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ALTERNATIVE DESIGN FOR STEEL ROOF TRUSS

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ABSTRACT

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This thesis work was done for Wärtsilä Power Plants. The idea and the content of this subject came up during the summer 2013 while I was working for Wärtsilä as a summer trainee.

The purpose of this work is to give an alternative solution for the existing roof truss when radiators are located on top of the roof. The current solution is relatively new and it was implemented in a short period of time of the moment when the idea of radiators being on top of the roof was introduced.

Most of the Wärtsiläs power plants are located on earthquake zones. This meant that seismic calculation needed to be done for this work. The structural modeling of the structure is done with Autodesk Robot Structural Analysis Professional 2014 and the load calculations are done with Microsoft Excel. The seismic loads are calculated based on response spectrum method and the calculations are done with Autodesk Robot Structural Analysis Professional 2014.

The results of this work were better than expected. The new geometry of the roof truss proved to be lighter and stiffer than the current truss. It also increased the natural frequencies of the whole structure compared to the existing one. Due to these factors also the support reactions of the whole structure decreased around 10 %.

Keywords Steel, roof truss, seismic design

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TIIVISTELMÄ

Tekijä	Jesse Iivonen
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Ohjaaja	Tapani Hahtokari, Simon Nyman

Tämä työ on tehty Wärtsilä Power Plantsille. Aiheen idea ja sisältö muodostuivat kesän 2013 aikana, kun työskentelin Wärtsilälle kesäharjoittelijana.

Työn tarkoituksena on antaa vaihtoehtoinen suunnittelumalli olemassa olevalle kattoristikolle, kun radiaattorit ovat katolla. Nykyinen ratkaisu on suhteellisen uusi ja se otettiin käyttöön lyhyen ajan sisällä hetkestä jolloin idea radiaattoreiden sijoittamisesta katolle esiteltiin.

Suurin osa Wärtsilän voimalaitoksista sijaitsevat maanjäristysalueilla, joten seismiset laskelmat suoritettiin osana tätä työtä. Rakenteellinen mallintaminen tehtiin Autodesk Robot Structural Analysis Professional 2014 ohjelmalla ja kuormat laskettiin Microsoft Excelillä. Seismiset kuormat laskettiin vastespektrimenetelmällä ja laskelmat suoritettiin Autodesk Robot Structural Professional 2014 -ohjelmalla.

Tämän työn tulokset ovat parempia kuin työn lähtötilanteessa oletettiin. Kattoristikon uusi geometria osoittautui kevyemmäksi sekä jäykemmäksi kuin nykyinen ristikko. Se kasvatti myös koko rakenteen ominaistaajuksia verrattuna olemassa olevaan ristikkoon. Ja näiden tekijöiden johdosta myös koko rakenteen tukireaktiot laskivat suunnilleen 10 %.

Avainsanat Teräs, kattoristikko, maanjäristyssuunnittelu

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LIST OF SYMBOLS USED

q	= uniform load
Q	= concentrated load
$q_p(z)$	= peak velocity pressure at height z
$l_v(z)$	= turbulence intensity at height z
ρ	= air density
$v_m(z)$	= mean wind velocity at height z
w_e	= pressure on external surface
$q_p(z_e)$	= peak velocity pressure on external surface at height z
c_{pe}	= pressure coefficient for external the external pressure
PGA	= peak ground acceleration
$S_e(t)$	= elastic response spectrum at time t
a_{gd}	= design ground acceleration
S	= soil factor
T	= vibration period
η	= damping factor
T_B, T_C, T_D	= limits of spectral acceleration
$S_d(t)$	= design elastic spectrum
q	= behavior factor
β	= lower bound factor

F_b	= base shear force
m	= mass
λ	= correction factor
H	= height
T_1	= fundamental period
C_t	= factor depending on structure type
c	= damping coefficient
k	= stiffness coefficient
x	= displacement
$\dot{x}$	= velocity
$\ddot{x}$	= acceleration
f	= excitation force
M	= mass matrix
C	= damping matrix
K	= stiffness matrix
F	= excitation matrix
N_{Ed}	= designing compressive strength
$N_{b,Rd}$	= designing buckling resistance under compression
A	= cross-sectional area
f_y	= materials yield strength

γ_{M1} = partial factor

κ = reduction factor

ϕ = equation that takes consideration buckling curve and non-dimensional slenderness

λ_k = non-dimensional slenderness

N_{Ed} = design normal force

N_{Rd} = yield strength

$M_{y,Ed}$ = design bending moment

x_y = reduction factor

x_{Lt} = reduction factor

k_{yy} = interaction factor

κ_y = reduction factor

κ_{Lt} = reduction factor

α_{Lt} = imperfection factor

Mcr = elastic critical moment

C_{my} = equivalent uniform moment

h_w = height of the web

t_w = thickness of the web

η = reduction factor

F_{Rd} = resistance of the web in relation to local buckling

f_{yw} = yield strength of the web

L_{eff}	= distribution width for the load in top part of the web
κ_f	= reduction factor for local buckling
l_y	= effective loading length
F_d	= design concentrated load
$N_{Rd,WQ}$	= yield strength of the WQ-beam
A_{WQ}	= cross-sectional are of the WQ-beam
RHS	= rectangular hollow section
$F_{v,Rd}$	= shear resistance
f_{ub}	= ultimate tensile strength of a bolt
A	= cross-sectional area
A_s	= cross-sectional area of one bolt in area of screw thread
γ_{M2}	= partial factor
f_u	= ultimate tensile strength
d	= diameter
d_0	= diameter of the hole
t	= smaller thickness of the two connected materials
p_1	= distance between two holes, along the direction of the load transfer
p_1	= distance between two holes, perpendicular to the direction of the load transfer
e_1	= distance between hole and the edge of the plate, along the direction of the load transfer

e_2	= distance between hole and the edge of the plate, perpendicular to the direction of the load transfer
$V_{eff,1,Rd}$	= block tearing resistance
A_{nt}	= net area, under tension
A_{nv}	= net area, under shear
R_d	= selected resistance of the connection
y_{ov}	= over strength factor
R_{fy}	= resistance of a member

1 INTRODUCTION

1.1 General Information

The purpose of this thesis work was to create and design an alternative solution(s) for the current roof truss of engine halls with 32/34 engines and radiators on the roof. The current roof truss is a relatively new solution and has been taken into use when radiators were designed so that they are on top of the roof. Coming as a totally new layout design on civil perspective, the current truss with radiators on top of it is not the finest and structurally the most effective solution. Radiators are going to be on the roof of the engine hall in the most part of Wärtsilä's upcoming projects. This solution provides the best efficiency ratio for the radiators and also saves a lot of land space of the site. Because Wärtsilä is a global company and supplies power plants all over the world, in every project there are different loads and climate conditions depending on the country where the project is delivered. In this thesis all the characteristic values, such as wind speed and earthquake magnitude according to Borneo, Indonesia, have been taken into account. Indonesia is an active country for Wärtsilä power plant deliveries.

1.2 Company

Wärtsilä Corporation is a worldwide leading company in providing energy solution to its customers. The company supplies power solutions for marine and power plant markets. Wärtsilä has close to 19 000 employees in 70 countries around the world. Wärtsilä can offer different project types for customers from material delivery to turnkey projects. Professional project management is one of Wärtsilä's strengths on the markets. /1/

1.3 Power Plants

Wärtsilä supplies power plants globally. Wärtsilä's leading technology and its wide selection of solutions gives a variety of choices for the customers. Power plants can be running on oil, gas and multi-fuel engines. Selection of engine sizes together with multiple fuel options gives Wärtsilä the leading position on lifecycle energy markets. /1/

2 STANDARDS

2.1 Eurocodes

The Eurocodes are harmonized building standards within Europe. The Eurocodes include ten different codes from EN 1990 to EN 1999. These different codes are common reference codes in Europe and it is mandatory for all EU countries to design according to the Eurocodes. As the Eurocodes are being reference codes for design, every country may include additional and detailed information to its National Annex. All the sections where the National Annex may be applied are marked in the Eurocodes. /3/

Calculations and design in this thesis are done according to the Eurocodes, despite the fact that the design building will be located in Asia. This point has to be considered in mentioned parts of the design. Seismic loads are calculated according to EN 1998. In most Wärtsilä projects, seismic calculations are done according to UBC 1997 (Uniform Building Code). All the values that are related to seismic calculations have been selected based on building location and corresponding values in UBC 1997.

2.2 UBC – Uniform Building Code

The UBC is a series of building codes that includes three volumes. The Uniform Building Code is the most used building standard in the United States of America. The UBC has been adopted widely in the US because it meets government's necessities. The three volumes of the UBC define everything from administrative provisions to structural engineering regulations and installation requirements. /4/

The Uniform Building Code is not the design code that is used in this thesis due to the fact that the Eurocodes are the standards used in Finland and therefore also applied in this thesis.

3 PROGRAMS

3.1 Windows Excel

Windows Excel is Microsoft's table calculation program. In this thesis it is used for wind calculations, diagonal member calculations and connection calculations.

3.2 Mathcad

Matchcad is a computer based calculation program by PTC. It is suitable for almost all kind of calculations, especially for engineering and scientific use. In this thesis Mathcad is used to calculate and to draw graphs for response spectrums.

3.3 Autodesk Robot Structural Analysis Professional

Autodesk Robot Structural Analysis Professional is a structural modeling and calculation program. In this thesis, it is used for creating a static structural model of one module line of the engine hall. *Finite element method* (FEM) calculations are made with this program.

4 PROJECT

Location:

- The project is located in Indonesia on the island of Borneo.

Year:

- The project has started in the late 2013 and is scheduled to be finished in 2014.

Engines:

- The engine type is 32/34 and the project contains 16 engines which are divided into two similar engine halls.

Engine hall measurements:

- One engine hall is 65 m wide, 21.45 m deep and its total structural height is 12.935 m.

Radiators:

- Radiators are located on the roof of engine halls.

5 LOADS

Loads are basically actions that impose forces to the structure. Loads are divided into sub-categories of *dead loads*, *live loads*, *accidental loads* and *seismic loads*. Load cases and combinations are determined by combining actions. These cases have to be formed by taking together actions which are able to act at the same time. Every combination has a leading variable action and accompanying variable actions. Accompanying variable actions are reduced with reduction factors according to the Eurocodes. Every combination has also permanent actions. Accidental and seismic combinations are made according to the Eurocode EN 1990. /2/

Permanent or dead loads are actions which are assumed to act during the whole time of the designed building age and they are assumed to remain unchanged. Permanent loads are mainly self-weight of structures and other permanent parts of the building. /2/

Live or imposed loads are actions that vary during time. They are categorized according to building types and building importance. Most common variable actions are people, furniture, wind and snow. /2/

In addition to these load types, there are installation loads. Installation loads are actions that occur and act during construction and have to be considered in the design of structures. /2/

5.1 Dead Loads

Dead loads which are taken into account in calculations are gathered from Wärtsilä standard loads. In these calculations dead loads consists of self weight of the frame, roof panels and radiators. The weight of the roof panels has been converted into *uniform load* for the lower chord of the roof truss. The weight of the radiators has been divided and converted into five *concentrated loads* for the upper chord of the roof truss, according to locations of radiator support beams. These conversions have been made because calculations and modeling are done with only one module of the Engine hall. Module spacing is 5.4 m.

5.1.1 Roof Panels

150 mm roof panels = 0.5 kN/m^2

The uniform load for the lower chord of roof truss =>

$$q = 0{,}5\frac{kN}{m2} \times 5{,}4\ m = 2{,}7\frac{kN}{m}$$

5.1.2 Radiators

The operating weight per radiator unit is 4370 kg. In calculation this weight is converted to 50 kN.

In this example project there are five radiator support beams for the radiator. The radiators are in a line along the building length on both edges of the roof. The width of one radiator unit is 2.5 m.

Two edge support beams =>

$$Q = \left(\frac{50\ kN}{(3\ \times 2\ \times 2{,}5\ m\ )}\right) \times 5{,}4\ m = 18\ kN$$

Three support beams in the middle =>

$$Q = \left(\frac{50\ kN}{(3\ \times 2{,}5\ m\ )}\right) \times 5{,}4\ m = 36\ kN$$

5.1.3 Radiator and Roof Fan Service Platform Steels

The weight of the platform steels is approximately 450 kg/m. In calculation this weight is converted to 4.5 kN/m. This weight is applied to two beams under the service platforms.

$$Q = \left(\frac{4,5 \frac{kN}{m}}{2} \right) \times 5,4\, m = 12,42\, kN$$

5.2 Live Loads

Live loads consist of wind and standard live load on the roof. Wind actions are calculated based on Wärtsilä standard wind speed of 100 mph (44.5 m/s). The height of the 32/34 engine hall is 12.9 m including roof truss and radiator supports. Live load on the roof is 0.75 kN/m^2.

5.2.1 Wind

Peak velocity pressure

Peak velocity pressure is the biggest pressure that wind causes to the surface of the building, calculated at different heights. In the Eurocodes, peak velocity pressure is marked as $q_p(z)$. /5/

$$q_p(z) = [1 + 7 * I_v(z)] * \frac{1}{2} * \rho * v^2{}_m(z)$$

qp(z) = peak velocity pressure at height z

$I_v(z)$ = turbulence intensity at height z

ρ = air density

$v_m(z)$ = mean wind velocity at height z

Wind actions are calculated based on wind speed of 30 m/s. When calculating the peak velocity pressure according to expression for $q_p(z)$, the factor 7 is based on the Eurocodes calculations of *peak factor*. Peak factor calculations are based on structure frequencies, mean wind velocities and aerodynamic functions. Peak factor of 3.5 is applied to get the factor 7 in the expression for $q_p(z)$. This factor 7 is used in calculations for this thesis and it has not been checked with more specific calculations. Turbulence intensity, air density and mean wind velocity are defined according to the Eurocodes and specific values are in the attached calculations. /5/

Wind pressure on surfaces

Forces that wind causes to the structure can be defined as pressures on the surfaces. These pressures are calculated according to the equation below. Wind causes pressure to the external and internal surfaces and both of them are considered to be acting at the same time. In calculations wind direction is either 0° or 90° in respect to the building facade. External and internal pressures are calculated with the same equation only with the change of pressure coefficient. Pressure coefficients are interpolated according to tables in the Eurocode 1991-1-4. Interpolations are done by relation between the height and depth of the building for the walls and by the pitch angle of the roof when calculating the pressures for the roof. The specific calculations are found attached.

$$w_e = q_p(z_e) * c_{pe}$$

w_e = pressure on external surface [kN/m2]

$q_p(z_e)$ = peak velocity pressure on external surface at height z

c_{pe} = pressure coefficient for external the external pressure

In this thesis, wind loads are modeled by the following method.

- Walls: As a uniform load gathered from external wind pressure on surface.
- Roof truss/radiator support structure: As two concentrated loads, acting to the lowest point of the roof truss and to the highest point of the truss.

5.2.2 Crane Load

Crane loads are most commonly seen in industrial halls. There can be several cranes in one building which are supported by crane beams. Cranes create wheel load and transverse wheel load for the supporting beams. There are cranes of different weight depending on their lifting capacity. For example, in this thesis crane loads are calculated based on two-two-ton cranes.

5.3 Seismic Loads

Seismic design has to be applied in the regions where earthquakes and seismicity are possible events to occur. When designing buildings for the seismic resistance according to the Eurocodes, the National Annexes of each country gives their own input to the design. The national Annex was not available for this thesis because the project is located outside Europe. In calculations of the seismic design in this work, recommended values by the Eurocodes are applied for all the sections where the National Annex could have been used. /6/

Design seismic action is the response of the structure under seismic event calculated according to the standards. Earthquakes, which are the most common events caused by seismicity, can affect different kinds of forces for the structure from ground motion to ground acceleration and time-history representation. Mostly used force caused by earthquakes among structural engineering is *peak ground acceleration* (PGA). It is calculated according to horizontal and vertical components of ground acceleration and is comparable with the probability to occur every 50 years. Using FEM calculations, these forces of seismicity are modeled and calculated according to response spectrums of elastic analysis and modal analysis. *Design spectrum for elastic analysis* is the most commonly used analysis method nowadays. /6/

Even though calculations can and should be done to protect the structures in case of seismic event, they are all based on saving human lives in situations of earthquakes and other seismic events. In addition to saving human lives, seismic design is done according to requirements of no global or local collapse and to requirement that the bearing structures stay operational for more seismic events than it is designed. /6/

5.3.1 Elastic Response Spectrum

Elastic response spectrum is a graph which describes the maximum response value of *single-degree-of-freedom* (SDOF) system for certain excitation as function of *fundamental period*. There are two types of elastic spectrums and they have to be checked in both horizontal and vertical directions. The choice between spectrum type 1 and type 2 depends on the magnitude level on the surface and it can be defined in the National Annex. In this thesis, calculations are done with both types. Vertical component is calculated with the same equations, only the *design ground acceleration* a_{gd} is changed to a_{vg} according to a_{vg} / a_{gd} = 0.45 / 0.90. /7/ /11/ See Figure 1.

$0 < T < T_B$: $S_e(t) = a_{gd} * S * [1 + T/T_B * (\eta * 2{,}5 + 1)]$

$T_B < T < T_C$: $S_e(t) = a_{gd} * S * \eta * 2{,}5$

$T_C < T < T_D$: $S_e(t) = a_{gd} * S * \eta * 2{,}5 * [T/T_C]$

$T_D < T$ 4s: $S_e(t) = a_{gd} * S * \eta * 2{,}5 * [T_C * T_D / T^2]$

Where:

$S_e(t)$ = elastic response spectrum at time t

a_{gd} = design ground acceleration

S = soil factor

T = vibration period

η = damping factor

T_B, T_C, T_D = limits of spectral acceleration

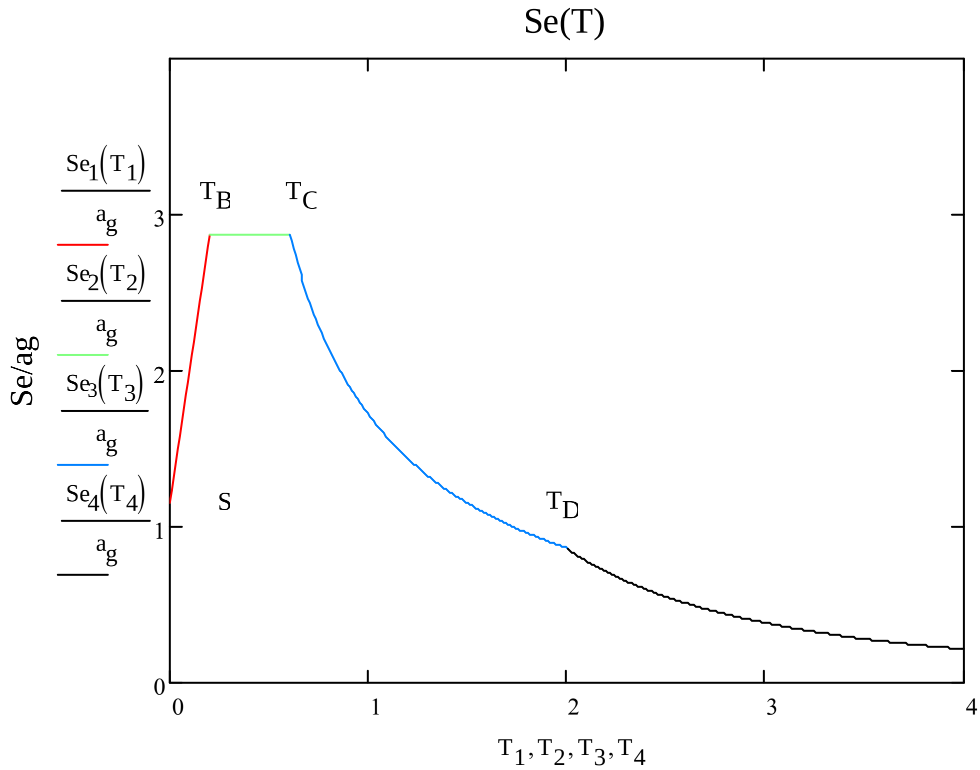


Figure 1. Elastic response spectrum, Type 1, horizontal direction

5.3.2 Design Response Spectrum for Elastic Analysis

Design response spectrum is used to model seismic forces especially in FEM calculation. Design spectrum differs from linear elastic response spectrum in a sense that it takes in consideration the structures capability to stand against the seismic forces. This is taken into account with *behavior factor* q (5.3.3). The behavior factor depends on the ability of the structure to dissipate energy which is caused by the seismic event and it can be defined according to various parts of the EN 1998. Design spectrum is corresponding with elastic spectrum, only reduced with behavior factor, and it is developed to evade rigorous inelastic analysis. /6/

Behavior factor is selected according to the EN 1998.

$0 < T < T_B$: $S_d(t) = a_{gd} * S * [2/3 + T/T_B * (2{,}5/q - 2/3)]$

$T_B < T < T_C$: $S_d(t) = a_{gd} * S * 2{,}5/q$

$T_C < T < T_D$: $S_d(t) \begin{cases} = a_{gd} * S * 2{,}5/q * [T_C/T] \\ \geq \beta * a_{gd} \end{cases}$

$T_D < T$ 4s: $S_d(t) \begin{cases} = a_{gd} * S * 2{,}5/q * [T_C * T_D/T^2] \\ \geq \beta * a_{gd} \end{cases}$

Where:

$S_d(t)$ = design elastic spectrum

q = behavior factor = 1.5

β = lower bound factor = 0.2

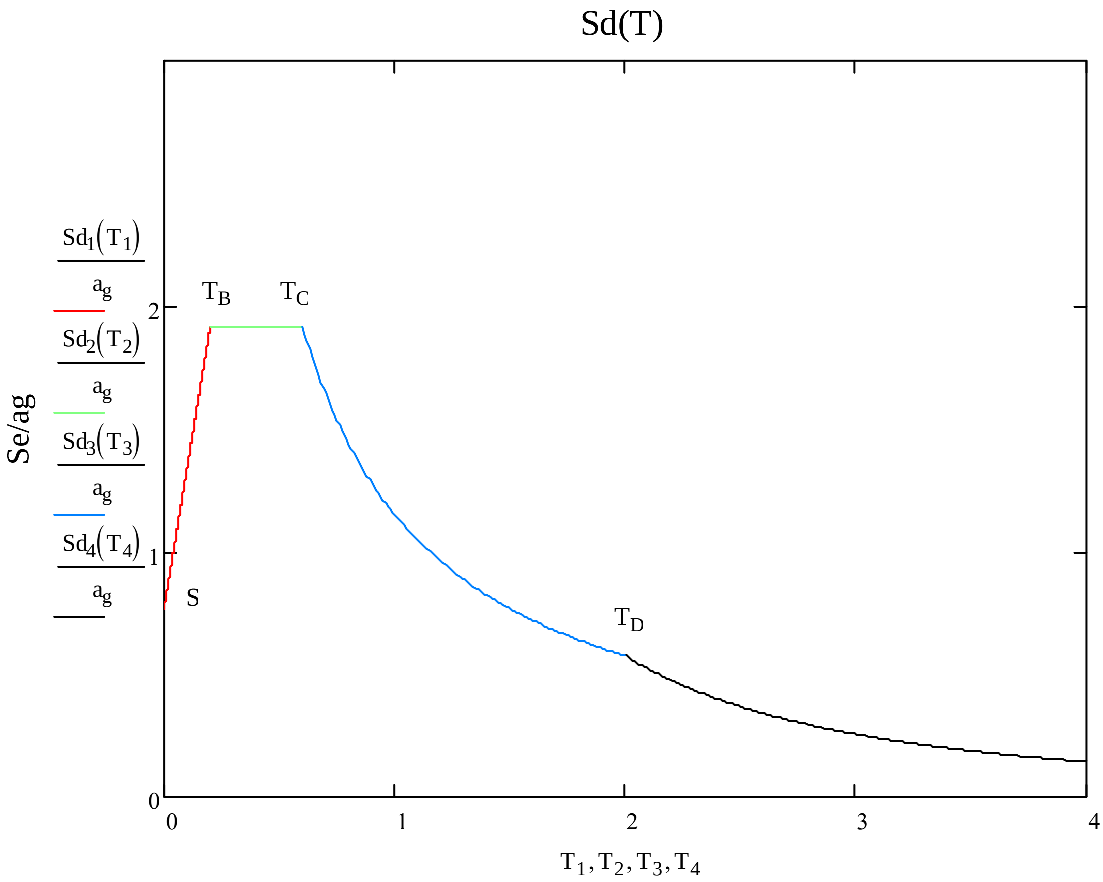


Figure 2. Design spectrum for elastic analysis

5.3.3 Behavior Factor

The behavior factor q is mentioned and defined in various parts of the EN 1998. Basically the behavior factor takes into consideration the full theoretical elastic response of the structure, including damping, to seismic forces and compares it to the response of the structure to seismic forces with elastic response designing values. This ratio of behavior factor is an approximation and as simplified it reflects for the capability of the structure to absorb energy caused by seismic even. It is given to different materials separately. /7/

The decision to use the behavior factor of 1.5 was decided according to the EN-1998-1-4 section 6.3.2. The structures belong to ductility class medium and cross-sectional classes one and two. With this information the behavior factor could have been up to 4, however that would have required more accurate justifications. Using the behavior factor of 1.5 ensures that analysis stay on satisfying results. /8/

5.3.4 Soil Type

The soil type is determined according to the *shear wave velocity, standard penetration tests* and *compression strength values*. In the Eurocodes soil types are divided into seven different categories from bed rock to very loose soils. The soil type has an effect on forces that a seismic event causes to the structure. The effects are shown in the response spectrums.

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,05	0,25	1,2
B	1,35	0,05	0,25	1,2
C	1,5	0,10	0,25	1,2
D	1,8	0,10	0,30	1,2
E	1,6	0,05	0,25	1,2

Table 1. Effects of different soil types on the response spectrum values. /7/

The soil type used in this thesis is C. The choice to use this soil type is based on the corresponding type and values of UBC 1997. This corresponding soil type is marked as S_D in UBC 1997 and its general description is stiff soil. This is the assumption of which soil type to be used in Wärtsilä projects before more specific ground inspections are made. /4/

6 ANALYSIS

6.1 Lateral Force Method

The lateral force method is applicable for simple structures where only the first modes are significant, meaning that modes for structure, which are caused by vibration of higher frequencies, do not affect any prominent deformation. The idea of lateral force method is to make a simplified modeling for seismic analysis, taking static forces which will substitute the worst case dynamic analysis and deformation for structure caused by it. The design spectrum of elastic analysis is used to get acceleration at structures lowest *fundamental period of vibration*. The EN 1998 determines lateral force or base shear force with the following equation. /7/

Equation for base shear force: /7/

$$F_b = S_d(T_1) \times m \times \lambda$$

Where:

- $S_d(T_1)$; see Figure 5
- T_1 is the fundamental period
- m is the buildings mass
- λ the correction factor (EN 1998)

The fundamental period can be defined for short structures (< 40m) with the following equation: /7/

$$T_1 = C_{t} * H^{3/4}$$

In which:

- H is the building height (m)
- C_t is factor depending on structure type (EN1998)

The requirement for the fundamental period is: /7/

$$T_1 \leq \begin{cases} 4 * T_c \\ 2\ s \end{cases}$$

6.2 Modal Analysis

When vibrations caused by external factors get in touch with a structural system, the structure will start to vibrate. At some specific frequency of vibration the whole structure will change its shape. This will be repeated with countless of frequencies and with each of them the structure will move to a different shape. The modal analysis in structural engineering is the tool that is used to calculate and estimate these shapes and the frequencies that cause them. Frequencies at which the structure will move are called *natural frequencies* and different shapes that structure will form at each frequency are *modes*. Each structure has multiple natural frequencies.

Each structure has a mass and stiffness. In addition structural systems may have dampers. Natural frequencies and corresponding modes of the structure are calculated in *modal analysis* using mass, stiffness and damping factors.

6.2.1 Single-Degree-of-Freedom System

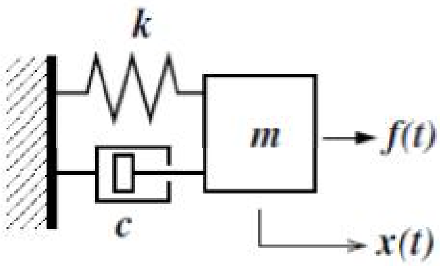


Figure 3. Single-degree-of-freedom system (SDOF). /10/

In the single-degree-of-freedom system the contents of mass, stiffness, damping, movement and force are the acting parts. Mass can only move to a certain direction. It is the simplest way to describe structure for the modal analysis, but in reality there are rarely structures which can be defined as single-degree-of-freedom systems.

The equation for the single-degree-of-freedom system is: /10/

$$m\ddot{x}(t) + c\dot{x}(t) + kx(t) = f(t)$$

In which:

- *m* represents mass
- *c* represents damping
- *k* represents stiffness
- *x* represents displacement (in certain direction) of mass
- $\dot{x}$ represents displacements first time derivative, velocity
- $\ddot{x}$ represents displacements second time derivative, acceleration
- *f* represents excitation force

The homogenous solution of equation and the natural frequency can be solved by taking *f* (t) as 0 and calculating it with differential theory. With the help of Laplace variable, the roots for characteristic equation and furthermore the damped natural frequency can be solved.

6.2.2 Multiple-Degree-of-Freedom System

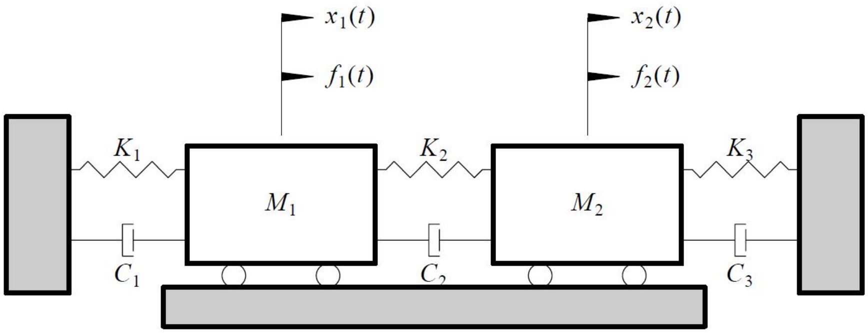


Figure 4. Multiple-degree-of-freedom system. This picture is example for two-degrees. /9/

The multiple-degree-of-freedom (MDOF) system demonstrates the real structures in the most accurate way. In MDOF one mass content can have six *degrees of* *freedom* which means that it is allowed to move in x, y and z direction and twist around x, y and z axels. In MDOF, the calculation will be done in a way that instead of just single content of mass, damping, stiffness and force, there will be formed matrixes for all of the contents. In figure 6 there is an example of the equation for figure 5, two-degree-of-freedom system:

$$\begin{bmatrix} M_1 & 0 \\ 0 & M_2 \end{bmatrix}\begin{bmatrix} \ddot{x}_1 \\ \ddot{x}_2 \end{bmatrix}+\begin{bmatrix} (C_1+C_2) & -C_2 \\ -C_2 & (C_2+C_{3)} \end{bmatrix}\begin{bmatrix} \dot{x}_1 \\ \dot{x}_2 \end{bmatrix}+\begin{bmatrix} (K_1+K_2) & -K_2 \\ -K_2 & (K_2+K_3) \end{bmatrix}\begin{bmatrix} x_1 \\ x_2 \end{bmatrix}=\begin{bmatrix} f_1 \\ f_2 \end{bmatrix}$$

Figure 5. Matrix equation for two-degree-of-freedom system. /9/

The general form of MDOF equation is: /9/

$$[M]\ddot{x} + [C]\dot{x} + [K]x = [F]$$

In which:

- M is the mass matrix
- C is the damping matrix
- K is the stiffness matrix
- F is the structure excitation matrix

In general, calculations are done with computer programs, for example with FEM programs. The modal analysis for structures demands several degrees of freedom which leads to complex matrices and calculation estimations. Most of the structural engineering programs, including Autodesk Robot Structural Analysis Professional used in this thesis, are capable to execute FEM calculations.

7 STRUCTURAL DESIGNING OF ROOF TRUSS

The design of structure always starts with calculating all the forces and deformation that are caused for every structural part from external forces and loads. Each structural part has to be able to withstand and distribute forces without collapse or remarkable deflections.

The roof truss is designed to take vertical loads which are on top of the roof or hanging from it inside the building. The truss is a good structural solution when a relatively light structure is needed with a long span and minimum deflection, especially vertical bending. It also distributes wind loads to the bearing columns via its diagonal components.

Geometrically the roof truss can be almost any kind, except some restrictions of manufacturing point of view. Roof trusses can be made of steel or wood. The equations, terms and standards used in this thsis are for steel truss.

7.1 Diagonals

The diagonals are designed either for buckling if they are under compression or for yield if they are under tension. Diagonals do not take any moment with traditional calculations for the roof truss, even though welded connections between diagonals and chords are not fully pinned or fixed. With a standard roof truss every second diagonal is under compression and every second is under tension. In this thesis, the new geometry of the roof truss will create a so called compression arch, which leads to a situation where every second diagonal member is not compressed and every second not tensioned.

7.1.1 Compressed Diagonals

The compressed diagonals are designed for buckling. The buckling resistance of structural member depends on its length, slenderness, procedure of manufacturing and reduction factor. The following equation needs to be fulfilled:

$$N_{Ed} \leq N_{b,Rd}$$

Where:

- N_{Ed} is the designing compressive strength
- $N_{b,Rd}$ is the designing buckling resistance under compression

$N_{b,Rd}$ is defined with following expression for cross-sectional classes 1,2 and 3:

$$N_{b,Rd} = \kappa \times A \times f_y / \gamma_{M1}$$

Where:

- A is profiles cross-sectional area [mm2]
- f_y is materials yield strength
- γ_{M1} is partial factor. For steel it is 1.0.
- κ is reduction factor

κ is calculated according to the Eurocodes with following expression:

$$\kappa = 1 \div (\phi + \sqrt{\phi^2 - \lambda_k{}^2})$$

κ needs to be smaller than 1.

Where:

- ϕ is an equation that takes buckling curve and non-dimensional slenderness into consideration. The EN 1993-1-1.
- λ_k is non-dimensional slenderness. The EN 1993-1-1.

7.1.2 Diagonals Under Tension

The diagonals under tension are designed for the yield strength of the member. The following expression needs to be fulfilled.

$$N_{Ed} \leq N_{Rd}$$

Where:

- N_{Ed} is designing yield force

- N_{Rd} is yield strength of diagonal member

N_{Rd} is calculated according to following expression:

$$N_{Rd} = f_y \times A$$

When the designing yield force is known from the static model, needed cross-sectional area can be calculated from the equation above with the following expression:

$$A = N_{Rd} \div f_y$$

After this, suitable cross-section can be chosen.

7.2 Verticals

The verticals are designed for buckling using the same expressions as were used for diagonals under compression. The biggest normal force for verticals affects at the both ends of the roof truss. All the other verticals are affected by a lot smaller forces. Despite of the different forces affecting among the verticals, they are all selected to be the same profiles with the end verticals. This is done according to instructions from Wärtsilä and the main reason is to ease the manufacturing of the truss and to keep extra safety in the design.

7.3 Upper Chord

The upper chord is designed for bending and compression. The Eurocodes has a pattern which has to be followed when bending and compression are affecting at the same time. The following equation is determined in the Eurocodes for members under axial compression and bending.

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

Figure 6. Equation for a member which is under axial compression and bending. /12/

Where:

- N_{Ed} is the design compressive force
- $M_{y,Ed}$ is the design bending moment
- x_y is the reduction factor (buckling regarding to y-axis)
- x_{Lt} is the reduction factor (lateral torsional buckling)
- k_{yy} is the interaction factor according to the EN 1993

When a member is supported so that buckling cannot happen along the weak axis of the member, the following equation can be adapted.

$$\frac{N_d}{\frac{\kappa_y N_{Rk}}{\gamma_{M1}}} + k_{yy} \frac{M_{yd} + \Delta M_{yd}}{\frac{\kappa_{LT} M_{y,Rk}}{\gamma_{M1}}} \leq 1.0$$

Figure 7. Equation for a member which is under axial compression and bending, and buckling along the weak axel of the member is prevented. /13/

Where:

- κ_y is the reduction factor (buckling regarding to y-axis)
- κ_{Lt} is the reduction factor (lateral torsional buckling)

Reduction factors κ_y and κ_{Lt} are calculated according to following equations. /12/

$$\kappa_y = 1 \div \left(\phi + \sqrt{\phi^2 - \lambda_k{}^2}\right) \leq 1$$

κ_y determined earlier as κ in section 7.1.1.

$$\kappa_{Lt} = 1 \div \left(\phi_{LT} + \sqrt{\phi_{LT}{}^2 - \lambda_{LT}{}^2}\right) \leq 1$$

Where:

- $\phi_{Lt}=0.5\times[1+\alpha_{LT}(\lambda_{LT}-0.2)+\lambda_{LT}{}^2]$
- α_{Lt} is an imperfection factor given in table 6.3 in the EN 1993
- $\lambda_{LT} = \sqrt{W_y \times f_y \div M_{cr}}$
- Mcr is the elastic critical moment

The value of the interaction factor k_{yy} is determined according to the Eurocode 1993 Annex B and the alternative method 2.

Table B.1: Interaction factors k_{ij} for members not susceptible to torsional deformations

Interaction factors	Type of sections	Design assumptions	
		elastic cross-sectional properties class 3, class 4	plastic cross-sectional properties class 1, class 2
k_{yy}	I-sections RHS-sections	$C_{my}\left(1+0{,}6\bar{\lambda}_y \frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right) \leq C_{my}\left(1+0{,}6\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$	$C_{my}\left(1+\left(\bar{\lambda}_y-0{,}2\right)\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right) \leq C_{my}\left(1+0{,}8\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$
k_{yz}	I-sections RHS-sections	k_{zz}	$0{,}6\ k_{zz}$
k_{zy}	I-sections RHS-sections	$0{,}8\ k_{yy}$	$0{,}6\ k_{yy}$
k_{zz}	I-sections	$C_{mz}\left(1+0{,}6\bar{\lambda}_z \frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right) \leq C_{mz}\left(1+0{,}6\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$	$C_{mz}\left(1+\left(2\bar{\lambda}_z-0{,}6\right)\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right) \leq C_{mz}\left(1+1{,}4\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$
	RHS-sections		$C_{mz}\left(1+\left(\bar{\lambda}_z-0{,}2\right)\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right) \leq C_{mz}\left(1+0{,}8\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$
For I- and H-sections and rectangular hollow sections under axial compression and uniaxial bending $M_{y,Ed}$ the coefficient k_{zy} may be $k_{zy} = 0$.			

Figure 8. Alternative method 2 for interaction factors. /12/

Factor C_{my} is determined according to the following table.

Table B.3: Equivalent uniform moment factors C_m in Tables B.1 and B.2

Moment diagram	range		C_{my} and C_{mz} and C_{mLT} uniform loading	concentrated load
	$-1 \le \psi \le 1$		$0,6 + 0,4\psi \ge 0,4$	
	$0 \le \alpha_s \le 1$	$-1 \le \psi \le 1$	$0,2 + 0,8\alpha_s \ge 0,4$	$0,2 + 0,8\alpha_s \ge 0,4$
	$-1 \le \alpha_s < 0$	$0 \le \psi \le 1$	$0,1 - 0,8\alpha_s \ge 0,4$	$-0,8\alpha_s \ge 0,4$
$\alpha_s = M_s/M_h$		$-1 \le \psi < 0$	$0,1(1-\psi) - 0,8\alpha_s \ge 0,4$	$0,2(-\psi) - 0,8\alpha_s \ge 0,4$
	$0 \le \alpha_h \le 1$	$-1 \le \psi \le 1$	$0,95 + 0,05\alpha_h$	$0,90 + 0,10\alpha_h$
	$-1 \le \alpha_h < 0$	$0 \le \psi \le 1$	$0,95 + 0,05\alpha_h$	$0,90 + 0,10\alpha_h$
$\alpha_h = M_h/M_s$		$-1 \le \psi < 0$	$0,95 + 0,05\alpha_h(1+2\psi)$	$0,90 - 0,10\alpha_h(1+2\psi)$

For members with sway buckling mode the equivalent uniform moment factor should be taken $C_{my} = 0,9$ or $C_{Mz} = 0,9$ respectively.

C_{my}, C_{mz} and C_{mLT} should be obtained according to the bending moment diagram between the relevant braced points as follows:

moment factor	bending axis	points braced in direction
C_{my}	y-y	z-z
C_{mz}	z-z	y-y
C_{mLT}	y-y	y-y

Figure 9. Table for C_{my}. /12/

7.3.1 Shear Buckling

The shear buckling of a no-braced web has to be checked if the following rule is fulfilled. /13/

$$h_w/t_w > (72/\eta) \times \varepsilon$$

Where:

- h_w is the height of the web (radius of the root fillet/welds not included)
- t_w is the thickness of the web
- η is 1.2 for steels with strength lower than s460
- $\varepsilon = \sqrt{235/f_y}$
- f_y is material strength

7.3.2 Resistance of a Web for Local Concentrated Loads

This case has to be checked for I-profiles. The resistance of an I-profile is determined by the slenderness of its web when load case is a local concentrated load. The resistance of the profile has to be checked either according to yielding of the web or its local buckling. /13/

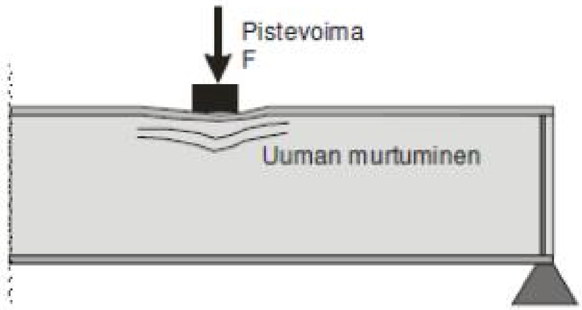


Figure 10. Yielding of the web under local concentrated load. /13/

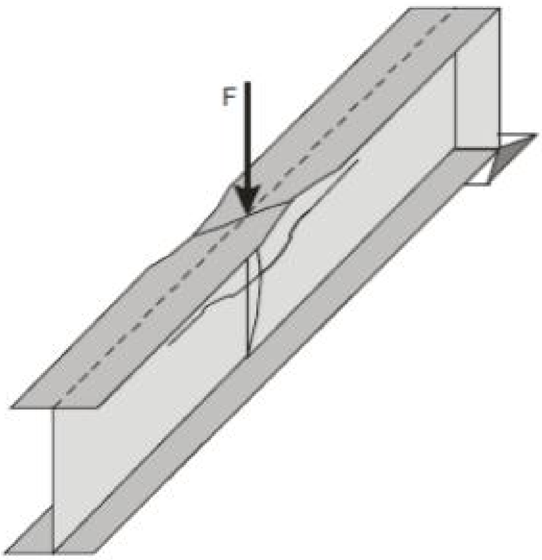


Figure 11. Local buckling of the web under concentrated load. /13/

The compressed flange has to be supported in a decent way to be able to apply the following rules given in the EN 1993. /13/

$$F_{Rd} = f_{yw} \text{ x } L_{eff} \times t_w / \gamma_{M1}$$

Where:

- F_{Rd} is the resistance of the web in relation to local buckling
- f_{yw} is yield strength of the web

- L_{eff} is the distribution width for the load in top part of the web
- t_w is the thickness of the web

L_{eff} is determined according to following expression.

$$L_{eff} = \kappa_f \times l_y$$

Where:

- κ_f is a reduction factor for local buckling
- l_y is an effective loading length

For the case below, κ_f and l_y are determined by several expressions in the EN 1993.

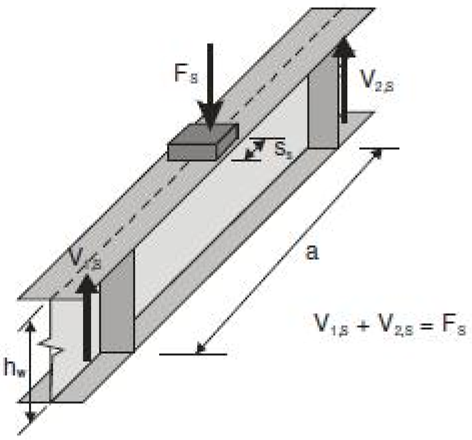


Figure 12. Concentrated load is distributed via shear of the web. /13/

The resistance of the web for local concentrated load has to be stated according to the following equation:

$$F_d < F_{Rd}$$

Where:

- F_d is the effective concentrated force

7.4 Lower Chord

The lower chord of the truss is designed for tension. The same calculation methods can be used as in diagonals under tension. With this project the lower chord is selected based on the existing roof truss used by Wärtsilä. For assembly reasons the lower chord is a WQ-beam, designed for Wärtsilä only. The tensile strength of the WQ-beam is enough to stand the normal force that is affecting in the beam.

$$N_{Rd,WQ} = f_y \times A_{WQ}$$

Where:

- $N_{Rd,WQ}$ is the yield strength of the WQ-beam
- f_y is the material strength
- A_{WQ} is the cross-sectional are of the WQ-beam

7.5 Connections

7.5.1 Welded connections

With this roof truss the following connections are welded:

- Lower chord to diagonals
- Lower chord to verticals
- Middle chord to diagonals below
- Middle chord to verticals below

The EN 1993 defines that the angle between members has to be over 30°. This is due to manufacturing reasons. Welds are hard to execute when the angle between members is smaller than 30°. However, if the workshop can prove by doing test welds that they are able to do welds tighter than 30° according to the certificates, then these tight angles are possible to be made.

The principle of welded connection of *rectangular hollow sections* (RHS) is that the weld is at least the same than the wall thickness of the weaker bar. For example, if RHS 100x100x4 is welded to RHS 140x140x6, the diameter of the weld should be 4 mm. With this principle it is ensured that the resistance of the weld is at least same as the resistance of the weaker bar.

The following figure demonstrates different failure types of welded connections between a rectangular hollow section bar and a rectangular hollow section chord.

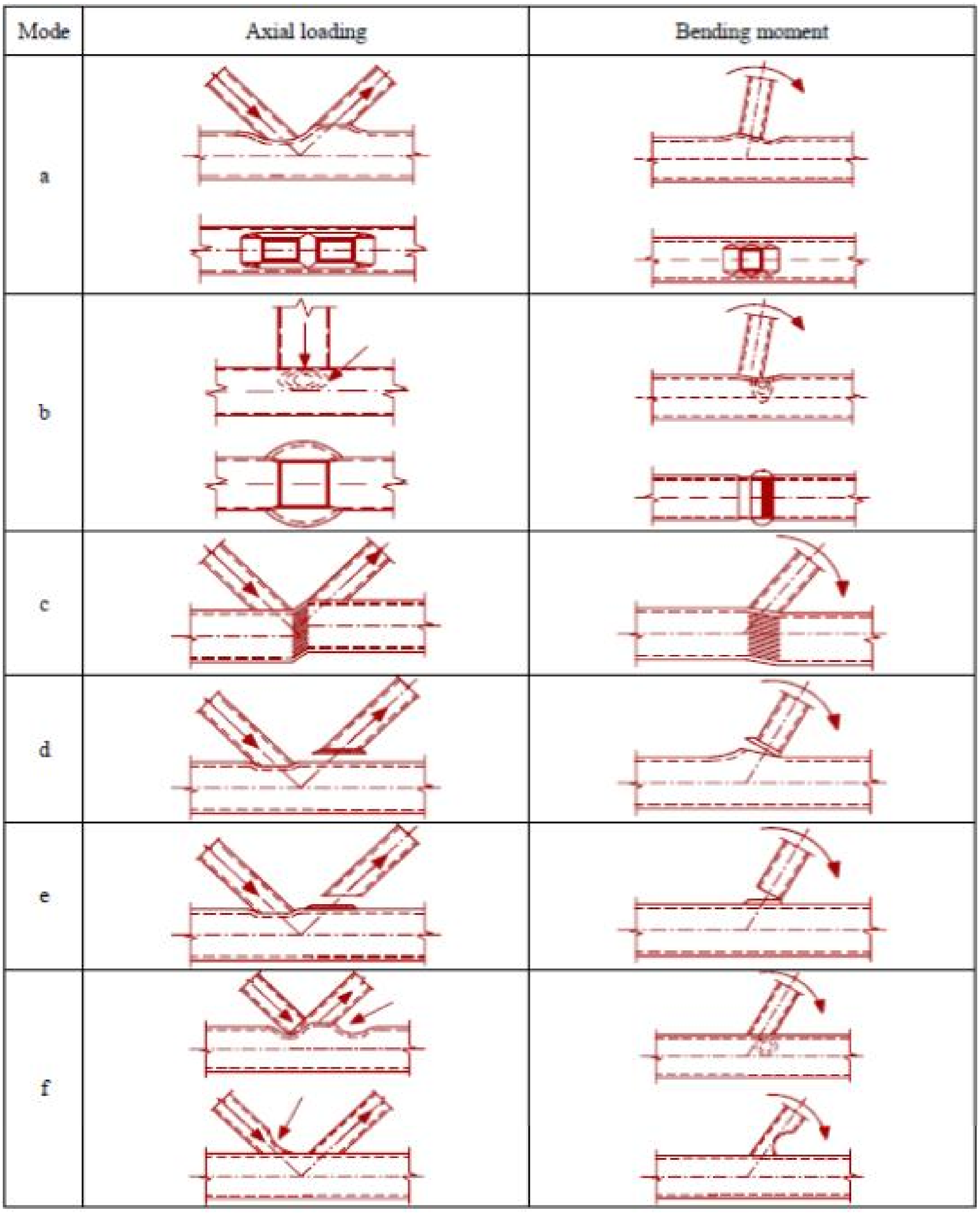


Figure 13. Different failure types of welded connections between rectangular members. /14/

Resistance of the K and N type gapped connections between bars that are hollow sections are determined in various parts in the EN 1993. The following figure is for connections between rectangular members.

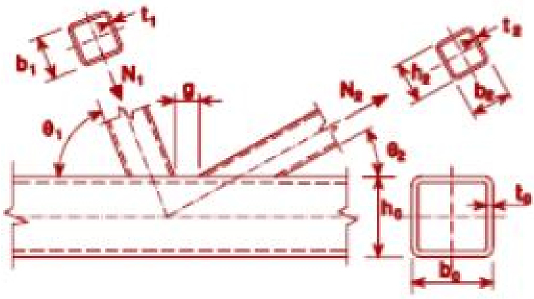
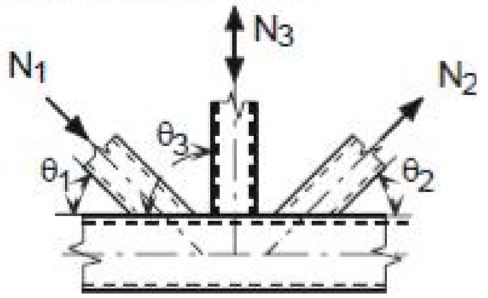
Type of joint	Design resistance $[i = 1 \text{ or } 2]$
K and N gap joints	Chord face failure
	$N_{i,Rd} = \frac{8{,}9 k_n f_{y0} t_0^2 \sqrt{\gamma}}{\sin\theta_i}\left(\frac{b_1+b_2+h_1+h_2}{4b_0}\right)/\gamma_{M5}$
	Chord shear
	$N_{i,Rd} = \frac{f_{y0}A_v}{\sqrt{3}\sin\theta_i}/\gamma_{M5}$ $N_{0,Rd} = \left[(A_0 - A_v)f_{y0} + A_v f_{y0}\sqrt{1-(V_{Ed}/V_{pl,Rd})^2}\right]/\gamma_{M5}$
	Brace failure
	$N_{i,Rd} = f_{yi}t_i\left(2h_i - 4t_i + b_i + b_{eff}\right)/\gamma_{M5}$
	Punching shear $\beta \le (1 - 1/\gamma)$
	$N_{i,Rd} = \frac{f_{y0}t_0}{\sqrt{3}\sin\theta_i}\left(\frac{2h_i}{\sin\theta_i}+b_i+b_{e,p}\right)/\gamma_{M5}$
K and N overlap joints	As in Table 7.10.
For circular braces, multiply the above resistances by $\pi/4$, replace b_1 and h_1 by d_1 and replace b_2 and h_2 by d_2.	
$A_v = (2h_0 + \alpha b_0)t_0$ For a square or rectangular brace member: $\alpha = \sqrt{\frac{1}{1+\frac{4g^2}{3t_0^2}}}$ where g is the gap, see Figure 1.3(a). For a circular brace member: $\alpha = 0$	$b_{eff} = \frac{10}{b_0/t_0}\frac{f_{y0}t_0}{f_{yi}t_i}b_i$ but $b_{eff} \le b_i$
	$b_{e,p} = \frac{10}{b_0/t_0}b_i$ but $b_{e,p} \le b_i$
	For $n > 0$ (compression): $k_n = 1{,}3 - \frac{0{,}4n}{\beta}$ but $k_n \le 1{,}0$ For $n \le 0$ (tension): $k_n = 1{,}0$

Figure 14. Equations for connection resistance. /14/

For KT type of connections the following changes have to be made for the equations.

The member 1 is always in compression and member 2 is always in tension.



$$N_{1,Ed}\sin\theta_1 + N_{3,Ed}\sin\theta_3 \le N_{1,Rd}\sin\theta_1$$
$$N_{2,Ed}\sin\theta_2 \le N_{1,Rd}\sin\theta_1$$

where $N_{1,Rd}$ is the value of $N_{1,Rd}$ for a K joint from Table 7.12, but with $\frac{b_1+b_2+h_1+h_2}{4b_0}$

replaced by: $\frac{b_1+b_2+b_3+h_1+h_2+h_3}{6b_0}$

Figure 15. Changes for KN type connections caused by KT connection. /14/

The resistance of welded connections between the chords and brace members can also be checked for the combined effect of normal force and bending moment, even though it was mentioned earlier that in the classic design of trusses these connections are assumed to be pinned. When calculating this combined effect, the following condition has to be fulfilled.

$$\frac{N_{i,Ed}}{N_{i,Rd}} + \frac{M_{ip,i,Ed}}{M_{ip,i,Rd}} + \frac{M_{op,i,Ed}}{M_{op,i,Rd}} \le 1{,}0$$

Figure 16. Equation for combined effect of normal force and moment. /14/

In the following figure are the equations given by the EN 1993.

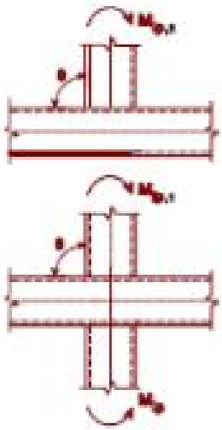
T and X joints	Design resistance
In-plane moments ($\theta = 90°$)	Chord face failure $\beta \le 0{,}85$
	$M_{ip,1,Rd} = k_n f_{y0} t_0^2 h_1 \left(\frac{1}{2\eta} + \frac{2}{\sqrt{1-\beta}} + \frac{\eta}{1-\beta} \right) / \gamma_{M5}$
	Chord side wall crushing $0{,}85 \le \beta \le 1{,}0$
	$M_{ip,1,Rd} = 0{,}5 f_{yk} t_0 (h_1 + 5t_0)^2 / \gamma_{M5}$ $f_{yk} = f_{y0}$ for T joints $f_{yk} = 0{,}8 f_{y0}$ for X joints
	Brace failure $0{,}85 \le \beta \le 1{,}0$
	$M_{ip,1,Rd} = f_{y1} \left(W_{pl,1} - (1 - b_{eff} / b_1) b_1 h_1 t_1 \right) / \gamma_{M5}$
Out-of-plane moments ($\theta = 90°$)	Chord face failure $\beta \le 0{,}85$
	$M_{op,1,Rd} = k_n f_{y0} t_0^2 \left(\frac{h_1(1+\beta)}{2(1-\beta)} + \sqrt{\frac{2b_0 b_1 (1+\beta)}{1-\beta}} \right) / \gamma_{M5}$
	Chord side wall crushing $0{,}85 \le \beta \le 1{,}0$
	$M_{op,1,Rd} = f_{yk} t_0 (b_0 - t_0)(h_1 + 5t_0) / \gamma_{M5}$ $f_{yk} = f_{y0}$ for T joints $f_{yk} = 0{,}8 f_{y0}$ for X joints
	Chord distortional failure (T joints only) *)
	$M_{op,1,Rd} = 2 f_{y0} t_0 \left(h_1 t_0 + \sqrt{b_0 h_0 t_0 (b_0 + h_0)} \right) / \gamma_{M5}$
	Brace failure $0{,}85 \le \beta \le 1{,}0$
	$M_{op,1,Rd} = f_{y1} \left(W_{pl,1} - 0{,}5(1 - b_{eff} / b_1)^2 b_1^2 t_1 \right) / \gamma_{M5}$
Parameters b_{eff} and k_n	
$b_{eff} = \frac{10}{b_0 / t_0} \frac{f_{y0} t_0}{f_{y1} t_1} b_1$ but $b_{eff} \le b_1$	For $n > 0$ (compression): $k_n = 1{,}3 - \frac{0{,}4n}{\beta}$ but $k_n \le 1{,}0$ For $n \le 0$ (tension): $k_n = 1{,}0$
*) This criterion does not apply where chord distortional failure is prevented by other means.	

Figure 17. Moment resistance of rectangular hollow section members. /14/

7.5.2 Bolted Connections

With this roof truss the following connections are bolted.

- Middle chord to upper diagonals
- Middle chord to upper verticals
- Upper chord to diagonals
- Upper chord to verticals
- Lower chord to columns
- Middle joint between the two trusses
- Middle joint between the two upper middle chords

All the bolted connections in this thesis are designed as pinned connection, which means that they are not transmitting moments. This means that the bolted connections need to be designed for shear force. The resistance of bolted connection is determined according to the EN 1993 with the following expressions. /14/

Shear resistance of a bolt: /13/

$$F_{v,Rd} = 0{,}6 \times f_{ub} \times A(A_s) \,/\, \gamma_{M2}$$

Where:

- $F_{v,Rd}$ is the shear resistance (one shear plane)
- 0,6 is reduction factor (bolts up to 8.8)
- f_{ub} is the ultimate tensile strength of one bolt
- A is the cross-sectional area of one bolt
- A_s is the cross-sectional area of one bolt in area of screw thread
- γ_{M2} is material safety factor

Tension resistance of a bolt: /13/

$$F_{t,Rd} = 0{,}9 \times f_{ub} \times A_s \,/\, \gamma_{M2}$$

Bearing resistance: /13/

$$F_{b,Rd} = k_1 \times \alpha_b \times f_u \times d \times t / \gamma_{M2}$$

Where:

- k_1 : Direction: Perpendicular to the load transfer
  - For edge bolts:
    - smallest of the following values:
      - $2{,}8 \times e_2 / d0 - 1{,}7$
      - 2,5
  - For other than edge bolts:
    - smallest of the following values:
      - $1{,}4 \times p_2 / d0 - 1{,}7$
      - 2,5
- α_b : Direction: Along load transfer
  - Smallest of the following values:
    - $e_1 / 3d0$
    - $p_1 / 3d0 - 1/4$
    - f_{ub} / f_u
    - 1,0
- f_u is the ultimate tensile strength of the material in which the bolts are connected
- d is the diameter of the bolt
- d_0 is the diameter of the hole
- t is the smaller thickness of the two connected materials
- p_1 is the distance between two holes, along the direction of the load transfer $\geq 2{,}2 \text{ x } d_0$, but smaller than 14 x t or 200 mm
- p_1 is the distance between two holes, perpendicular to the direction of the load transfer $\geq 3{,}0 \text{ x } d_0$, but smaller than 14 x t or 200 mm
- e_1 is the distance between the hole and the edge of the plate in which the bolts are connected, along the direction of the load transfer $\geq 1.2 \text{ x } d_0$

- e_2 is the distance between the hole and the edge of the plate in which the bolts are connected, perpendicular to the direction of the load transfer ≥ 1,5 x d_0

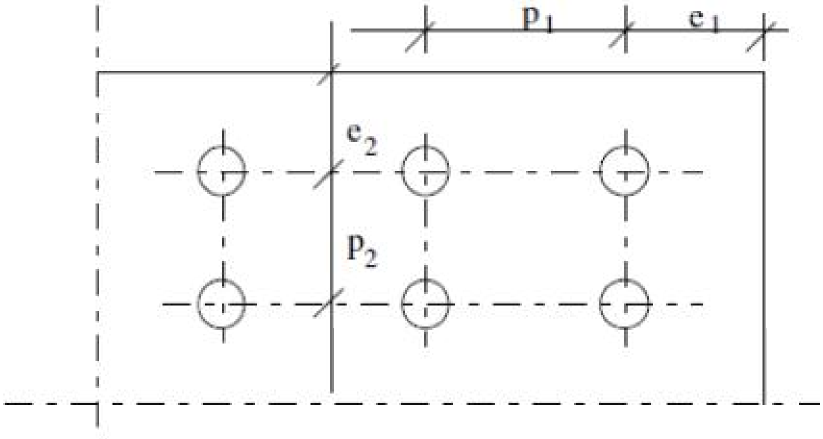


Figure 18. Explanation for bolt connection distances. Direction of load transfer is horizontal. /13/

Block tearing of bolted connection (concentric load): /14/

$$V_{eff,1,Rd} = f_u \times A_{nt} / \gamma_{M2} + (1/\sqrt{3}) \times f_y \times A_{nt} / \gamma_{M0}$$

Where:

- A_{nt} is the net area which is under tension
- A_{nv} is the net area which is under shear

The EN 1998 gives a condition for connections, which needs to be fulfilled for structures that have to be designed for seismic loads.

$$R_d \geq 1,1 \times y_{ov} \times R_{fy}$$

Where:

- R_d is the selected resistance of the connection determined according to the equations above
- y_{ov} is over strength factor. Selected according to either the National Annex or the recommended value of 1,25 given in the EN 1998

- R_{fy} is the resistance of the member that is connected, determined according to plasticity

8 CONCLUSION

The new geometry of the roof truss has many benefits compared to the existing one. It has many structural benefits and in addition to that it is lighter than the existing truss that leads also for cost savings.

The new truss is over 500 kg lighter than the existing truss. It does not sound like much, but when one engine hall can contain dozens of module lines, the savings in weight of the steel amount increases quite a lot.

But the cost savings were just a good addition. The real benefits came with the new geometry of the truss. The existing model of the roof truss when the radiators are on top of the engine hall uses less than half of the possible height for the truss. With this new model the entire height that is possible to use is actually used. With this change the stiffness of the whole module line increased radically. Also the first modes and natural frequencies of the structure went up. This is an important factor regarding that now there is a bigger gap between the first significant natural frequency of the structure and the frequency caused by the running engine.

Also the support reactions of the whole structure went down about 10 %. That gives an opportunity to check the possible reduction of the base slab thickness.

Even though the results and benefits of this new roof truss were excellent, all the problems could not be escaped. A couple of the weld connections were on the edge of being 30°. This is not a deal breaking issue, but it has to be checked and tested with the steel manufacturer. The geometry could have been changed in a way that all the connection would have been clearly bigger that 30° but this would have affected the placing of the braces too much. Tentatively this issue of welding angles smaller than 30° was checked with one manufacturer and they replied that it is possible to execute these tight angles. They only have to carry out test welding for that.

RESOURCES

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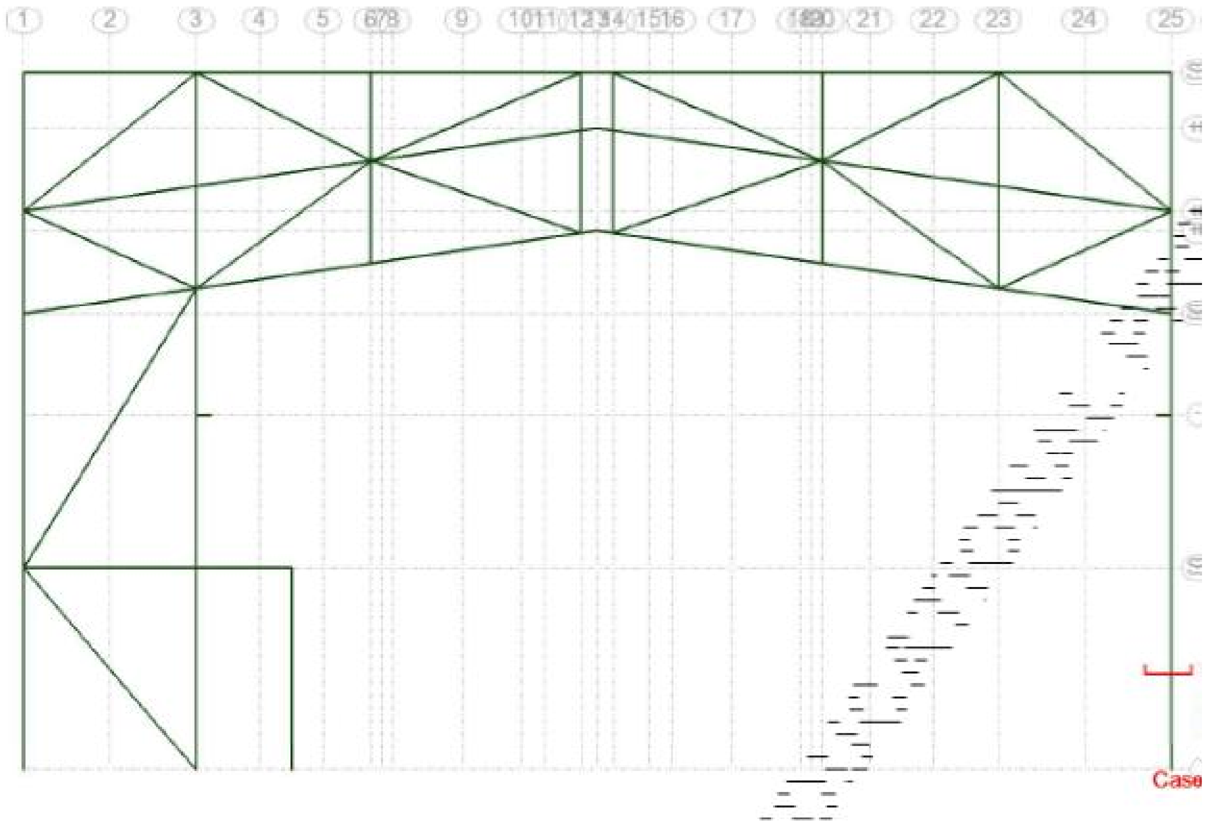
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