \mathrm{m} \\ &= 640\ \mathrm{mm}^2\end{aligned}$$

EC2 also prescribes the maximum spacing for the horizontal bars to:

$$s_h \leq 400\ \mathrm{mm}$$

The horizontal reinforcement must consider the actions impacting the wall, i.e. the lateral forces. This means that more reinforcement than the minimum might be needed.

According to EC8:

$$\begin{aligned}k &= \min\left(1 + \sqrt{\frac{200\ \mathrm{mm}}{d}}, 2\right) \\ &= \min\left(1 + \sqrt{\frac{200\ \mathrm{mm}}{164\ \mathrm{mm}}}, 2\right) \\ &= 2\end{aligned}$$

Vertical reinforcement content:

$$\begin{aligned}\rho_v &= \min\left(\frac{A_{\phi,v,boundary,tot} + A_{\phi,v,web,tot}}{l_w \cdot d}, 0.002\right) \\ &= \min\left(\frac{679\ \mathrm{mm}^2 + 1257\ \mathrm{mm}^2}{4\ \mathrm{m} \cdot 0.164\ \mathrm{m}}, 0.002\right) \\ &= 0.002 = 0.2\ \%\end{aligned}$$

Factors according to EC8:

$$k_1 = 0.15 \qquad k_2 = 0.15$$

$$\begin{aligned} C_{Rd,c} &= \frac{k_2}{\gamma_c} \\ &= \frac{0.15}{1.5} \\ &= 0.1 \end{aligned}$$

$$\begin{aligned} v_{min} &= 0.035 \cdot k^{(3/2)} \cdot \sqrt{f_{ck}} \\ &= 0.035 \cdot 2^{(3/2)} \cdot \sqrt{30\ \mathrm{MPa}} \\ &= 0.542 \end{aligned}$$

Limitation of compression strain:

$$\begin{aligned} \sigma_{cp} &= \min\left(\frac{N_{DCM}}{b_w \cdot l_w}, 0.2 \cdot f_{cd}\right) \\ &= \min\left(\frac{2333\ \mathrm{kN}}{0.200\ \mathrm{m} \cdot 4\ \mathrm{m}}, 0.2 \cdot 20\ \mathrm{MPa}\right) \\ &= 2.92\ \mathrm{MPa} \end{aligned}$$

Shear resistance of the wall:

$$\begin{aligned} V_{Rd,c,V} &= C_{Rd,c} \cdot k \cdot (100 \cdot \rho_v \cdot f_{ck})^{(1/3)} \cdot l_w \cdot d \\ &= 0.1 \cdot 2 \cdot (100 \cdot 0.002 \cdot 30\ \mathrm{MPa})^{(1/3)} \cdot 4\ \mathrm{m} \cdot 0.164\ \mathrm{m} \\ &= 14.79\ \mathrm{kN} \end{aligned}$$

Resistance contribution of axial force:

$$\begin{aligned} V_{Rd,c,N} &= k_1 \cdot \sigma_{cp} \cdot l_w \cdot d \\ &= 0.15 \cdot 2.92\ \mathrm{MPa} \cdot 4\ \mathrm{m} \cdot 0.164\ \mathrm{m} \\ &= 286.9\ \mathrm{kN} \end{aligned}$$

Lower limit for the shear resistance:

$$\begin{aligned} V_{Rd,c,min} &= v_{min} \cdot l_w \cdot d \\ &= 0.542 \cdot 4\ \mathrm{m} \cdot 0.164\ \mathrm{m} \\ &= 355.7\ \mathrm{kN} \end{aligned}$$

Lateral resistance without horizontal reinforcement:

$$\begin{aligned} V_{Rd,c} &= \max(V_{Rd,c,V} + V_{Rd,c,N}, V_{Rd,c,min} \cdot d \\ &= \max(14.79\text{ kN} + 286.9\text{ kN}, 355.7\text{ kN} \cdot 0.164\text{ m} \\ &= 355.7\text{ kN} \end{aligned}$$

Force that has to be horizontal reinforcement:

$$V_s = \begin{cases} 0 & \text{if } V_{DCM} \leq V_{Rd,c} \\ V_{DCM} - V_{Rd,c} & \text{if } V_{DCM} \geq V_{Rd,c} \end{cases}$$

$$\begin{aligned} V_{DCM} &\geq V_{Rd,c} \\ 885\text{ kN} &> 355.7\text{ kN} \\ \Rightarrow V_s &= V_{DCM} - V_{Rd,c} \\ &= 885\text{ kN} - 355.7\text{ kN} \\ &= 529.2\text{ kN} \end{aligned}$$

Required area of the horizontal reinforcement:

$$\begin{aligned} A_{s,h} &= \sqrt{3} \cdot \frac{V_s}{f_{yd}} \\ &= \sqrt{3} \cdot \frac{529.2\text{ kN}}{435\text{ MPa}} \\ &= 2108\text{ mm}^2 \end{aligned}$$

Selection of the horizontal reinforcement in the web and boundary is $\phi_h = 8$ mm. The area of a single bar is:

$$\begin{aligned} A_{\phi,h} &= \frac{\pi \cdot \phi_h^2}{4} \\ &= \frac{\pi \cdot (8\text{ mm})^2}{4} \\ &= 50\text{ mm}^2 \end{aligned}$$

Amount of rebars in the web and boundary of the wall:

$$\begin{aligned} n &= \frac{A_{s,h}}{A_{\phi,h}} \\ &= \frac{2108\text{ mm}^2}{50\text{ mm}^2} \\ &\approx 42 \end{aligned}$$

Total area of the horizontal reinforcement bars:

$$\begin{aligned} A_{\phi,h,tot} &= n \cdot A_{\phi,h} \\ &= 42 \cdot 50 \text{ mm}^2 \\ &= 2111 \text{ mm}^2 \end{aligned}$$

Spacing of the horizontal reinforcement:

$$\begin{aligned} s_h &= \frac{h_s}{n/2} \\ &= \frac{3.2 \text{ m}}{42/2} \\ &\approx 150 \text{ mm} < 400 \text{ mm} \Rightarrow \text{OK!} \end{aligned}$$

This means that at each side of the wall $2 \times 21\phi 8s150$ is used for the detailing.

In order to check if special detailing is needed for the boundaries of the wall in DCM-design the normalized axial load is checked:

$$\begin{aligned} \nu_d &= \frac{N_{DCM}}{b_w \cdot l_w \cdot f_{cd}} \\ &= \frac{2333 \text{ kN}}{0.200 \text{ m} \cdot 4 \text{ m} \cdot 20 \text{ MPa}} \\ &= 0.146 \end{aligned}$$

If $\nu_d < 0.15$ the DCL-design of the horizontal reinforcement applies for the DCM-design, i.e. no extra hoops are needed in the wall boundaries.

Table 5.10: *Compilation of data for detailing of wall in DCM.*

Type	Location	ϕ [mm]	Amount	s [mm]	Anchorage [mm]
Vertical	Boundary	12	6	190	
Dowel	Boundary	12	6	190	850
Vertical	Web	10	16	350	
Shear dowel	Mid-span	-	-	-	-
Horizontal	Boundary+web	8	42	150	

Detailing in the critical zone (only for DCM if $\nu_d > 0.15$)

For the model analyzed, this part is not applicable, i.e. no extra detailing in the critical zone is necessary.

Calculation of the curvature ductility is computed with the fundamental period T_1 respectively ductility factor used in analysis q_0.

$$\mu_\phi = \begin{cases} 2q_0 - 1 & \text{if } T_1 \geq T_C \\ 1 + 2(q_0 - 1)\dfrac{T_C}{T_1} & \text{if } T_1 < T_C \end{cases}$$

Sufficient volume of the hoops are computed. The following expression must be fulfilled in order for the amount of hoop-reinforcement to be sufficient. Volumetric ratio of hoops in boundaries:

$$\alpha\omega_{wd} \geq 30\mu_\phi(\nu_d + \omega_v)\varepsilon_{sy,d}\frac{b_c}{b_0} - 0.035 \tag{5.4}$$

Normalized axial load:

$$\nu_d = \frac{N_{Ed}}{b_w \cdot l_w \cdot f_{cd}}$$

Ratio between vertical reinforcement and concrete area:

$$\rho_v = \frac{A_{s,v}}{b_w \cdot l_w}$$

$$\omega_v = \rho_v \cdot \frac{f_{yd}}{f_{cd}}$$

$$b_0 = b_w - 2c_{conf} - \phi_i$$

The length of the critical zone, i.e. length of the hoop, in the section is computed from:

$$l_c = max(x \cdot (1 - \varepsilon_{cu2}/\varepsilon_{cu2,c}), 0.15 \cdot l_w, 1.5 \cdot b_w)$$

Strain limit for hoop reinforced concrete:

$$\varepsilon_{cu2,c} = 0.0035 + 0.1 \cdot \alpha\omega_{wd}$$

Choice of hoop reinforcement area, ϕ_w, is done. A spacing distance (horizontal) is assumed, s_w. Accumulated volume of the hoops per 1 m (b_i denotes the spacing between the cross ties in the hoops):

$$V_{s,vol} = A_{\phi,w} \cdot \frac{2 \cdot (l_c + b_0) + 2 \cdot 2(b_i + b_0)}{s_w}$$

Volume of confined concrete:

$$V_{c,vol} = l_c \cdot b_0$$

$$\alpha_n = 1 - \frac{8 \cdot s_w^2 + 2 \cdot b_0^2}{6 \cdot b_0 \cdot h_0}$$

$$\alpha_s = 1 - \frac{8 \cdot s_w^2 + 2 \cdot b_0^2}{6 \cdot b_0 \cdot h_0}$$

The confinement effectiveness factor:

$$\alpha = \alpha_n \cdot \alpha_s \tag{5.5}$$

$$\omega_{wd} = \frac{V_{s,vol} \cdot f_{yd}}{V_{c,vol} \cdot f_{cd}} \tag{5.6}$$

Eqs. (5.5 – 5.6) are ultimately checked against the inequality in Eq. (5.4). If it is fulfilled the selected hoops and spacing are sufficient.

5.6 Economical assessment and comparison

The retail price for reinforcement steel in the total cost assessment are retrieved from suppliers in Sweden (BE Group) respectively Norway (Norsk Stål), see Table 5.11. The prices refer to reinforcement of steel quality *B500* and length of 6 m.

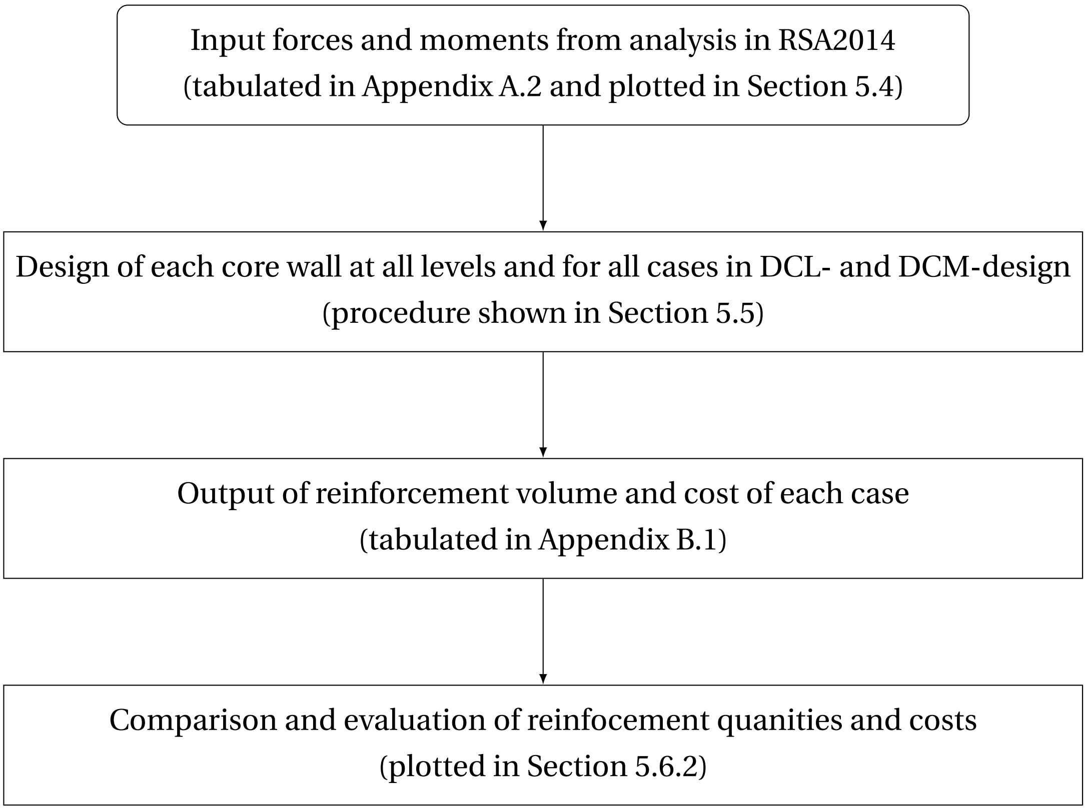
Table 5.11: *Prices of reinforcement per* kg *(BE Group, 2014) and (Norsk Stål, 2014).*

	BE Group	Norsk Stål
ϕ [mm]	Price [SEK/kg]	Price [NOK/kg]
6	15,15	16,50
8	12,03	13,95
10	11,77	13,25
12	11,71	13,25
16	11,62	12,85
20	11,62	12,85
25	11,12	13,00

The volume of the reinforcement elements is calculated by multiplying the total length respectively the rebar's cross-section area. For the conversion of the volume to weight of each element, the density of steel is set to $\rho_s = 7850$ kg/m^3.

5.6.1 Methodology

The following flowchart describes the methodology for computing the data necessary for the final cost comparison.



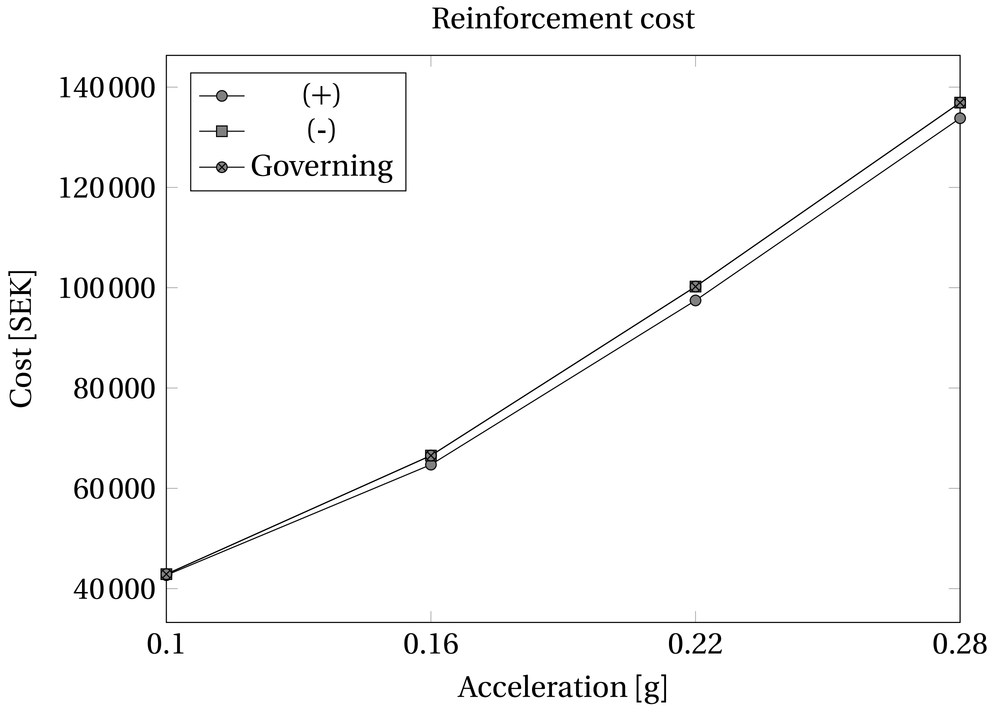
5.6.2 Quantitative evaluation

The following graphs presents the cost development of the reinforcement in the entire shaft (i.e. all four core walls) depending on ground acceleration and ductility class.

In order to get the total cost of the entire shaft the calculation procedures presented in section 5.5 are done at all levels in the core walls for all acceleration cases using *Microsoft Excel*.

Output data for the entire shaft is presented in Appendix B.1.

Figures 5.15 and 5.16 illustrates the reinforcement cost development for DCL- and DCM-design respectively.



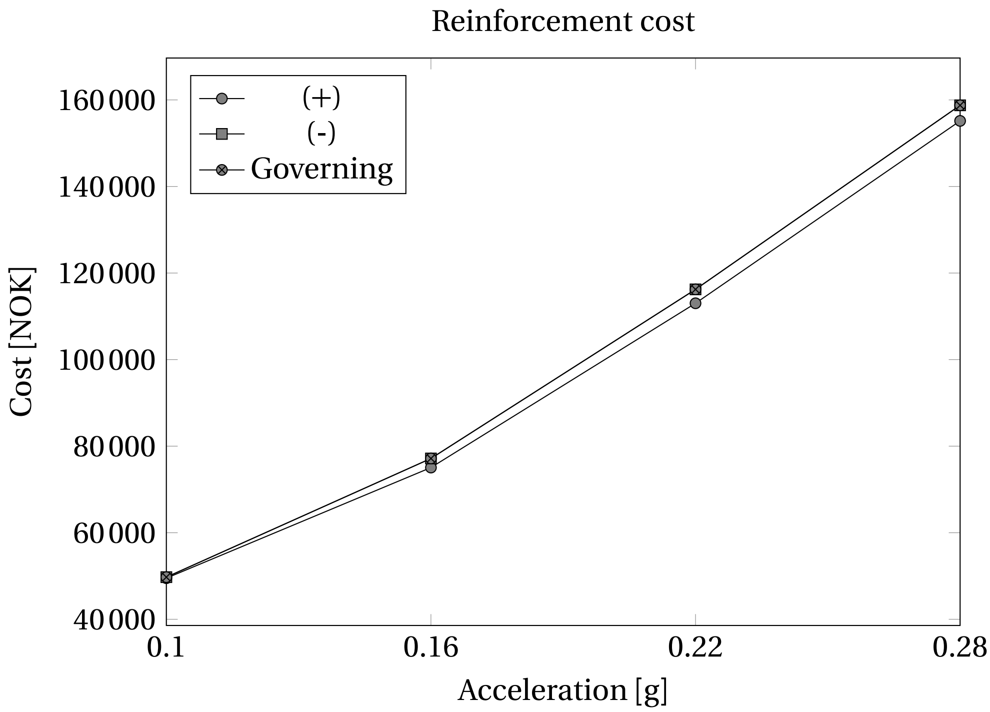
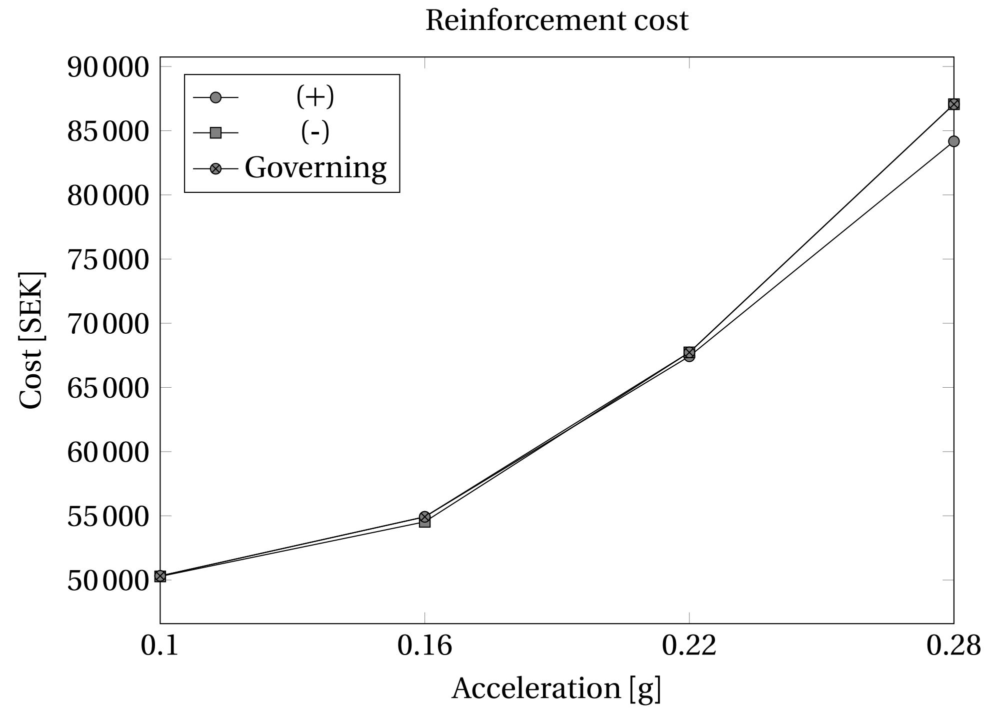


Figure 5.15: *[DCL: $q = 1.5$] Total cost development of reinforcement in DCL with increasing $a_g \cdot S$. (+) and (-) denotes positive respectively negative direction of seismic load in the load combination for analysis.*



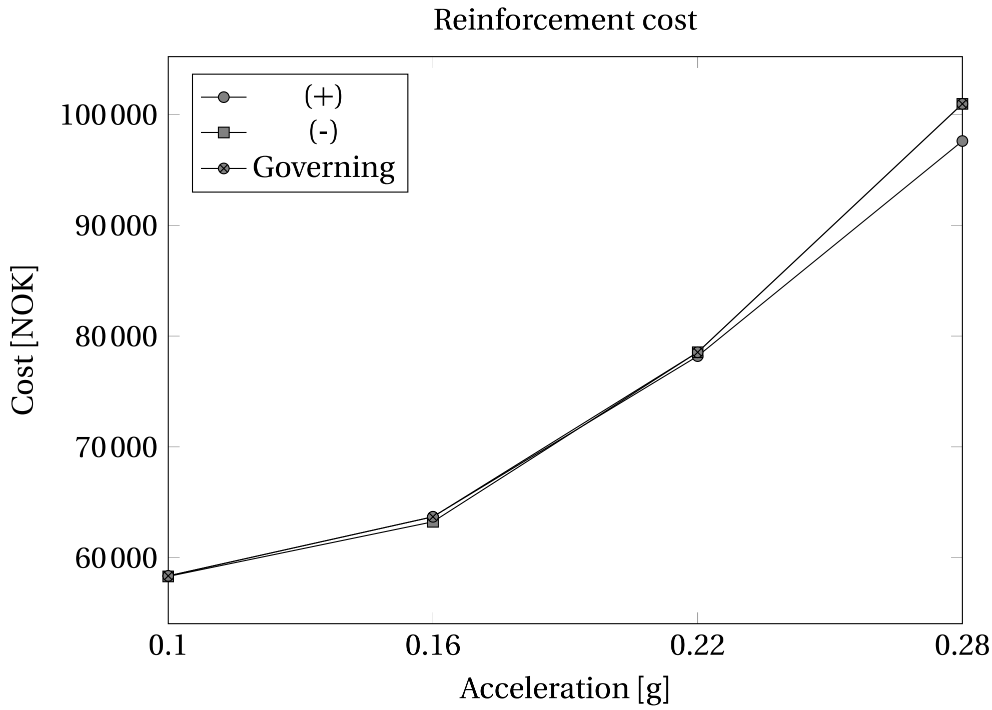


Figure 5.16: *[DCM: $q = 3.0$] Total cost development of reinforcement in DCM with increasing $a_g \cdot S$. (+) and (-) denotes positive respectively negative direction of seismic load in the load combination for analysis.*

Relationship between the selection of ductility class

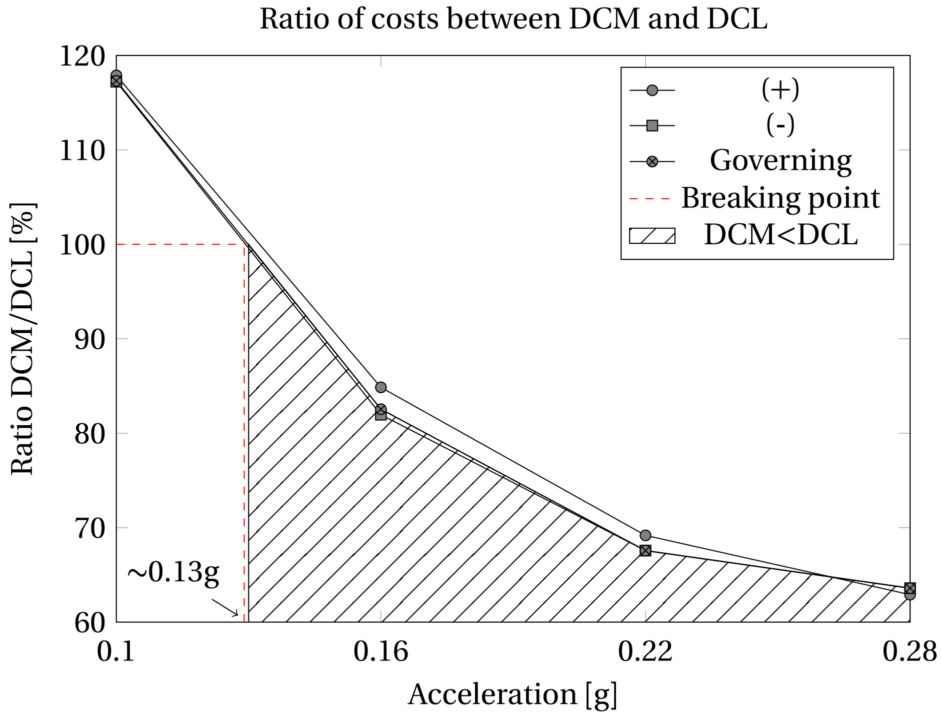


Figure 5.17: *Ratio of total reinforcement costs in DCL- and DCM-design. (+) and (-) denotes positive respectively negative direction of seismic load in the load combination for analysis.*

The graph in Figure 5.17 indicates that the cost of reinforcement in DCM-design is lower than in DCL-design when *Ratio DCM/DCL* < 100 %.

The governing curve in Figure 5.17 can be approximated to the second order polynomial $y = 21.38x^2 - 11.06x + 2.06$, where y $[0 \leq y \leq 1]$ is equal to the ratio between the cost in DCM-design divided by DCL-design and x $[0.10\text{ g} \leq x \leq 0.28\text{ g}]$ is the design ground acceleration subjected to the structure for design and detailing.

A limit-value for the design ground acceleration can be estimated based on the trendline for the cost-relationship.

$$
\begin{aligned}
y = 100\% = 1 \text{ (inserted in) } \rightarrow y &= 21.38x^2 - 11.06x + 2.06 \Rightarrow \\
0 &= x^2 - 0.517x + 0.05 \Rightarrow \\
x &= 0.259 \pm \sqrt{0.067 - 0.05} \Rightarrow \\
x &= 0.259 \pm 0.132 \Rightarrow \\
x &= 0.127 \approx 0.13
\end{aligned}
$$

This implies that the breaking point for when the DCM-design becomes more profitable than the DCL-design in the matter of reinforcement cost is approximately when the design ground acceleration is $a_g \cdot S > 0.13$ g.

$$\text{Lowermost reinforcement cost*} = \begin{cases} DCL & \text{if } a_g \cdot S < 0.13 \text{ g} \\ DCM & \text{if } a_g \cdot S > 0.13 \text{ g} \end{cases}$$

* Note that the breaking point is based on design calculations where four different discrete values of design ground accelerations are used, which may imply that it may differ if design is conducted with more than four acceleration values.

Figures 5.18 and 5.19 shows the cost difference between the DCL- and DCM-design in Swedish kronor (SEK) and Norwegian kroner (NOK) respectively.

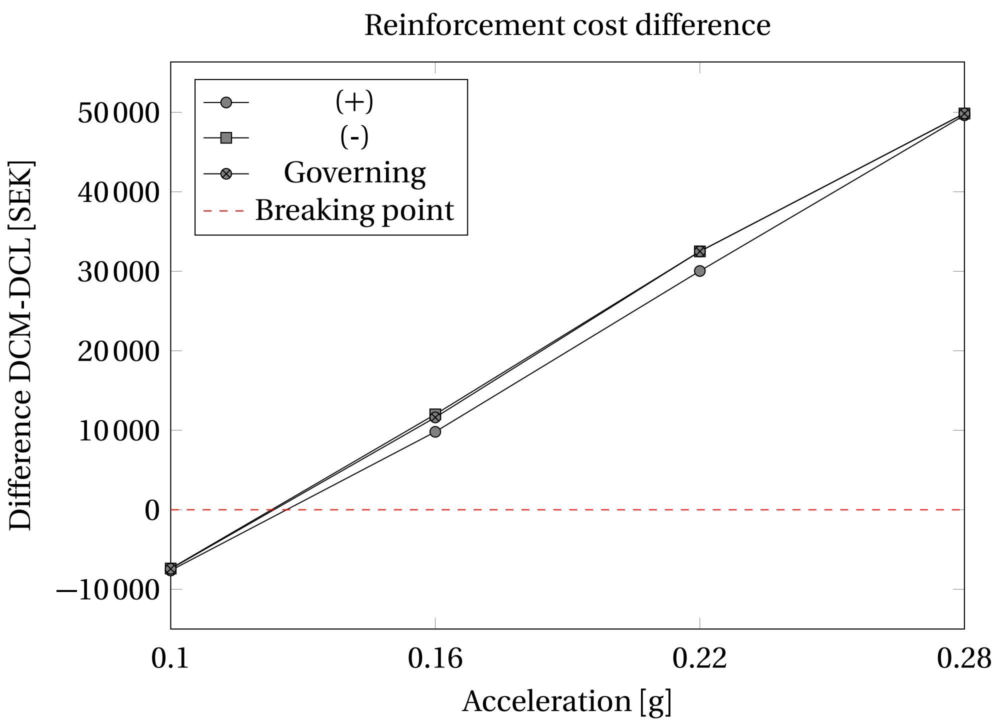


Figure 5.18: *Difference between the total reinforcement costs for DCL- and DCM-design in Swedish kronor. (+) and (-) denotes positive respectively negative direction of seismic load in the load combination for analysis.*

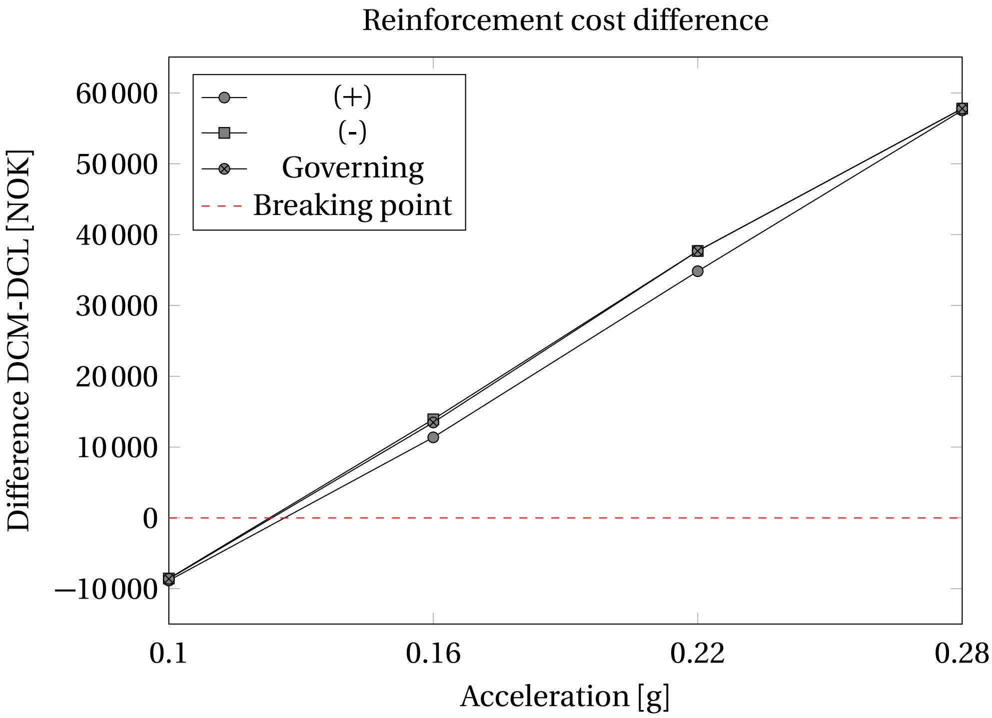


Figure 5.19: *Difference between the total reinforcement costs in DCL- and DCM-design for Norwegian kronor. (+) and (-) denotes positive respectively negative direction of seismic load in the load combination for analysis.*

Seismic zones where DCM-design in beneficial

In order for the design ground acceleration to exceed the breaking point (0.13 g) defined in the previous chapter, the importance class of the building has to be II or higher. The following tables define the zones where DCM-design is more cost effective than the DCL-design.

To find out if the location of the building in question might give a more cost effective design in DCM, the isocurves on the seismic zone maps (Figure 4.4) are compared to a_{g40Hz} for the relevant importance class (Tables 5.12, 5.13 and 5.14).

Table 5.12: *Design ground acceleration, $a_g \cdot S$, for buildings in importance class II ($\gamma_I = 1.0$). Highlighted acceleration values indicates seismic zones above the breaking point ($a_g \cdot S \geq 0.13$ g).*

			Ground type				
		$\gamma_I = 1.0$	A	B	C	D	E
a_{g40Hz} [m/s^2]*	a_{g40Hz} [g]**	a_{gR} [g]***	$a_g \cdot S$ [g]****				
0.95	0.097	0.077	0.08	0.10	0.11	0.12	0.13
1	0.102	0.082	0.08	0.11	0.11	0.13	0.13
1.05	0.107	0.086	0.09	0.11	0.12	0.13	0.14

Table 5.13: *Design ground acceleration, $a_g \cdot S$, for buildings in importance class III ($\gamma_I = 1.4$). Highlighted acceleration values indicates seismic zones above the breaking point ($a_g \cdot S \geq 0.13$ g).*

			Ground type				
		$\gamma_I = 1.4$	A	B	C	D	E
a_{g40Hz} [m/s^2]*	a_{g40Hz} [g]**	a_{gR} [g]***	$a_g \cdot S$ [g]****				
0.7	0.071	0.057	0.08	0.10	0.11	0.12	0.13
0.75	0.076	0.061	0.09	0.11	0.12	0.13	0.14
0.8	0.082	0.065	0.09	0.12	0.13	0.14	0.15
0.85	0.087	0.069	0.10	0.13	0.14	0.15	0.16
0.9	0.092	0.073	0.10	0.13	0.14	0.16	0.17
0.95	0.097	0.077	0.11	0.14	0.15	0.17	0.18
1	0.102	0.082	0.11	0.15	0.16	0.18	0.19
1.05	0.107	0.086	0.12	0.16	0.17	0.19	0.20

Table 5.14: *Design ground acceleration, $a_g \cdot S$, for buildings in importance class IV ($\gamma_I = 2.0$). Highlighted acceleration values indicates seismic zones above the breaking point ($a_g \cdot S \geq 0.13$ g).*

		$\gamma_I = 2.0$	Ground type A	B	C	D	E
a_{g40Hz} [m/s^2]*	a_{g40Hz} [g]**	a_{gR} [g]***	$a_g \cdot S$ [g]****				
0.5	0.051	0.041	0.08	0.11	0.11	0.13	0.13
0.55	0.056	0.045	0.09	0.12	0.13	0.14	0.15
0.6	0.061	0.049	0.10	0.13	0.14	0.15	0.16
0.65	0.066	0.053	0.11	0.14	0.15	0.16	0.17
0.7	0.071	0.057	0.11	0.15	0.16	0.18	0.19
0.75	0.076	0.061	0.12	0.16	0.17	0.19	0.20
0.8	0.082	0.065	0.13	0.17	0.18	0.20	0.22
0.85	0.087	0.069	0.14	0.18	0.19	0.21	0.23
0.9	0.092	0.073	0.15	0.19	0.21	0.23	0.24
0.95	0.097	0.077	0.15	0.20	0.22	0.24	0.26
1	0.102	0.082	0.16	0.21	0.23	0.25	0.27
1.05	0.107	0.086	0.17	0.22	0.24	0.27	0.28

* Values are extracted from the map of the seismic zones (Figure 4.4 on page 50)

** Unit conversion into gravitational acceleration of the first column.

*** Computed according to Eq. (4.1) on page 49.

**** Computed according to Eq. (4.2) with the amplification factors (S) of ground types A-E defined in Table 4.3 on page 48.

Tables 5.12–5.14 indicates the regions in Norway where DCM-design of the case study are more cost beneficial than DCL-design. The regions are illustrated in Figure 5.20 for each importance class.

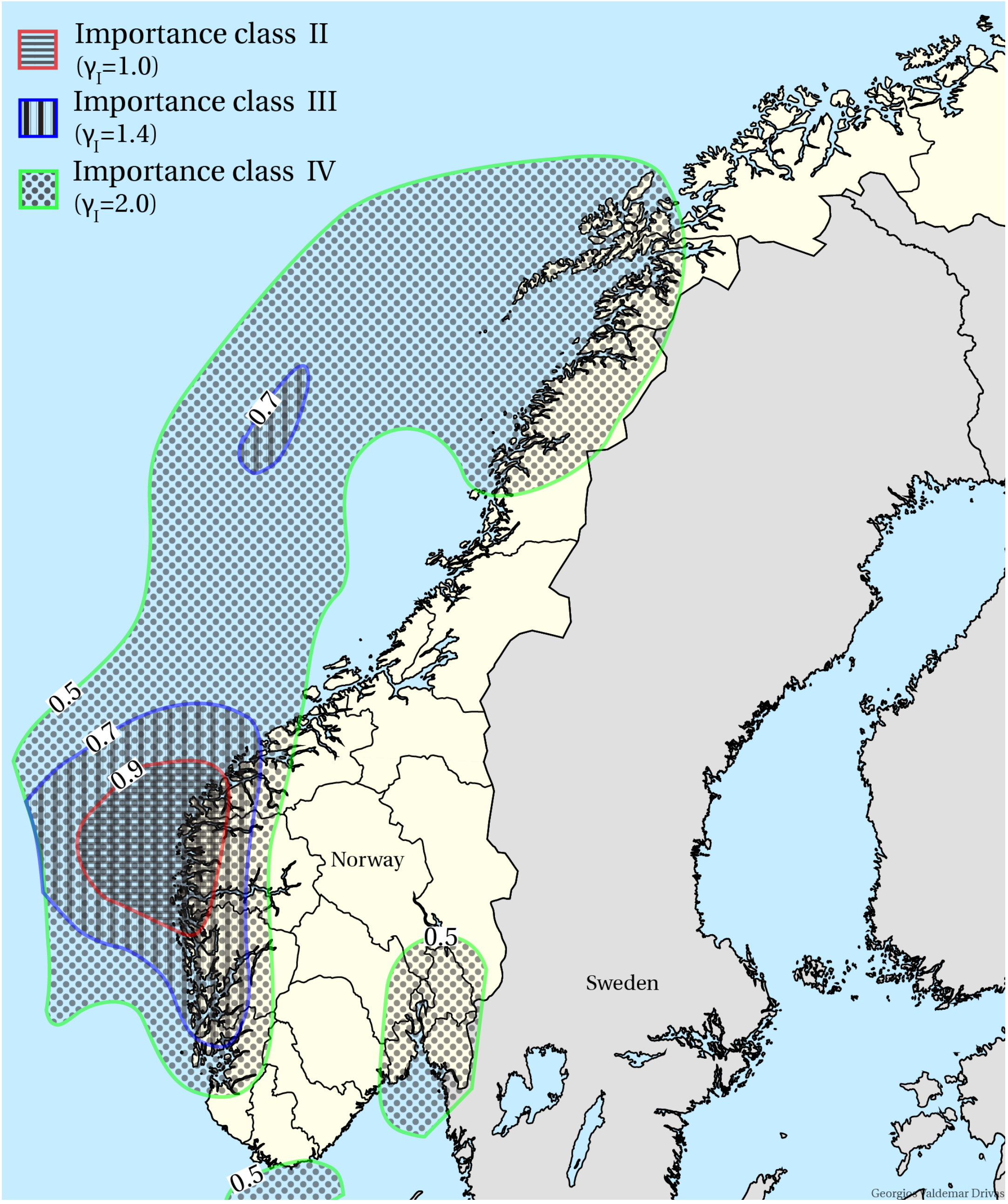


Figure 5.20: *Regions in Norway where DCM-design is more cost efficient than DCL-design. The illustration is based on the Norwegian seismic zones represented Figure 4.4 and the results listed in Tables 5.12–5.14 referring to the seismic hazard parameter a_{g40Hz} [m/s^2].*

5.6.3 Qualitative evaluation

Because the normalized axial force is lower than 15% the detailing of the core walls have the same lay-out in both DCL- and DCM-design. The following schematic drawings illustrates the reinforcement set-up of the wall (the visualization of the detailing is drawn in *Autodesk Revit Structure 2014*).

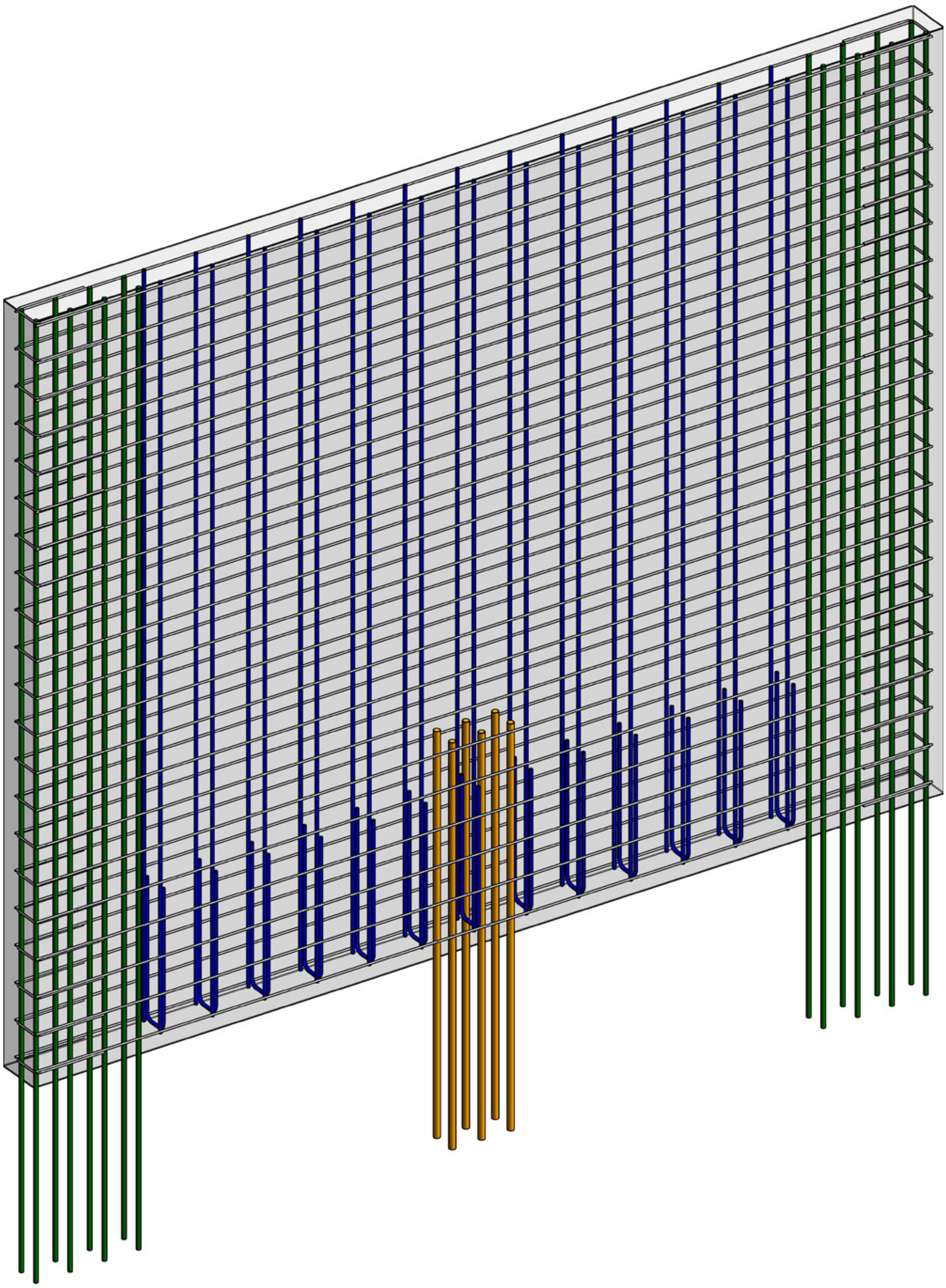
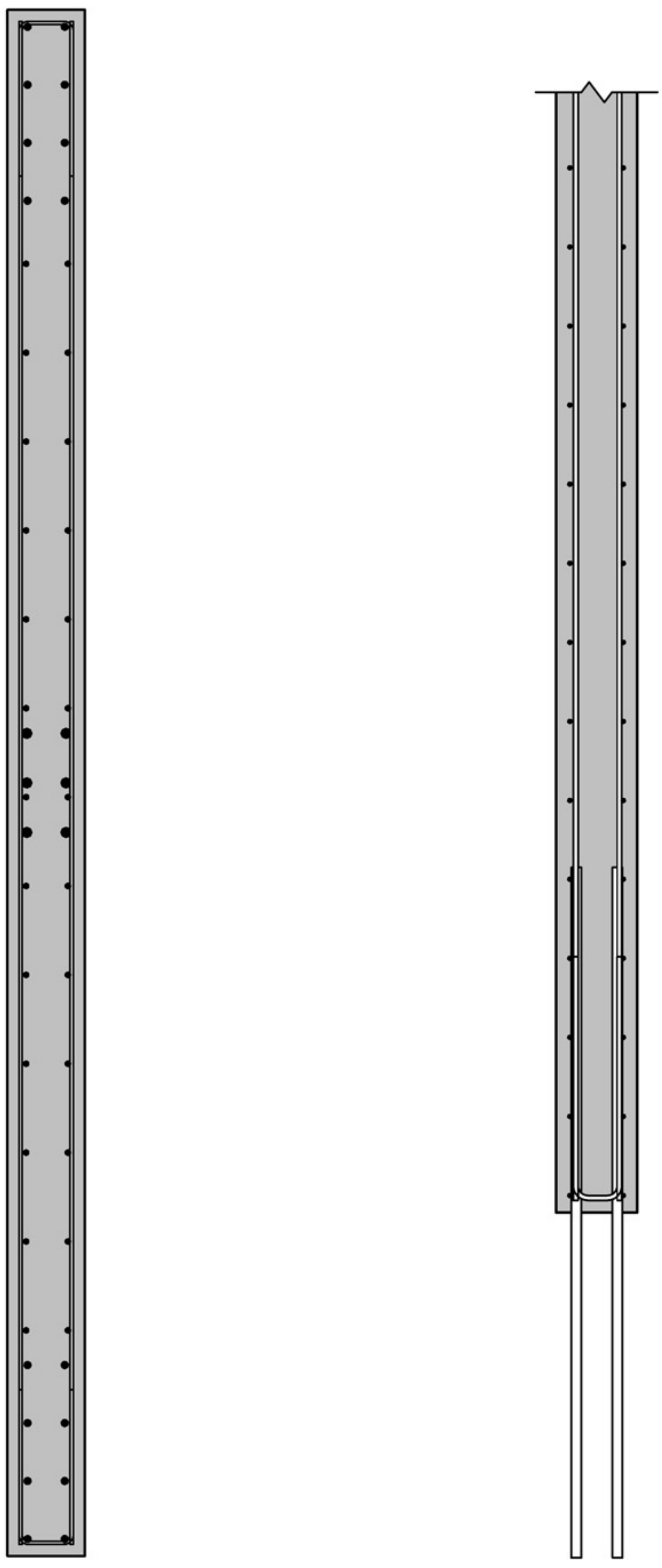


Figure 5.21: *Schematic detailing of core wall in 3D.*



(a) *Plan view.* **(b)** *Section view.*

Figure 5.22: *Schematic detailing of core wall.*

6
Conclusions

Based on the case study examined the general conclusion drawn regarding the reinforcement content and ultimately the total cost of the required amount of steel in the wall is that DCM-design is more beneficial than the DCL-design when the design ground acceleration is increasing. A breaking point ($a_g \cdot S \approx 0.13$ g) for the most beneficial design choice is defined, which for example corresponds to a building structure of importance class IV on soil type D and located in the Oslo region. The map illustrated in Figure 5.20 implies that DCM-design for lower building importance classes is beneficial on the western Norwegian coastline.

The results demonstrate that when the design ground acceleration is between 0.10 g — 0.13 g the DCL-design is more beneficial than the DCM-design when it comes to material costs. This might be explained by the minimum reinforcement requirement prescribed in the standards and if the cross-section of the wall is entirely under compression. This is because the minimum reinforcement ratio in the DCL-design is 0.2 % and in the DCM-design, it is 0.5 %. This means that if the minimum reinforcements are governing in both designs, the DCL-design will give less reinforcement content than the DCM-design.

In the case study, the model gives a normalized axial force less than 15 %, which means that the detailing in DCL-design and DCM-design is similar. This implies that the practical assemblage of the wall reinforcement will be the same and no differences in assemblage difficulty in-situ or in the factory where the wall is built will be in question.

The DCM-design, according to theory, will give an elastoplastic structure, which is allowed to enter the plastic zone and maintain deformations after an earthquake event. This means that more energy will be dissipated, and ultimately the maximum acceleration response will be less than in the DCL-design, where an entirely elastic structure is designed. This means that lower lateral forces will be acting on both bearing elements (primary seismic elements) and non-structural elements (secondary seismic elements), which leads to less probability of damaging the interior etc.

Another advantage of the DCM-design in general is the lower lateral forces on the structure. In a precast building, the slabs, which are usually hollow-core slabs, need a layer of confinement concrete on top to act as a more monolithic element, which means that with higher forces a thicker layer of concrete and reinforcement must be applied in order to fulfill the requirements. Ultimately, this affects the effective story height, which might have an impact on the architectural design.

For the design and detailing process, it might be less time consuming for the structural engineer to design an totally elastic structure according to EC2 keeping in mind that this is a more familiar and therefore faster procedure to carry out. This is despite the fact that in this specific case, the prerequisites and requirements result in two designs without major differences due to the normalized axial forces. Extra detailing is necessary if the normalized axial forces are higher than 15%, which then leads to extra steps in the detailing procedures and ultimately a more complex detailing and assemblage when building the wall.

Suggestions of further research

In order to examine the profitability further, an interesting topic would be to analyze the design and detailing depending on the normalized axial force, i.e. examine the study case with this as a changing parameter in order to find out what is occurring with the reinforcement content when the extra DCM-detailing is required to fulfill the additional criteria in EC8.

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A
Analysis Data

In this appendix the modal analysis data from RSA2014 are listed.

The direction of the seismic input for the load combination in the analysis is symbolized with either (+) or (-). This means that two load combinations are analyzed for each acceleration case.

A.1 Modal analysis

Table A.1: *Modal analysis.*

Mode	Period [s]	Current mass UX [t]	Current mass UY [t]
1	**0.79**	**29.84**	**0.07**
2	0.66	44.58	0.15
3	0.56	0.02	67.71
4	0.22	14.54	0.02
5	0.19	1.77	0.36
6	0.14	3.34	0.13
7	0.13	0.09	19.43
8	0.13	1.20	0.84
9	0.12	0.71	0.01

Continued on next page

Table A.1 – *Continued from previous page.*

Mode	Period [s]	Current mass UX [t]	Current mass UY [t]
10	0.12	0.00	0.57
11	0.12	0.13	0.00
12	0.12	0.05	0.00
13	0.12	0.00	0.00
14	0.11	0.81	0.03
15	0.11	0.26	0.01
16	0.10	0.48	0.00
17	0.10	0.00	0.00
18	0.10	0.00	0.01
19	0.10	0.00	0.00
20	0.10	0.00	0.00

Table A.2: *Modal analysis.*

Mode	Period [s]	Relative mass UX [%]	Relative mass UY [%]
1	0.79	29.84	0.07
2	0.66	74.42	0.21
3	0.56	74.44	67.92
4	0.22	88.98	67.94
5	0.19	90.75	68.30
6	0.14	94.09	68.44
7	0.13	94.18	87.87
8	0.13	95.39	88.71
9	0.12	96.10	88.72
10	0.12	96.10	89.28
11	0.12	96.23	89.28
12	0.12	96.28	89.29
13	0.12	96.28	89.29
14	0.11	97.10	89.31
15	0.11	97.35	89.32
16	0.10	97.83	89.33
17	0.10	97.83	89.33

Continued on next page

Table A.2 – *Continued from previous page.*

Mode	Period [s]	Relative mass UX [%]	Relative mass UY [%]
18	0.10	97.83	89.34
19	0.10	97.83	89.34
20	0.10	97.83	89.34

A.2 Force and moment results from RSA2014

WFX and WFY denotes the lateral force action in X- and Y-direction respectively. WMY and WMX denotes the moment action around the Y- and X-axis respectively. WFZ is the vertical static action in Z-direction.

A.2.1 DCL

Table A.3: *Force action on lower section of core wall 3 with DCL.*

			Case ($a_g \cdot S$)			
Core wall	Level	Action [kN]	1 (0.10 g)	2 (0.16 g)	3 (0.22 g)	4 (0.28 g)
CW3	+0 (+)	WFX	421	674	927	1180
CW3	+3,2 (+)	WFX	307	495	682	869
CW3	+6,4 (+)	WFX	197	321	444	567
CW3	+9,6 (+)	WFX	135	223	311	399
CW3	+12,8 (+)	WFX	120	201	282	364
CW3	+16 (+)	WFX	106	182	258	334
CW3	+19,2 (+)	WFX	45	86	128	169
CW3	+0 (-)	WFX	-421	-674	-926	-1179
CW3	+3,2 (-)	WFX	-315	-501	-687	-874
CW3	+6,4 (-)	WFX	-211	-332	-454	-575
CW3	+9,6 (-)	WFX	-157	-244	-332	-419
CW3	+12,8 (-)	WFX	-151	-232	-313	-395
CW3	+16 (-)	WFX	-145	-220	-294	-369
CW3	+19,2 (-)	WFX	-91	-131	-171	-211

Table A.4: *Force action on lower section of core wall 4 with DCL.*

			Case ($a_g \cdot S$)			
Core wall	Level	Action [kN]	1 (0.10 g)	2 (0.16 g)	3 (0.22 g)	4 (0.28 g)
CW4	+0 (+)	WFY	765	1227	1690	2154
CW4	+3,2 (+)	WFY	779	1225	1671	2118
CW4	+6,4 (+)	WFY	728	1137	1546	1955
CW4	+9,6 (+)	WFY	654	1013	1372	1731
CW4	+12,8 (+)	WFY	557	852	1148	1444
CW4	+16 (+)	WFY	428	643	858	1072
CW4	+19,2 (+)	WFY	245	349	452	555
CW4	+0 (-)	WFY	-779	-1245	-1711	-2177
CW4	+3,2 (-)	WFY	-757	-1205	-1652	-2100
CW4	+6,4 (-)	WFY	-700	-1111	-1521	-1932
CW4	+9,6 (-)	WFY	-618	-980	-1341	-1703
CW4	+12,8 (-)	WFY	-513	-811	-1109	-1407
CW4	+16 (-)	WFY	-375	-591	-807	-1022
CW4	+19,2 (-)	WFY	-188	-289	-390	-491

Table A.5: *Force action on lower section of core wall 5 with DCL.*

			Case ($a_g \cdot S$)			
Core wall	Level	Action [kN]	1 (0.10 g)	2 (0.16 g)	3 (0.22 g)	4 (0.28 g)
CW5	+0 (+)	WFY	627	1008	1389	1771
CW5	+3,2 (+)	WFY	663	1041	1418	1795
CW5	+6,4 (+)	WFY	626	972	1318	1665
CW5	+9,6 (+)	WFY	556	856	1155	1455
CW5	+12,8 (+)	WFY	481	733	985	1237
CW5	+16 (+)	WFY	391	588	785	982
CW5	+19,2 (+)	WFY	248	358	468	578
CW5	+0 (-)	WFY	-645	-1029	-1413	-1798
CW5	+3,2 (-)	WFY	-639	-1017	-1395	-1773
CW5	+6,4 (-)	WFY	-594	-941	-1288	-1636
CW5	+9,6 (-)	WFY	-524	-826	-1127	-1429
CW5	+12,8 (-)	WFY	-452	-706	-960	-1214
CW5	+16 (-)	WFY	-362	-560	-757	-954
CW5	+19,2 (-)	WFY	-220	-327	-434	-540

Table A.6: *Force action on lower section of core wall 6 with DCL.*

			Case ($a_g \cdot S$)			
Core wall	Level	Action [kN]	1 (0.10 g)	2 (0.16 g)	3 (0.22 g)	4 (0.28 g)
CW6	+0 (+)	WFX	862	1380	1898	2415
CW6	+3,2 (+)	WFX	777	1254	1732	2210
CW6	+6,4 (+)	WFX	651	1054	1457	1860
CW6	+9,6 (+)	WFX	515	838	1160	1483
CW6	+12,8 (+)	WFX	399	653	907	1161
CW6	+16 (+)	WFX	287	473	659	846
CW6	+19,2 (+)	WFX	131	221	312	402
CW6	+0 (-)	WFX	-874	-1398	-1921	-2445
CW6	+3,2 (-)	WFX	-822	-1304	-1785	-2267
CW6	+6,4 (-)	WFX	-707	-1116	-1525	-1934
CW6	+9,6 (-)	WFX	-581	-911	-1241	-1571
CW6	+12,8 (-)	WFX	-470	-732	-993	-1255
CW6	+16 (-)	WFX	-354	-546	-737	-929
CW6	+19,2 (-)	WFX	-184	-275	-365	-456

Table A.7: *Moment action on lower section of core wall 3 with DCL.*

			Case ($a_g \cdot S$)			
Core wall	Level	Action [kNm]	1 (0.10 g)	2 (0.16 g)	3 (0.22 g)	4 (0.28 g)
CW3	+0 (+)	WMY	2147	3569	4991	6413
CW3	+3,2 (+)	WMY	1080	1827	2574	3321
CW3	+6,4 (+)	WMY	650	1118	1586	2054
CW3	+9,6 (+)	WMY	606	1033	1460	1887
CW3	+12,8 (+)	WMY	540	915	1290	1664
CW3	+16 (+)	WMY	360	612	864	1116
CW3	+19,2 (+)	WMY	155	264	373	482
CW3	+0 (-)	WMY	-1550	-2348	-3146	-3944
CW3	+3,2 (-)	WMY	-897	-1344	-1790	-2236
CW3	+6,4 (-)	WMY	-793	-1204	-1614	-2024
CW3	+9,6 (-)	WMY	-710	-1091	-1472	-1853
CW3	+12,8 (-)	WMY	-511	-791	-1071	-1352
CW3	+16 (-)	WMY	-290	-460	-629	-798
CW3	+19,2 (-)	WMY	-74	-138	-202	-265

Table A.8: *Moment action on lower section of core wall 4 with DCL.*

			Case ($a_g \cdot S$)			
Core wall	Level	Action [kNm]	1 (0.10 g)	2 (0.16 g)	3 (0.22 g)	4 (0.28 g)
CW4	+0 (+)	WMX	3872	6889	9906	12923
CW4	+3,2 (+)	WMX	2635	4819	7003	9187
CW4	+6,4 (+)	WMX	1831	3444	5057	6669
CW4	+9,6 (+)	WMX	1227	2378	3528	4679
CW4	+12,8 (+)	WMX	743	1495	2246	2998
CW4	+16 (+)	WMX	342	737	1132	1526
CW4	+19,2 (+)	WMX	64	177	291	404
CW4	+0 (-)	WMX	-5002	-7297	-9592	-11888
CW4	+3,2 (-)	WMX	-3855	-5528	-7202	-8876
CW4	+6,4 (-)	WMX	-2894	-4075	-5257	-6439
CW4	+9,6 (-)	WMX	-2040	-2808	-3575	-4342
CW4	+12,8 (-)	WMX	-1255	-1661	-2066	-2471
CW4	+16 (-)	WMX	-622	-760	-899	-1037
CW4	+19,2 (-)	WMX	-315	-404	-492	-581

Table A.9: *Moment action on lower section of core wall 5 with DCL.*

			Case ($a_g \cdot S$)			
Core wall	Level	Action [kNm]	1 (0.10 g)	2 (0.16 g)	3 (0.22 g)	4 (0.28 g)
CW5	+0 +	WMX	3766	6574	9382	12191
CW5	+3,2 +	WMX	2580	4616	6651	8686
CW5	+6,4 +	WMX	1789	3279	4769	6258
CW5	+9,6 +	WMX	1181	2221	3261	4300
CW5	+12,8 +	WMX	695	1350	2006	2661
CW5	+16 +	WMX	319	653	987	1322
CW5	+19,2 +	WMX	88	189	290	391
CW5	+0 (-)	WMX	-4522	-6665	-8807	-10949
CW5	+3,2 (-)	WMX	-3497	-5049	-6601	-8153
CW5	+6,4 (-)	WMX	-2611	-3687	-4762	-5837
CW5	+9,6 (-)	WMX	-1827	-2504	-3181	-3857
CW5	+12,8 (-)	WMX	-1148	-1499	-1851	-2202
CW5	+16 (-)	WMX	-646	-784	-921	-1059
CW5	+19,2 (-)	WMX	-335	-406	-476	-547

Table A.10: *Moment action on lower section of core wall 6 with DCL.*

			Case ($a_g \cdot S$)			
Core wall	Level	Action [kNm]	1 (0.10 g)	2 (0.16 g)	3 (0.22 g)	4 (0.28 g)
CW6	+0 +	WMY	1603	2591	3579	4566
CW6	+3,2 +	WMY	869	1409	1948	2487
CW6	+6,4 +	WMY	563	907	1251	1595
CW6	+9,6 +	WMY	424	678	933	1188
CW6	+12,8 +	WMY	334	531	728	925
CW6	+16 +	WMY	226	356	485	614
CW6	+19,2 +	WMY	136	206	275	345
CW6	+0 (-)	WMY	-996	-1566	-2136	-2707
CW6	+3,2 (-)	WMY	-591	-940	-1290	-1640
CW6	+6,4 (-)	WMY	-406	-659	-911	-1164
CW6	+9,6 (-)	WMY	-303	-502	-702	-902
CW6	+12,8 (-)	WMY	-192	-332	-472	-613
CW6	+16 (-)	WMY	-93	-181	-268	-356
CW6	+19,2 (-)	WMY	-35	-85	-136	-186

A.2.2 DCM

Table A.11: *Force action on lower section of core wall 3 with DCM.*

			Case ($a_g \cdot S$)			
Core wall	Level	Action [kN]	1 (0.10 g)	2 (0.16 g)	3 (0.22 g)	4 (0.28 g)
CW3	+0 (+)	WFX	211	337	463	590
CW3	+3,2 (+)	WFX	151	245	338	432
CW3	+6,4 (+)	WFX	95	156	218	280
CW3	+9,6 (+)	WFX	62	106	150	194
CW3	+12,8 (+)	WFX	52	93	133	174
CW3	+16 (+)	WFX	43	81	119	157
CW3	+19,2 (+)	WFX	10	31	52	73
CW3	+0 (-)	WFX	-211	-337	-464	-590
CW3	+3,2 (-)	WFX	-161	-255	-349	-443
CW3	+6,4 (-)	WFX	-111	-172	-234	-295
CW3	+9,6 (-)	WFX	-85	-129	-173	-217
CW3	+12,8 (-)	WFX	-83	-124	-164	-205
CW3	+16 (-)	WFX	-83	-121	-158	-196
CW3	+19,2 (-)	WFX	-59	-79	-100	-121

Table A.12: *Force action on lower section of core wall 4 with DCM.*

			Case ($a_g \cdot S$)			
Core wall	Level	Action [kN]	1 (0.10 g)	2 (0.16 g)	3 (0.22 g)	4 (0.28 g)
CW4	+0 (+)	WFY	379	610	842	1073
CW4	+3,2 (+)	WFY	407	630	853	1076
CW4	+6,4 (+)	WFY	387	592	796	1001
CW4	+9,6 (+)	WFY	355	534	714	893
CW4	+12,8 (+)	WFY	310	458	606	754
CW4	+16 (+)	WFY	249	356	464	571
CW4	+19,2 (+)	WFY	159	211	263	314
CW4	+0 (-)	WFY	-392	-624	-855	-1087
CW4	+3,2 (-)	WFY	-337	-560	-783	-1006
CW4	+6,4 (-)	WFY	-295	-499	-704	-908
CW4	+9,6 (-)	WFY	-243	-423	-602	-782
CW4	+12,8 (-)	WFY	-183	-331	-479	-627
CW4	+16 (-)	WFY	-110	-217	-325	-432
CW4	+19,2 (-)	WFY	-13	-64	-116	-168

Table A.13: *Force action on lower section of core wall 5 with DCM.*

			Case ($a_g \cdot S$)			
Core wall	Level	Action [kN]	1 (0.10 g)	2 (0.16 g)	3 (0.22 g)	4 (0.28 g)
CW5	+0 (+)	WFY	309	500	690	881
CW5	+3,2 (+)	WFY	349	538	726	915
CW5	+6,4 (+)	WFY	337	510	683	856
CW5	+9,6 (+)	WFY	306	456	606	756
CW5	+12,8 (+)	WFY	271	397	523	649
CW5	+16 (+)	WFY	227	326	424	523
CW5	+19,2 (+)	WFY	156	211	266	321
CW5	+0 (-)	WFY	-326	-517	-708	-898
CW5	+3,2 (-)	WFY	-280	-469	-657	-846
CW5	+6,4 (-)	WFY	-240	-413	-586	-760
CW5	+9,6 (-)	WFY	-193	-343	-493	-643
CW5	+12,8 (-)	WFY	-149	-275	-401	-527
CW5	+16 (-)	WFY	-101	-199	-298	-396
CW5	+19,2 (-)	WFY	-27	-82	-137	-192

Table A.14: *Force action on lower section of core wall 6 with DCM.*

			Case ($a_g \cdot S$)			
Core wall	Level	Action [kN]	1 (0.10 g)	2 (0.16 g)	3 (0.22 g)	4 (0.28 g)
CW6	+0 (+)	WFX	430	689	948	1207
CW6	+3,2 (+)	WFX	379	617	856	1095
CW6	+6,4 (+)	WFX	315	516	718	919
CW6	+9,6 (+)	WFX	246	407	569	730
CW6	+12,8 (+)	WFX	188	315	442	568
CW6	+16 (+)	WFX	132	225	318	411
CW6	+19,2 (+)	WFX	56	101	146	191
CW6	+0 (-)	WFX	-433	-692	-951	-1210
CW6	+3,2 (-)	WFX	-418	-657	-895	-1134
CW6	+6,4 (-)	WFX	-357	-559	-760	-962
CW6	+9,6 (-)	WFX	-292	-453	-614	-776
CW6	+12,8 (-)	WFX	-235	-362	-489	-616
CW6	+16 (-)	WFX	-178	-271	-365	-457
CW6	+19,2 (-)	WFX	-95	-140	-185	-231

Table A.15: *Moment action on lower section of core wall 3 with DCM.*

			Case ($a_g \cdot S$)			
Core wall	Level	Action [kNm]	1 (0.10 g)	2 (0.16 g)	3 (0.22 g)	4 (0.28 g)
CW3	+0 (+)	WMY	963	1673	2384	3095
CW3	+3,2 (+)	WMY	457	831	1204	1578
CW3	+6,4 (+)	WMY	261	494	728	962
CW3	+9,6 (+)	WMY	250	464	677	891
CW3	+12,8 (+)	WMY	228	415	603	790
CW3	+16 (+)	WMY	150	276	402	528
CW3	+19,2 (+)	WMY	64	118	173	228
CW3	+0 (-)	WMY	-1407	-2118	-2829	-3540
CW3	+3,2 (-)	WMY	-788	-1162	-1536	-1909
CW3	+6,4 (-)	WMY	-519	-753	-987	-1221
CW3	+9,6 (-)	WMY	-461	-675	-888	-1101
CW3	+12,8 (-)	WMY	-397	-584	-772	-959
CW3	+16 (-)	WMY	-270	-396	-522	-648
CW3	+19,2 (-)	WMY	-118	-173	-228	-282

Table A.16: *Moment action on lower section of core wall 4 with DCM.*

			Case ($a_g \cdot S$)			
Core wall	Level	Action [kNm]	1 (0.10 g)	2 (0.16 g)	3 (0.22 g)	4 (0.28 g)
CW4	+0 (+)	WMX	1358	2866	4375	5883
CW4	+3,2 (+)	WMX	815	1907	2999	4091
CW4	+6,4 (+)	WMX	487	1294	2100	2906
CW4	+9,6 (+)	WMX	269	844	1419	1994
CW4	+12,8 (+)	WMX	117	493	868	1244
CW4	+16 (+)	WMX	13	210	408	605
CW4	+19,2 (+)	WMX	-31	26	83	139
CW4	+0 (-)	WMX	-3670	-5179	-6687	-8196
CW4	+3,2 (-)	WMX	-2825	-3918	-5010	-6102
CW4	+6,4 (-)	WMX	-2201	-3007	-3813	-4620
CW4	+9,6 (-)	WMX	-1649	-2224	-2799	-3375
CW4	+12,8 (-)	WMX	-1136	-1511	-1887	-2263
CW4	+16 (-)	WMX	-646	-843	-1040	-1238
CW4	+19,2 (-)	WMX	-220	-277	-334	-391

Table A.17: *Moment action on lower section of core wall 5 with DCM.*

			Case ($a_g \cdot S$)			
Core wall	Level	Action [kNm]	1 (0.10 g)	2 (0.16 g)	3 (0.22 g)	4 (0.28 g)
CW5	+0 (+)	WMX	1426	2830	4234	5638
CW5	+3,2 (+)	WMX	884	1902	2920	3937
CW5	+6,4 (+)	WMX	548	1293	2037	2782
CW5	+9,6 (+)	WMX	315	835	1355	1874
CW5	+12,8 (+)	WMX	149	476	804	1132
CW5	+16 (+)	WMX	40	207	374	542
CW5	+19,2 (+)	WMX	4	54	105	155
CW5	+0 (-)	WMX	-3255	-4659	-6063	-7467
CW5	+3,2 (-)	WMX	-2508	-3526	-4543	-5561
CW5	+6,4 (-)	WMX	-1935	-2680	-3425	-4170
CW5	+9,6 (-)	WMX	-1418	-1938	-2457	-2977
CW5	+12,8 (-)	WMX	-944	-1272	-1600	-1927
CW5	+16 (-)	WMX	-517	-685	-852	-1019
CW5	+19,2 (-)	WMX	-165	-215	-266	-316

Table A.18: *Moment action on lower section of core wall 6 with DCM.*

			Case ($a_g \cdot S$)			
Core wall	Level	Action [kNm]	1 (0.10 g)	2 (0.16 g)	3 (0.22 g)	4 (0.28 g)
CW6	+0 (+)	WMY	780	1274	1768	2262
CW6	+3,2 (+)	WMY	420	690	959	1229
CW6	+6,4 (+)	WMY	276	448	621	793
CW6	+9,6 (+)	WMY	212	339	466	593
CW6	+12,8 (+)	WMY	170	268	367	465
CW6	+16 (+)	WMY	119	183	248	313
CW6	+19,2 (+)	WMY	77	112	147	182
CW6	+0 (-)	WMY	-866	-1360	-1854	-2347
CW6	+3,2 (-)	WMY	-479	-749	-1018	-1288
CW6	+6,4 (-)	WMY	-297	-469	-641	-813
CW6	+9,6 (-)	WMY	-213	-340	-467	-595
CW6	+12,8 (-)	WMY	-159	-257	-355	-454
CW6	+16 (-)	WMY	-97	-161	-226	-290
CW6	+19,2 (-)	WMY	-39	-74	-109	-144

A.2.3 Static action

Table A.19: *Vertical force action.*

Level	Core wall	WFZ [kN]	Core wall	WFZ [kN]
0	CW3	2333	CW5	1501
3.2	CW3	2042	CW5	1296
6.4	CW3	1741	CW5	1093
9.6	CW3	1412	CW5	875
12.8	CW3	1062	CW5	647
16	CW3	699	CW5	410
19.2	CW3	325	CW5	169
0	CW4	1495	CW6	1606
3.2	CW4	1261	CW6	1443
6.4	CW4	1038	CW6	1250
9.6	CW4	815	CW6	1035
12.8	CW4	593	CW6	802
16	CW4	371	CW6	557
19.2	CW4	152	CW6	300

B
Design Calculation

B.1 Output data from design calculations

The following tables presents the output quantities from the design calculation at each level of the core walls in DCL- and DCM-designs.

The direction of the seismic input for the load combination in the analysis is symbolized with either (+) or (-). This means that two load combinations are analyzed for each acceleration case.

Table B.1: *Output data from design calculation: Case 1:* $a_g \cdot S = 0.10$ g *(-) (DCL).*

		Vertical reinforcement						Dowel reinforcement						Horizontal reinforcement				
		Boundary			Web			Boundary			Web			Boundary+web				
Core wall	Level [m]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	Cost [SEK]	Cost [NOK]
CW3	0	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	3.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	6.4	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	9.6	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	12.8	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	16	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	19.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW4	0	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	855	11.9	10.20	2354	2730
CW4	3.2	1005	3.2	3.22	1709	3.2	5.47	1005	1.7	1.71	101	1.7	0.17	653	11.9	7.80	1763	2044
CW4	6.4	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	251	1.7	0.43	653	11.9	7.80	1694	1965
CW4	9.6	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	302	1.7	0.51	653	11.9	7.80	1702	1974
CW4	12.8	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	302	1.7	0.51	653	11.9	7.80	1702	1974
CW4	16	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	201	1.7	0.34	653	11.9	7.80	1686	1955
CW4	19.2	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	0	1.7	0.00	653	11.9	7.80	1654	1918
CW5	0	1206	3.2	3.86	1709	3.2	5.47	1206	1.7	2.05	0	1.7	0.00	653	11.9	7.80	1840	2134
CW5	3.2	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	0	1.7	0.00	653	11.9	7.80	1654	1918
CW5	6.4	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	0	1.7	0.00	653	11.9	7.80	1654	1918
CW5	9.6	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	0	1.7	0.00	653	11.9	7.80	1654	1918
CW5	12.8	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	0	1.7	0.00	653	11.9	7.80	1654	1918
CW5	16	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	0	1.7	0.00	653	11.9	7.80	1654	1918
CW5	19.2	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	0	1.7	0.00	653	11.9	7.80	1654	1918
CW6	0	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	101	1.7	0.17	1810	7.9	14.37	2095	2430
CW6	3.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	251	1.7	0.43	1608	7.9	12.77	1960	2273
CW6	6.4	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	151	1.7	0.26	1206	7.9	9.58	1625	1884
CW6	9.6	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	50	1.7	0.09	754	7.9	5.99	1250	1449
CW6	12.8	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	50	1.7	0.09	653	7.9	5.19	1170	1357
CW6	16	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	101	1.7	0.17	653	7.9	5.19	1178	1366
CW6	19.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
																Accumulated cost:	SEK 42 893	NOK 49 739

Table B.2: *Output data from design calculation: Case 1: $a_g \cdot S = 0.10$ g (+) (DCL).*

		Vertical reinforcement						Dowel reinforcement						Horizontal reinforcement				
		Boundary			Web			Boundary			Web			Boundary+web				
Core wall	Level [m]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	Cost [SEK]	Cost [NOK]
CW3	0	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	3.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	6.4	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	9.6	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	12.8	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	16	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	19.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW4	0	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	0	1.7	0.00	804	11.9	9.60	1830	2123
CW4	3.2	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	402	1.7	0.68	855	11.9	10.20	1954	2266
CW4	6.4	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	603	1.7	1.03	704	11.9	8.40	1809	2098
CW4	9.6	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	754	1.7	1.28	653	11.9	7.80	1775	2058
CW4	12.8	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	804	1.7	1.37	653	11.9	7.80	1783	2067
CW4	16	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	754	1.7	1.28	653	11.9	7.80	1775	2058
CW4	19.2	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	452	1.7	0.77	653	11.9	7.80	1726	2002
CW5	0	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	0	1.7	0.00	653	11.9	7.80	1654	1918
CW5	3.2	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	0	1.7	0.00	653	11.9	7.80	1654	1918
CW5	6.4	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	101	1.7	0.17	653	11.9	7.80	1670	1936
CW5	9.6	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	251	1.7	0.43	653	11.9	7.80	1694	1965
CW5	12.8	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	402	1.7	0.68	653	11.9	7.80	1718	1993
CW5	16	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	553	1.7	0.94	653	11.9	7.80	1743	2021
CW5	19.2	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	452	1.7	0.77	653	11.9	7.80	1726	2002
CW6	0	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	101	1.7	0.17	1759	7.9	13.97	2055	2383
CW6	3.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	101	1.7	0.17	1508	7.9	11.97	1856	2152
CW6	6.4	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	1056	7.9	8.38	1481	1717
CW6	9.6	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW6	12.8	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW6	16	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW6	19.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
																Accumulated cost:	SEK 42 686	NOK 49 499

Table B.3: *Output data from design calculation: Case 2:* $a_g \cdot S = 0.16$ g *(-) (DCL).*

		Vertical reinforcement						Dowel reinforcement						Horizontal reinforcement				
		Boundary			Web			Boundary			Web			Boundary+web				
Core wall	Level [m]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	Cost [SEK]	Cost [NOK]
CW3	0	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	1106	7.9	8.78	1521	1764
CW3	3.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	6.4	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	9.6	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	12.8	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	16	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	19.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW4	0	4222	3.2	13.51	1709	3.2	5.47	4222	1.7	7.18	1759	1.7	2.99	2463	11.9	29.41	5637	6537
CW4	3.2	2815	3.2	9.01	1709	3.2	5.47	2815	1.7	4.79	1860	1.7	3.16	2161	11.9	25.81	4649	5391
CW4	6.4	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	1910	1.7	3.25	1810	11.9	21.61	3779	4383
CW4	9.6	1206	3.2	3.86	1709	3.2	5.47	1206	1.7	2.05	1759	1.7	2.99	1307	11.9	15.60	2888	3348
CW4	12.8	1005	3.2	3.22	1709	3.2	5.47	1005	1.7	1.71	1508	1.7	2.56	704	11.9	8.40	2048	2375
CW4	16	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	1056	1.7	1.79	653	11.9	7.80	1823	2114
CW4	19.2	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	302	1.7	0.51	653	11.9	7.80	1702	1974
CW5	0	3619	3.2	11.58	1709	3.2	5.47	3619	1.7	6.15	905	1.7	1.54	1709	11.9	20.41	4338	5030
CW5	3.2	2212	3.2	7.08	1709	3.2	5.47	2212	1.7	3.76	1056	1.7	1.79	1558	11.9	18.61	3534	4098
CW5	6.4	1407	3.2	4.50	1709	3.2	5.47	1407	1.7	2.39	1106	1.7	1.88	1206	11.9	14.40	2758	3198
CW5	9.6	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	1005	1.7	1.71	754	11.9	9.00	2398	2781
CW5	12.8	1206	3.2	3.86	1709	3.2	5.47	1206	1.7	2.05	955	1.7	1.62	653	11.9	7.80	1993	2311
CW5	16	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	804	1.7	1.37	653	11.9	7.80	1783	2067
CW5	19.2	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	352	1.7	0.60	653	11.9	7.80	1710	1983
CW6	0	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	2161	1.7	3.67	3569	7.9	28.34	3822	4432
CW6	3.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	2161	1.7	3.67	3267	7.9	25.94	3582	4154
CW6	6.4	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	1759	1.7	2.99	2614	7.9	20.75	3000	3478
CW6	9.6	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	1307	1.7	2.22	1860	7.9	14.77	2329	2700
CW6	12.8	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	1056	1.7	1.79	1206	7.9	9.58	1770	2053
CW6	16	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	804	1.7	1.37	653	7.9	5.19	1291	1497
CW6	19.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	302	1.7	0.51	653	7.9	5.19	1210	1404
																Accumulated cost:	SEK 66 537	NOK 77 156

Table B.4: *Output data from design calculation: Case 2: $a_g \cdot S = 0.16$ g (+) (DCL).*

		Vertical reinforcement						Dowel reinforcement						Horizontal reinforcement				
		Boundary			Web			Boundary			Web			Boundary+web				
Core wall	Level [m]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	Cost [SEK]	Cost [NOK]
CW3	0	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	1106	7.9	8.78	1521	1764
CW3	3.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	6.4	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	9.6	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	12.8	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	16	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	19.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW4	0	2413	3.2	7.72	1709	3.2	5.47	2413	1.7	4.10	1709	1.7	2.91	2413	11.9	28.81	4733	5488
CW4	3.2	1206	3.2	3.86	1709	3.2	5.47	1206	1.7	2.05	2161	1.7	3.67	2413	11.9	28.81	4247	4925
CW4	6.4	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	2262	1.7	3.85	2111	11.9	25.21	3724	4318
CW4	9.6	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	2212	1.7	3.76	1709	11.9	20.41	3245	3763
CW4	12.8	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	2011	1.7	3.42	1106	11.9	13.20	2506	2906
CW4	16	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	1608	1.7	2.73	653	11.9	7.80	1912	2217
CW4	19.2	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	855	1.7	1.45	653	11.9	7.80	1791	2077
CW5	0	2011	3.2	6.43	1709	3.2	5.47	2011	1.7	3.42	804	1.7	1.37	1659	11.9	19.81	3518	4080
CW5	3.2	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	1357	1.7	2.31	1759	11.9	21.01	3167	3672
CW5	6.4	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	1508	1.7	2.56	1558	11.9	18.61	2955	3427
CW5	9.6	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	1458	1.7	2.48	1156	11.9	13.80	2476	2872
CW5	12.8	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	1407	1.7	2.39	704	11.9	8.40	1939	2248
CW5	16	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	1307	1.7	2.22	653	11.9	7.80	1864	2161
CW5	19.2	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	855	1.7	1.45	653	11.9	7.80	1791	2077
CW6	0	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	2161	1.7	3.67	3569	7.9	28.34	3822	4432
CW6	3.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	2011	1.7	3.42	3116	7.9	24.74	3439	3987
CW6	6.4	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	1558	1.7	2.65	2463	7.9	19.56	2848	3302
CW6	9.6	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	1156	1.7	1.97	1709	7.9	13.57	2185	2534
CW6	12.8	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	855	1.7	1.45	1056	7.9	8.38	1618	1876
CW6	16	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	653	1.7	1.11	653	7.9	5.19	1267	1469
CW6	19.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	151	1.7	0.26	653	7.9	5.19	1186	1376
																Accumulated cost:	SEK 64 725	NOK 75 055

Table B.5: *Output data from design calculation: Case 3: $a_g \cdot S = 0.22$ g (-) (DCL).*

		Vertical reinforcement						Dowel reinforcement						Horizontal reinforcement				
		Boundary			Web			Boundary			Web			Boundary+web				
Core wall	Level [m]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	Cost [SEK]	Cost [NOK]
CW3	0	2614	3.2	8.36	1206	3.2	3.86	2614	1.7	4.44	0	1.7	0.00	2011	7.9	15.96	3169	3675
CW3	3.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	1206	7.9	9.58	1601	1856
CW3	6.4	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	9.6	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	12.8	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	16	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	19.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW4	0	6836	3.2	21.88	1709	3.2	5.47	6836	1.7	11.62	3619	1.7	6.15	4072	11.9	48.61	9029	10469
CW4	3.2	4624	3.2	14.80	1709	3.2	5.47	4624	1.7	7.86	3669	1.7	6.24	3720	11.9	44.41	7601	8814
CW4	6.4	3217	3.2	10.29	1709	3.2	5.47	3217	1.7	5.47	3519	1.7	5.98	3217	11.9	38.41	6337	7348
CW4	9.6	2011	3.2	6.43	1709	3.2	5.47	2011	1.7	3.42	3217	1.7	5.47	2564	11.9	30.61	4965	5758
CW4	12.8	1608	3.2	5.15	1709	3.2	5.47	1608	1.7	2.73	2664	1.7	4.53	1709	11.9	20.41	3690	4279
CW4	16	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	1910	1.7	3.25	653	11.9	7.80	1960	2273
CW4	19.2	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	704	1.7	1.20	653	11.9	7.80	1767	2049
CW5	0	6032	3.2	19.30	1709	3.2	5.47	6032	1.7	10.25	2413	1.7	4.10	3066	11.9	36.61	7285	8448
CW5	3.2	3820	3.2	12.22	1709	3.2	5.47	3820	1.7	6.49	2564	1.7	4.36	2865	11.9	34.21	6051	7017
CW5	6.4	2614	3.2	8.36	1709	3.2	5.47	2614	1.7	4.44	2463	1.7	4.19	2413	11.9	28.81	4947	5736
CW5	9.6	3016	3.2	9.65	1709	3.2	5.47	3016	1.7	5.13	2212	1.7	3.76	1810	11.9	21.61	4386	5086
CW5	12.8	2011	3.2	6.43	1709	3.2	5.47	2011	1.7	3.42	1960	1.7	3.33	1156	11.9	13.80	3115	3613
CW5	16	1206	3.2	3.86	1709	3.2	5.47	1206	1.7	2.05	1608	1.7	2.73	653	11.9	7.80	2098	2433
CW5	19.2	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	804	1.7	1.37	653	11.9	7.80	1783	2067
CW6	0	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	4273	1.7	7.26	5378	7.9	42.70	5875	6813
CW6	3.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	4072	1.7	6.92	4926	7.9	39.11	5205	6036
CW6	6.4	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	3368	1.7	5.73	3971	7.9	31.53	4334	5026
CW6	9.6	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	2614	1.7	4.44	2966	7.9	23.55	3416	3961
CW6	12.8	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	2061	1.7	3.50	2111	7.9	16.76	2649	3072
CW6	16	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	1558	1.7	2.65	1257	7.9	9.98	1891	2192
CW6	19.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	653	1.7	1.11	653	7.9	5.19	1267	1469
																Accumulated cost:	SEK 100 231	NOK 116 228

Table B.6: *Output data from design calculation: Case 3: $a_g \cdot S = 0.22$ g (+) (DCL).*

		Vertical reinforcement						Dowel reinforcement						Horizontal reinforcement				
		Boundary			Web			Boundary			Web			Boundary+web				
Core wall	Level [m]	$A_{\phi_{tot}}$ [mm²]	Length [m]	Volume [cm³]	$A_{\phi_{tot}}$ [mm²]	Length [m]	Volume [cm³]	$A_{\phi_{tot}}$ [mm²]	Length [m]	Volume [cm³]	$A_{\phi_{tot}}$ [mm²]	Length [m]	Volume [cm³]	$A_{\phi_{tot}}$ [mm²]	Length [m]	Volume [cm³]	Cost [SEK]	Cost [NOK]
CW3	0	1810	3.2	5.79	1206	3.2	3.86	1810	1.7	3.08	0	1.7	0.00	2011	7.9	15.96	2797	3243
CW3	3.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	1156	7.9	9.18	1561	1810
CW3	6.4	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	9.6	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	12.8	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	16	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	19.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW4	0	4825	3.2	15.44	1709	3.2	5.47	4825	1.7	8.20	3569	1.7	6.07	4021	11.9	48.01	8031	9313
CW4	3.2	2815	3.2	9.01	1709	3.2	5.47	2815	1.7	4.79	3971	1.7	6.75	3971	11.9	47.41	7107	8241
CW4	6.4	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	3870	1.7	6.58	3519	11.9	42.01	6095	7068
CW4	9.6	1005	3.2	3.22	1709	3.2	5.47	1005	1.7	1.71	3619	1.7	6.15	2915	11.9	34.81	4977	5771
CW4	12.8	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	3167	1.7	5.38	2161	11.9	25.81	3928	4555
CW4	16	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	2463	1.7	4.19	1156	11.9	13.80	2638	3059
CW4	19.2	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	1307	1.7	2.22	653	11.9	7.80	1864	2161
CW5	0	4423	3.2	14.15	1709	3.2	5.47	4423	1.7	7.52	2362	1.7	4.02	2966	11.9	35.41	6415	7439
CW5	3.2	2614	3.2	8.36	1709	3.2	5.47	2614	1.7	4.44	2865	1.7	4.87	3066	11.9	36.61	5776	6698
CW5	6.4	1407	3.2	4.50	1709	3.2	5.47	1407	1.7	2.39	2865	1.7	4.87	2765	11.9	33.01	4865	5642
CW5	9.6	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	2664	1.7	4.53	2161	11.9	25.81	4313	5001
CW5	12.8	1206	3.2	3.86	1709	3.2	5.47	1206	1.7	2.05	2413	1.7	4.10	1608	11.9	19.21	3346	3880
CW5	16	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	2111	1.7	3.59	905	11.9	10.80	2287	2652
CW5	19.2	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	1307	1.7	2.22	653	11.9	7.80	1864	2161
CW6	0	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	4222	1.7	7.18	5378	7.9	42.70	5867	6803
CW6	3.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	3921	1.7	6.67	4775	7.9	37.92	5061	5869
CW6	6.4	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	3167	1.7	5.38	3820	7.9	30.33	4182	4850
CW6	9.6	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	2413	1.7	4.10	2815	7.9	22.35	3264	3785
CW6	12.8	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	1910	1.7	3.25	1910	7.9	15.17	2466	2859
CW6	16	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	1407	1.7	2.39	1056	7.9	8.38	1707	1979
CW6	19.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	503	1.7	0.85	653	7.9	5.19	1243	1441
																Accumulated cost:	SEK 97 462	NOK 113 017

Table B.7: *Output data from design calculation: Case 4:* $a_g \cdot S = 0.28$ g *(-) (DCL).*

		Vertical reinforcement						Dowel reinforcement						Horizontal reinforcement				
		Boundary			Web			Boundary			Web			Boundary+web				
Core wall	Level [m]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	Cost [SEK]	Cost [NOK]
CW3	0	4825	3.2	15.44	1206	3.2	3.86	4825	1.7	8.20	0	1.7	0.00	2865	7.9	22.75	4870	5648
CW3	3.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	1860	7.9	14.77	2119	2457
CW3	6.4	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	804	7.9	6.39	1282	1486
CW3	9.6	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	12.8	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	16	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	19.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	101	1.7	0.17	653	7.9	5.19	1178	1366
CW4	0	9651	3.2	30.88	1709	3.2	5.47	9651	1.7	16.41	5479	1.7	9.31	5680	11.9	67.82	12513	14510
CW4	3.2	6434	3.2	20.59	1709	3.2	5.47	6434	1.7	10.94	5479	1.7	9.31	5278	11.9	63.02	10554	12238
CW4	6.4	4423	3.2	14.15	1709	3.2	5.47	4423	1.7	7.52	5177	1.7	8.80	4624	11.9	55.22	8810	10216
CW4	9.6	3016	3.2	9.65	1709	3.2	5.47	3016	1.7	5.13	4624	1.7	7.86	3770	11.9	45.01	7069	8197
CW4	12.8	2011	3.2	6.43	1709	3.2	5.47	2011	1.7	3.42	3870	1.7	6.58	2714	11.9	32.41	5247	6084
CW4	16	1206	3.2	3.86	1709	3.2	5.47	1206	1.7	2.05	2765	1.7	4.70	1407	11.9	16.80	3167	3672
CW4	19.2	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	1106	1.7	1.88	653	11.9	7.80	1831	2124
CW5	0	8646	3.2	27.67	1709	3.2	5.47	8646	1.7	14.70	3921	1.7	6.67	4373	11.9	52.21	10267	11906
CW5	3.2	5630	3.2	18.02	1709	3.2	5.47	5630	1.7	9.57	4122	1.7	7.01	4172	11.9	49.81	8669	10052
CW5	6.4	3820	3.2	12.22	1709	3.2	5.47	3820	1.7	6.49	3870	1.7	6.58	3619	11.9	43.21	7144	8284
CW5	9.6	4222	3.2	13.51	1709	3.2	5.47	4222	1.7	7.18	3418	1.7	5.81	2815	11.9	33.61	6315	7323
CW5	12.8	2815	3.2	9.01	1709	3.2	5.47	2815	1.7	4.79	2966	1.7	5.04	2061	11.9	24.61	4708	5460
CW5	16	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	2362	1.7	4.02	1156	11.9	13.80	3087	3580
CW5	19.2	1005	3.2	3.22	1709	3.2	5.47	1005	1.7	1.71	1257	1.7	2.14	653	11.9	7.80	1949	2260
CW6	0	2614	3.2	8.36	1206	3.2	3.86	2614	1.7	4.44	6333	1.7	10.77	7188	7.9	57.07	8293	9616
CW6	3.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	5982	1.7	10.17	6585	7.9	52.28	6827	7917
CW6	6.4	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	4976	1.7	8.46	5378	7.9	42.70	5709	6620
CW6	9.6	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	3921	1.7	6.67	4072	7.9	32.33	4503	5222
CW6	12.8	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	3116	1.7	5.30	2966	7.9	23.55	3497	4055
CW6	16	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	2312	1.7	3.93	1860	7.9	14.77	2490	2888
CW6	19.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	1005	1.7	1.71	653	7.9	5.19	1323	1535
																Accumulated cost:	SEK 136 906	NOK 158 757

Table B.8: *Output data from design calculation: Case 4:* $a_g \cdot S = 0.28$ g *(+) (DCL).*

		Vertical reinforcement						Dowel reinforcement						Horizontal reinforcement				
		Boundary			Web			Boundary			Web			Boundary+web				
Core wall	Level [m]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	Cost [SEK]	Cost [NOK]
CW3	0	4222	3.2	13.51	1206	3.2	3.86	4222	1.7	7.18	0	1.7	0.00	2865	7.9	22.75	4591	5324
CW3	3.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	1810	7.9	14.37	2079	2411
CW3	6.4	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	754	7.9	5.99	1242	1440
CW3	9.6	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	12.8	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	16	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW3	19.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	0	1.7	0.00	653	7.9	5.19	1162	1347
CW4	0	7640	3.2	24.45	1709	3.2	5.47	7640	1.7	12.99	5429	1.7	9.23	5630	11.9	67.22	11516	13354
CW4	3.2	4624	3.2	14.80	1709	3.2	5.47	4624	1.7	7.86	5730	1.7	9.74	5529	11.9	66.02	10051	11655
CW4	6.4	3016	3.2	9.65	1709	3.2	5.47	3016	1.7	5.13	5529	1.7	9.40	4926	11.9	58.82	8568	9936
CW4	9.6	2011	3.2	6.43	1709	3.2	5.47	2011	1.7	3.42	5077	1.7	8.63	4172	11.9	49.81	7147	8288
CW4	12.8	1005	3.2	3.22	1709	3.2	5.47	1005	1.7	1.71	4373	1.7	7.43	3167	11.9	37.81	5392	6253
CW4	16	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	3318	1.7	5.64	1910	11.9	22.81	3658	4242
CW4	19.2	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	1709	1.7	2.91	653	11.9	7.80	1928	2236
CW5	0	6836	3.2	21.88	1709	3.2	5.47	6836	1.7	11.62	3870	1.7	6.58	4323	11.9	51.61	9363	10858
CW5	3.2	4222	3.2	13.51	1709	3.2	5.47	4222	1.7	7.18	4373	1.7	7.43	4373	11.9	52.21	8293	9617
CW5	6.4	2614	3.2	8.36	1709	3.2	5.47	2614	1.7	4.44	4273	1.7	7.26	3921	11.9	46.81	7003	8121
CW5	9.6	3016	3.2	9.65	1709	3.2	5.47	3016	1.7	5.13	3870	1.7	6.58	3217	11.9	38.41	6300	7306
CW5	12.8	2011	3.2	6.43	1709	3.2	5.47	2011	1.7	3.42	3418	1.7	5.81	2463	11.9	29.41	4880	5659
CW5	16	1206	3.2	3.86	1709	3.2	5.47	1206	1.7	2.05	2915	1.7	4.96	1558	11.9	18.61	3367	3905
CW5	19.2	804	3.2	2.57	1709	3.2	5.47	804	1.7	1.37	1759	1.7	2.99	653	11.9	7.80	1936	2245
CW6	0	2614	3.2	8.36	1206	3.2	3.86	2614	1.7	4.44	6333	1.7	10.77	7138	7.9	56.67	8253	9570
CW6	3.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	5831	1.7	9.91	6434	7.9	51.09	6684	7750
CW6	6.4	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	4825	1.7	8.20	5228	7.9	41.51	5565	6453
CW6	9.6	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	3720	1.7	6.32	3921	7.9	31.13	4351	5045
CW6	12.8	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	2915	1.7	4.96	2815	7.9	22.35	3345	3878
CW6	16	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	2111	1.7	3.59	1709	7.9	13.57	2338	2711
CW6	19.2	603	3.2	1.93	1206	3.2	3.86	603	1.7	1.03	855	1.7	1.45	653	7.9	5.19	1299	1507
																Accumulated cost:	SEK 133 798	NOK 155 153

Table B.9: *Output data from design calculation: Case 1 (-) (DCM).*

		Vertical reinforcement						Dowel reinforcement						Horizontal reinforcement				
		Boundary			Web			Boundary			Web			Boundary+web				
Core wall	Level [m]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	Cost [SEK]	Cost [NOK]
CW3	0	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	3.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	6.4	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	9.6	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	12.8	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	16	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	19.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW4	0	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW4	3.2	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW4	6.4	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW4	9.6	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW4	12.8	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW4	16	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW4	19.2	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW5	0	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW5	3.2	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW5	6.4	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW5	9.6	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW5	12.8	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW5	16	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW5	19.2	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW6	0	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	1206	7.9	9.58	1880	2180
CW6	3.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW6	6.4	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW6	9.6	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW6	12.8	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW6	16	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW6	19.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
																Accumulated cost:	SEK 50 280	NOK 58 305

Table B.10: *Output data from design calculation: Case 1 (+) (DCM).*

		Vertical reinforcement						Dowel reinforcement						Horizontal reinforcement				
		Boundary			Web			Boundary			Web			Boundary+web				
Core wall	Level [m]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	Cost [SEK]	Cost [NOK]
CW3	0	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	3.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	6.4	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	9.6	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	12.8	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	16	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	19.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW4	0	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW4	3.2	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW4	6.4	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW4	9.6	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW4	12.8	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW4	16	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	101	1.7	0.17	653	11.9	7.80	2135	2476
CW4	19.2	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	201	1.7	0.34	653	11.9	7.80	2151	2495
CW5	0	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW5	3.2	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW5	6.4	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW5	9.6	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW5	12.8	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW5	16	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW5	19.2	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	201	1.7	0.34	653	11.9	7.80	2151	2495
CW6	0	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	1156	7.9	9.18	1840	2134
CW6	3.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW6	6.4	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW6	9.6	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW6	12.8	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW6	16	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW6	19.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
																Accumulated cost:	SEK 50 321	NOK 58 353

Table B.11: *Output data from design calculation: Case 2 (-) (DCM).*

		Vertical reinforcement						Dowel reinforcement						Horizontal reinforcement				
		Boundary			Web			Boundary			Web			Boundary+web				
Core wall	Level [m]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	Cost [SEK]	Cost [NOK]
CW3	0	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	3.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	6.4	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	9.6	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	12.8	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	16	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	19.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW4	0	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	653	1.7	1.11	1608	11.9	19.21	3342	3876
CW4	3.2	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW4	6.4	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW4	9.6	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW4	12.8	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW4	16	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW4	19.2	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW5	0	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	1005	11.9	12.00	2531	2935
CW5	3.2	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW5	6.4	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW5	9.6	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW5	12.8	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW5	16	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW5	19.2	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW6	0	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	955	1.7	1.62	2765	7.9	21.95	3269	3791
CW6	3.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	201	1.7	0.34	1759	7.9	13.97	2351	2726
CW6	6.4	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	1056	7.9	8.38	1760	2041
CW6	9.6	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW6	12.8	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW6	16	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW6	19.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
																Accumulated cost:	SEK 54 534	NOK 63 237

Table B.12: *Output data from design calculation: Case 2 (+) (DCM).*

		Vertical reinforcement						Dowel reinforcement						Horizontal reinforcement				
		Boundary			Web			Boundary			Web			Boundary+web				
Core wall	Level [m]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	Cost [SEK]	Cost [NOK]
CW3	0	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	3.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	6.4	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	9.6	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	12.8	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	16	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	19.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW4	0	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	603	1.7	1.03	1558	11.9	18.61	3275	3798
CW4	3.2	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	402	1.7	0.68	905	11.9	10.80	2478	2873
CW4	6.4	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	402	1.7	0.68	653	11.9	7.80	2184	2532
CW4	9.6	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	402	1.7	0.68	653	11.9	7.80	2184	2532
CW4	12.8	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	553	1.7	0.94	653	11.9	7.80	2208	2560
CW4	16	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	603	1.7	1.03	653	11.9	7.80	2216	2569
CW4	19.2	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	402	1.7	0.68	653	11.9	7.80	2184	2532
CW5	0	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	905	11.9	10.80	2413	2798
CW5	3.2	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW5	6.4	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW5	9.6	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	101	1.7	0.17	653	11.9	7.80	2135	2476
CW5	12.8	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	302	1.7	0.51	653	11.9	7.80	2167	2513
CW5	16	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	503	1.7	0.85	653	11.9	7.80	2200	2551
CW5	19.2	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	503	1.7	0.85	653	11.9	7.80	2200	2551
CW6	0	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	955	1.7	1.62	2714	7.9	21.55	3229	3745
CW6	3.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	1558	7.9	12.37	2159	2503
CW6	6.4	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	855	7.9	6.78	1601	1856
CW6	9.6	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW6	12.8	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW6	16	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW6	19.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
																Accumulated cost:	SEK 54 922	NOK 63 688

Table B.13: *Output data from design calculation: Case 3 (-) (DCM).*

		Vertical reinforcement						Dowel reinforcement						Horizontal reinforcement				
		Boundary			Web			Boundary			Web			Boundary+web				
Core wall	Level [m]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	Cost [SEK]	Cost [NOK]
CW3	0	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	1357	7.9	10.78	1999	2318
CW3	3.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	6.4	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	9.6	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	12.8	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	16	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	19.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW4	0	2614	3.2	8.36	1709	3.2	5.47	2614	1.7	4.44	2262	1.7	3.85	3016	11.9	36.01	5621	6518
CW4	3.2	3217	3.2	10.29	1709	3.2	5.47	3217	1.7	5.47	1206	1.7	2.05	1659	11.9	19.81	4141	4802
CW4	6.4	2614	3.2	8.36	1709	3.2	5.47	2614	1.7	4.44	955	1.7	1.62	1005	11.9	12.00	3056	3544
CW4	9.6	2011	3.2	6.43	1709	3.2	5.47	2011	1.7	3.42	704	1.7	1.20	653	11.9	7.80	2325	2696
CW4	12.8	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	653	1.7	1.11	653	11.9	7.80	2224	2579
CW4	16	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	452	1.7	0.77	653	11.9	7.80	2192	2541
CW4	19.2	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	0	1.7	0.00	653	11.9	7.80	2119	2457
CW5	0	2011	3.2	6.43	1709	3.2	5.47	2011	1.7	3.42	1257	1.7	2.14	2111	11.9	25.21	4121	4778
CW5	3.2	2413	3.2	7.72	1709	3.2	5.47	2413	1.7	4.10	452	1.7	0.77	1056	11.9	12.60	2942	3411
CW5	6.4	2011	3.2	6.43	1709	3.2	5.47	2011	1.7	3.42	251	1.7	0.43	653	11.9	7.80	2252	2612
CW5	9.6	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	302	1.7	0.51	653	11.9	7.80	2167	2513
CW5	12.8	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	352	1.7	0.60	653	11.9	7.80	2175	2523
CW5	16	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	352	1.7	0.60	653	11.9	7.80	2175	2523
CW5	19.2	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	101	1.7	0.17	653	11.9	7.80	2135	2476
CW6	0	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	2765	1.7	4.70	4273	7.9	33.92	4756	5515
CW6	3.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	1508	1.7	2.56	2865	7.9	22.75	3438	3986
CW6	6.4	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	855	1.7	1.45	1960	7.9	15.57	2615	3032
CW6	9.6	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	603	1.7	1.03	1307	7.9	10.38	2056	2385
CW6	12.8	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	503	1.7	0.85	754	7.9	5.99	1602	1857
CW6	16	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	452	1.7	0.77	653	7.9	5.19	1514	1755
CW6	19.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	101	1.7	0.17	653	7.9	5.19	1457	1690
																Accumulated cost:	SEK 67 730	NOK 78 540

Table B.14: *Output data from design calculation: Case 3 (+) (DCM).*

		Vertical reinforcement						Dowel reinforcement						Horizontal reinforcement				
		Boundary			Web			Boundary			Web			Boundary+web				
Core wall	Level [m]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	Cost [SEK]	Cost [NOK]
CW3	0	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	1357	7.9	10.78	1999	2318
CW3	3.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	6.4	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	9.6	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	12.8	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	16	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	19.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW4	0	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	2161	1.7	3.67	2915	11.9	34.81	5115	5931
CW4	3.2	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	1608	1.7	2.73	1960	11.9	23.41	3908	4531
CW4	6.4	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	1458	1.7	2.48	1407	11.9	16.80	3236	3752
CW4	9.6	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	1206	1.7	2.05	754	11.9	9.00	2430	2818
CW4	12.8	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	1206	1.7	2.05	653	11.9	7.80	2313	2682
CW4	16	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	1106	1.7	1.88	653	11.9	7.80	2297	2663
CW4	19.2	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	653	1.7	1.11	653	11.9	7.80	2224	2579
CW5	0	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	1106	1.7	1.88	2011	11.9	24.01	3886	4506
CW5	3.2	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	855	1.7	1.45	1357	11.9	16.20	3080	3572
CW5	6.4	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	754	1.7	1.28	905	11.9	10.80	2534	2939
CW5	9.6	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	855	1.7	1.45	653	11.9	7.80	2256	2616
CW5	12.8	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	955	1.7	1.62	653	11.9	7.80	2272	2635
CW5	16	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	1005	1.7	1.71	653	11.9	7.80	2280	2644
CW5	19.2	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	754	1.7	1.28	653	11.9	7.80	2240	2598
CW6	0	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	2714	1.7	4.61	4273	7.9	33.92	4748	5506
CW6	3.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	1307	1.7	2.22	2714	7.9	21.55	3286	3810
CW6	6.4	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	653	1.7	1.11	1759	7.9	13.97	2423	2810
CW6	9.6	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	352	1.7	0.60	1106	7.9	8.78	1856	2153
CW6	12.8	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	251	1.7	0.43	653	7.9	5.19	1481	1718
CW6	16	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	201	1.7	0.34	653	7.9	5.19	1473	1709
CW6	19.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
																Accumulated cost:	SEK 67 427	NOK 78 188

Table B.15: *Output data from design calculation: Case 4 (-) (DCM).*

		Vertical reinforcement						Dowel reinforcement						Horizontal reinforcement				
		Boundary			Web			Boundary			Web			Boundary+web				
Core wall	Level [m]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	Cost [SEK]	Cost [NOK]
CW3	0	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	2111	7.9	16.76	2597	3012
CW3	3.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	704	7.9	5.59	1481	1717
CW3	6.4	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	9.6	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	12.8	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	16	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	19.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW4	0	4222	3.2	13.51	1709	3.2	5.47	4222	1.7	7.18	3870	1.7	6.58	4373	11.9	52.21	8213	9523
CW4	3.2	4624	3.2	14.80	1709	3.2	5.47	4624	1.7	7.86	2463	1.7	4.19	2714	11.9	32.41	6230	7225
CW4	6.4	3820	3.2	12.22	1709	3.2	5.47	3820	1.7	6.49	2011	1.7	3.42	1860	11.9	22.21	4785	5548
CW4	9.6	3016	3.2	9.65	1709	3.2	5.47	3016	1.7	5.13	1508	1.7	2.56	1005	11.9	12.00	3331	3863
CW4	12.8	2212	3.2	7.08	1709	3.2	5.47	2212	1.7	3.76	1307	1.7	2.22	653	11.9	7.80	2515	2916
CW4	16	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	955	1.7	1.62	653	11.9	7.80	2272	2635
CW4	19.2	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	201	1.7	0.34	653	11.9	7.80	2151	2495
CW5	0	3418	3.2	10.94	1709	3.2	5.47	3418	1.7	5.81	2564	1.7	4.36	3267	11.9	39.01	6336	7347
CW5	3.2	3820	3.2	12.22	1709	3.2	5.47	3820	1.7	6.49	1508	1.7	2.56	1960	11.9	23.41	4822	5591
CW5	6.4	3016	3.2	9.65	1709	3.2	5.47	3016	1.7	5.13	1156	1.7	1.97	1206	11.9	14.40	3510	4071
CW5	9.6	2413	3.2	7.72	1709	3.2	5.47	2413	1.7	4.10	1056	1.7	1.79	704	11.9	8.40	2626	3046
CW5	12.8	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	1005	1.7	1.71	653	11.9	7.80	2280	2644
CW5	16	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	855	1.7	1.45	653	11.9	7.80	2256	2616
CW5	19.2	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	402	1.7	0.68	653	11.9	7.80	2184	2532
CW6	0	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	4524	1.7	7.69	5831	7.9	46.30	6274	7276
CW6	3.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	2815	1.7	4.79	4021	7.9	31.93	4565	5293
CW6	6.4	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	1860	1.7	3.16	2815	7.9	22.35	3454	4006
CW6	9.6	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	1407	1.7	2.39	2011	7.9	15.96	2744	3182
CW6	12.8	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	1156	1.7	1.97	1307	7.9	10.38	2145	2487
CW6	16	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	905	1.7	1.54	653	7.9	5.19	1586	1840
CW6	19.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	352	1.7	0.60	653	7.9	5.19	1498	1737
																Accumulated cost:	SEK 87 062	NOK 100 957

Table B.16: *Output data from design calculation: Case 4 (+) (DCM).*

		Vertical reinforcement						Dowel reinforcement						Horizontal reinforcement				
		Boundary			Web			Boundary			Web			Boundary+web				
Core wall	Level [m]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	$A_{\phi_{tot}}$ [mm^2]	Length [m]	Volume [cm^3]	Cost [SEK]	Cost [NOK]
CW3	0	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	2111	7.9	16.76	2597	3012
CW3	3.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	6.4	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	9.6	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	12.8	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	16	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW3	19.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	0	1.7	0.00	653	7.9	5.19	1441	1671
CW4	0	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	3770	1.7	6.41	4323	11.9	51.61	7021	8142
CW4	3.2	2413	3.2	7.72	1709	3.2	5.47	2413	1.7	4.10	2865	1.7	4.87	3066	11.9	36.61	5683	6590
CW4	6.4	2011	3.2	6.43	1709	3.2	5.47	2011	1.7	3.42	2463	1.7	4.19	2262	11.9	27.01	4491	5208
CW4	9.6	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	2061	1.7	3.50	1458	11.9	17.40	3392	3933
CW4	12.8	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	1910	1.7	3.25	905	11.9	10.80	2720	3154
CW4	16	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	1558	1.7	2.65	653	11.9	7.80	2369	2747
CW4	19.2	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	905	1.7	1.54	653	11.9	7.80	2264	2626
CW5	0	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	2413	1.7	4.10	3167	11.9	37.81	5449	6319
CW5	3.2	2011	3.2	6.43	1709	3.2	5.47	2011	1.7	3.42	1860	1.7	3.16	2262	11.9	27.01	4394	5095
CW5	6.4	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	1608	1.7	2.73	1659	11.9	19.81	3554	4122
CW5	9.6	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	1608	1.7	2.73	1206	11.9	14.40	3025	3507
CW5	12.8	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	1608	1.7	2.73	754	11.9	9.00	2495	2893
CW5	16	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	1508	1.7	2.56	653	11.9	7.80	2361	2738
CW5	19.2	1810	3.2	5.79	1709	3.2	5.47	1810	1.7	3.08	1056	1.7	1.79	653	11.9	7.80	2288	2654
CW6	0	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	4524	1.7	7.69	5831	7.9	46.30	6274	7276
CW6	3.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	2614	1.7	4.44	3820	7.9	30.33	4373	5071
CW6	6.4	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	1659	1.7	2.82	2614	7.9	20.75	3263	3783
CW6	9.6	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	1206	1.7	2.05	1810	7.9	14.37	2552	2959
CW6	12.8	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	905	1.7	1.54	1106	7.9	8.78	1945	2256
CW6	16	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	653	1.7	1.11	653	7.9	5.19	1546	1793
CW6	19.2	1206	3.2	3.86	1206	3.2	3.86	1206	1.7	2.05	151	1.7	0.26	653	7.9	5.19	1465	1699
																Accumulated cost:	SEK 84 170	NOK 97 604