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E6 STAVSJØFJELLTUNNELLEN

Excavation performance estimation of the
new parallel tunnel

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Master in Science of Project Management
Submission date: September 2016
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DEPARTMENT OF CIVIL AND TRANSPORT ENGINEERING

Report Title: E6 STAVSJØFJELLTUNNELLEN – Excavation performance estimation of the new parallel tunnel	Date: 30.09.2016 Number of pages (incl. appendices): 139			
	Master Thesis	X	Project Work	
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Abstract:

The aim of this Master Thesis is to estimate the time of the excavation of the second tube of Stavsjøfjelltunnelen. The Semester Project was used as a background for the Master Thesis when it comes to vibration limits, method selection and vibration restriction estimation. The estimation of the vibration limits and restrictions was considered to be crucial when it comes to conventional Drill and Blast excavation of the parallel tunnel tube. In this Thesis the suggestion for drill and blast design is proposed. As a result, a prototype of Tunnel Excavation Performance (TEP) Model was created.

Important part of the analysis is based on the estimated excavation parameters and geological condition in order to achieve the best possible results. The geological parameters were chosen based on the available information. The calculation of the excavation time is based on the tunnel excavation performance program called Tunsim and tunnel support installation performance model made by NTNU.

The new tube of the Stavsjøfjelltunnelen has the T8.5 profile, a length of around 1720m, and was assumed to be excavated using traditional D&B methods.

Keywords:

1. tunnel excavation performance model
2. ground vibration
3. drill and blast
4. tunnel design

Preface

In order to get the Master Degree in Project Management in Civil Engineering at NTNU at the Department of Industrial Economics and Technology Management the student is obligated to deliver a Thesis that comprises 30 ECTS. The Master Thesis should be delivered before 1.10.2016. The Thesis was written under supervision of the Civil and Transport Engineering Department at NTNU.

The project concerns the new parallel tube for Stavsjøfjelltunnelen, since the increasing traffic between Trondheim and Værnes requires a more efficient road.

The Thesis requires a knowledge of civil engineering, especially tunnel construction. The personal experience in tunnelling construction is also valuable. However, the topic of parallel drill and blast tunnelling is extremely broad and requires enormous experience and knowledge. Thereby, the project was undertaken with cooperation from Reinertsen AS and SANDVIK. The subject of the project was agreed therefor, by Reinertsen AS and NTNU. The project includes estimation for vibration restriction, the D&B design, as well as estimates of the possible construction time.

Special thanks are dedicated to the supervisor Professor Amund Bruland, who was supporting me like no one before from the very beginning of the studies at NTNU; to Geological Engineer Bjørn Erling Eggen; Project Manager Frode Austgulen from Reinertsen AS; as well as Jouko Muona from Sandvik, whose eye for detail and personal experience were very helpful. Additionally, I would like to thank Gisli Gudjonsson, the Project Manager in Marti Norge who believed in me and employed me on the parallel drill and blast tunnelling project, thereby giving me a possibility to gain experience in the parallel tunnelling. At the end I would like to thank my family: parents, grandfather and sister, for their support and belief in me.

The Thesis will be forwarded to Reinertsen AS for further planning.

Norwegian Science and Technology University

Trondheim 30.09.2016

Marek Multan

Summary

This Master Thesis, written in 2016, is a result of the cooperation between NTNU and Reinertsen AF. The aim of the Thesis is to estimate the performance of the excavation of the second tube of Stavsjøfjelltunnelen. The tunnel is located between Trondheim and Værnes, and is an important connection between Trondheim and the airport. The planned construction is the result of increasing traffic and is the way to meet the demands of new safety regulations.

The existing tunnel is 1730m long, including portals, from which 1700m is the excavated tunnel. According to the available information the new Tunnel is around 1720m long. Due to the changing geological conditions, close residential areas as well as existing parallel tunnel located close to the planned tunnel, the vibration restrictions were expected to become the most important factor influencing the excavation performance.

The vibration limits have to be set, due to the existing requirements, in order to prevent unexpected fall downs in the existing tunnel, as well as to ensure safe blasting around the residential area sections. The estimation of the vibration level caused by the blast has been established based on four widely used methods, from which one has been selected.

The vibration restrictions influence the Drill and Blast design that was suggested in this Thesis.

The analysis of the excavation performance using a prototype of Tunnel Excavation Performance (TEP) Model shows that the tunnel could be finished between 368 and 393 days depending on the rock condition.

The time windows that the blasts are allowed to be undertaken, that are the result of the existing traffic in the parallel existing tunnel, have been found to have influence on total time. By removing the time constrains the tunnel excavation would be reduced to 323 days. By allowing constant work 24 hours 5 days a week the total duration of the project would be reduced to 312 days. Not taking the injection into consideration that would delay the project by 7 weeks in case of Stavsjøfjelltunnelen.

Therefore it is highly recommended to evaluate if the existing tunnel could be closed for the duration of the construction period and if the work could be performed 24 hour 5 days a week.

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1. *Attachment 1 - Tunnel Excavation Performance Model (TEP)*

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Abréviations:

Injection – High Pressure Grouting

PEG – tunnel chainage

TEP – Tunnel Excavation Performance Model being a result of the Thesis

Blasting Time Windows – the periods during the day when the blasting is allowed

D&B – Drill and Blast

SK – “sikringsklasse” – rock support class

NPRA – Norwegian Public Roads Administration

1. Introduction

1.1 Background

The topic of parallel tunnels has become of great interest to engineers and scientists in Norway. Increasing population and changing safety regulations require more efficient infrastructure system. Parallel tunnels have become a good solution for the new standards. Due to the Norwegian Transportation Plan, 15 tunnels are going to be built between 2014 and 2023 (Fjærtoft, 2013). Recently the section between Ranheim and Værnes was taken into consideration by Statens Vegvesen. The tunnel safety requirements desire existing tunnels, where the traffic exceeds 20 000 ADT, to have double tube (AsplanViak, 2015). Statens Vegvesen decided to accommodate these needs and planned to widen three tunnels between Ranheim and Værnes. The section was divided into 3 sections (Delstrekning) (Figure 1). Section 1 includes Væretunnelen, section 2 includes Stavsjøfjelltunnelen (which is the object of this project), and section 3 presents Helltunellen. In terms of selection of the tunnel excavation method the Norwegian tunnelling industry has preferred conventional drill and blast methods. In most of the tunnels in Norway, geological conditions are diverse. The D&B method provides safe tunnelling because of the ability to apply continuous pre grouting and probe drilling. This method gives great flexibility when it comes to water seepage, as well as appropriate rock support (Eivind, 2014). Despite many advantages, the D&B method induces ground vibration and represents a potential danger for the stability of the rock pillar and existing tunnel. In order to provide proper design of the tunnel, the amount of explosives has to be safely evaluated and is constrained by the induced ground vibration. The ground vibration limitation will therefore influence the time and cost of the tunnel excavation.

Figure 1 Aerial map of Ranheim - Værnes connection (ReinertsenAS, 2015a)

1.2 Aim

The Master Thesis is based on the Semester Project of the same author and aims to propose a design of the parallel tunnel tube of Stavsjøfjelltunnelen, as well as evaluate the blast design along the tunnel. Issues such as the influence of tunnel tube dimension, rock properties, vibrations induced while blasting on rock pillar dimension, and tunnel safety might become crucial. In this Thesis, the comparison presented in the Semester Project will be presented. Using results from the Semester Project, the vibration limit requirements will be calculated in the Thesis. There are many factors influencing the time consumption in the tunnel. The existing NTNU tunnel excavation performance model does not include the time consumption for the rock support installation, nor does it allow the user to assume changing geological conditions and changing round lengths. In order to fulfill the analysis requirements, a prototype for the tunnel excavation performance will be proposed.

1.3 Structure

To begin, the current situation will be presented. Further, geological conditions will be introduced. In order to provide the technical background for the project, basic information about vibration characteristics and the influence of geological condition on vibration distribution will be provided. Later, NS 8141.E and NS 8141-1:2102+A1:2013 methods for vertical particle velocity limits estimation will be presented and compared. In order to recommend the method used to estimate the amount of explosive per drill hole, four popular methods will be presented and compared. Finally, the results obtained by the selected method will be shown. Additionally, the living areas and surrounding buildings not included in the Project Work will be considered while estimating the vibration limits. Part of the Master Thesis will describe the standard Norwegian D&B design. The whole project will be analyzed while paying special attention to the construction time, for which a prototype for the Tunnel Excavation Performance Model will be proposed.

At the beginning of each chapter a short summary will be provided.

In the case of a lack of information, the closest possible estimation was used.

2. Work Process

The whole assignment focuses on using the selected method for vibration estimation for the design of the parallel tunnel tube of Stavsjøfjelltunnelen, evaluation of the blast design along the tunnel and time consumption. In this chapter the work process concerning the Master Thesis will be presented.

In order to get the overall knowledge regarding the design of double tube drill and blast tunnels, a literature review was performed. The literature used in the Thesis was found during the research for the obligatory Literature Review Report for the course TBA4128 - Project Management, Advance Course, as well as the literature review undertaken while writing the Project Work. The main outcome of the report indicates that there are few researchers exploring the topic of parallel drill and blast tunnels. When it comes to parallel tunnel design, empirical studies are of low reliability, whereas reliable studies are numerical and limited to a theoretical approach. There are general rules concerning rock pillar width and strength, however, none of the rules deal with rock pillar strength when it comes to drill and blast tunnelling. The topic is not explored and presents an opportunity for further research.

Since an assessment of the literature shows a low rank of the researches, it was decided to focus on standards and reports from NTNU, Reinertsen, Statens Vegvesen, and Sandvik.

When it comes to the calculation of the parameters dependent on the geological condition used for time consumption and cost estimation, the following NTNU reports shown in the Table 1 formed the bases the estimation.

Table 1 Main reports and models used in the Master Thesis

Vibration calculation:	14A-98 Fjellsprengningsteknikk, Sprengning med restriksjoner (1998)
Excavation performance:	2A-05 Drill and Blast Tunnelling Blast Design (2006)
Rock support time consumption:	2B-05 Drill and Blast Tunnelling Advance Rate (2007)
	2F-99 Tunneldrift, Enhedstidssystem for driving, sikring og innredning (1999)

However, the cooperation with Reinertsen, Sandvik and NTNU ensured knowledge transfer and helped to clarify uncertainties - there is much further work to be done. Since this assignment gives the general recommendations, further work suggestions were included at the end of the Thesis.

3. Current situation

This chapter presents the current situation and describes the existing tunnel tube. The information about the existing tunnel tube was taken from (StatensVegvesen, 1991a). Information about the current tunnel condition, existing rock support and traffic intensity is included. In addition, the chapter describes the planned construction work.

3.1 Existing Tunnel

The Stavsjøfjelltunnelen is located in Malvik commune in Sør-Trøndelag and forms part of European Road E6. The road is an important connection between northern and southern Norway. The tunnel was opened in 1990 and has a total length of 1730m and a T9 profile (62,75m^2)(StatensVegvesen, 2012). The tunnel is assumed to be of H7+ class which results in a 90km/h speed limitation (AsplanViak, 2015). The tunnel has 5 emergency lay-bys located on the left-hand side when looking towards Stjørdal. It passes under the residential area of Selbuvegen at the vertical distance of 15m (ReinertsenAS, 2015b). Increasing traffic intensity gives a good reason for second tunnel tube construction.

3.2 Traffic

The traffic intensity is high and tends to increase. Due to the prognosis of traffic increment for National Transport plan the traffic intensity will change from 17 570 ADT in 2014 to 19 030 in 2020 and 21 020 in 2030. The European Union safety regulations require existing tunnels that have an Average Annual Daily Traffic over 20 000 to be widened up to 4 lanes and separated. In order to address these requirements, Statens vegvesen has planned to excavate the second tube of Stavsjøfjelltunnel by the end of the 2019. (StatensVegvesen, 2015b)

Figure 2 and Figure 3 show the average amount of cars passing the Være and Hell tunnels per hour during 24 hours. No information about the average amount of cars passing the

Stavsjøfjelltunnelen per hour during 24 hours has been found. However, similarity of the two figures suggests that the traffic peaks would be located between the same hours as it happens in case of Være and Hell tunnels.

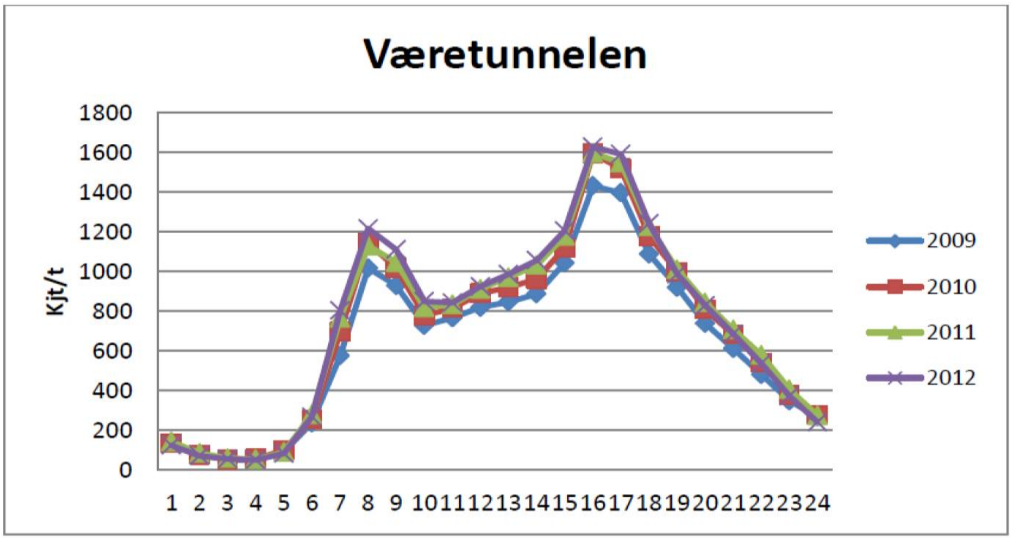

Figure 2 Average amount of cars passing the Væretunnelen (AsplanViak, 2012)

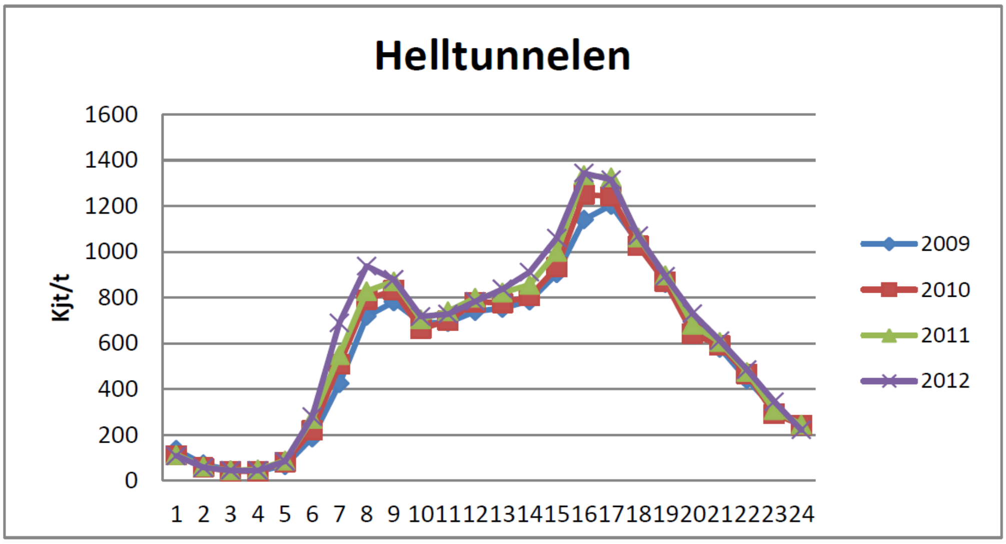

Figure 3 Average amount of cars passing the Helltunnelen (AsplanViak, 2012)

As the result of safety regulations and the traffic characteristics the blasting time windows were introduced. The suggestion of Erin Fjaertoft for the time windows shown in the Table 2 has been decided to be implemented in this Thesis by Reinertsen AS (Fjærtoft, 2013).

Table 2 Blasting time windows - the periods during the day when the blasting is allowed

Blasting time windows	
From:	**To:**
5:00	6:30
10:00	14:00
19:30	23:30

3.3 Existing tunnel support

Bolting:

In the tunnel, polyester anchored and grouted steal bolts of length 3m were installed. When it comes to difficult ground condition 4m bolts were used. Polyester anchored bolts were used if the need for immediate rock support appeared.

Shotcrete:

Shotcrete with steel fibres was applied along the tunnel in combination with bolting. The average thickness of the concrete is 6-8cm. In the areas of good ground conditions, the average thickness is 2-3cm.

Tunnel roof and walls are covered by shotcrete up to 2-4 meters of the walls depending on rock mass quality. In total, from 12 to 16 meters of tunnel circuit is covered by shotcrete. On average, 0.85 m^3/m of shotcrete was used along the tunnel.

Concrete casting:

In total 17.5m of the tunnel was supported by concrete casting. The support was applied between 14309,5 and 14327 profile. The average thickness of the concrete casting is 0,85m.

Injection:

Under the populated area around Selbuvegen between tunnel profiles 14836 and 14912, cement injection was applied. Between those profiles the tunnel crosses under deep valley and thick moraine cover.

Water and frost protection:

Water and frost protection covers 92% of the total area of the tunnel and is estimated to be 31 200 m^2. From both sites of the tunnel 200m of concrete lining was applied. Additionally, 23 576 m^2 of PE foam was applied because of water leakages. The process was repeated many times because of constantly appearing new leakages.

In general, the following temporary rock support was installed during construction:

- 3782 rock bolts
- steel reinforced concrete ribs 52m
- 1328 m^3 sprayed concrete
- 17,5 m concrete casting
- 150 m^3 extra concrete

The following permanent rock support was installed:

- 1504 rock bolts
- 130m^3 sprayed concrete
- 23 600 m^3 PE foam
- 7 600 m^2 concrete lining
- 10 m^3 extra concrete

3.4 Excavation duration – existing tunnel

The tunnel was excavated from both ends. The first blasting salve was done from the west on 7.12.1988. Tunnel excavation from the east had begun on 20.04.1989 and stopped on 12.10.1989. The excavation of the tunnel was finished on 25.10.1989. The efficiency and the total drilled length from two sites is presented below (StatensVegvesen, 1991b):

From west: 1248m/40 weeks = 31,2 m/week

From east: 452m/20 weeks = 22,6m/week

3.5 Plans

Based on the current situation, Statens Vegvesen decided to include 200 tunnels in the national tunnel improvement plan. The plan includes 5 tunnels along the E6 between Ranheim and Værnes: Grilstad-, Egge-, Hell-, Være- and Stavsjøfjelltunnelen. The plan assumes to finish the Stavsjøfjelltunnelen in 2019. Additionally, Statens Vegvesen decided to renovate the existing tunnel. (StatensVegvesen, 2015a).

3.6 New tunnel tube

The new parallel tube of Stavsjøfjelltunnelen is planned to have an approx. length of 1730m, the tunnel profile T10,5 and tunnel class E (SINTEF, 2015). The cross passages between the two parallel tubes will be established every 250m at profile numbers: 13690, 13940, 14190, 14440, 14690, 14940, 15140 and will have a tunnel profile of T4. The niches will have the tunnel profile T13.5 and will be established every 500m at profile numbers: 13930, 14430 and 14940 (ReinertsenAS, 2015a).

3.7 Residential areas around the new tunnel tube

The tunnel is located in immediate proximity to residential zones and buildings. The locations marked with A, B and C circle show the considered locations Figure 4.

Location A (PEG 15260):

Portal area from the Stjørdal side. The residential area is located 15m over the new tunnel tube (ReinertsenAS, 2015b).

Location B (PEG 14750 -14950):

The residential area is located 15m over the new tunnel tube (ReinertsenAS, 2015b).

Location C (PEG 13540):

Portal area from the Trondheim side. The residential area is located 150m from the tunnel portal (GoogleMaps, 2016).

The drill and blast design is further adjusted with respect to the vibration limits influenced by close location of the residential areas.

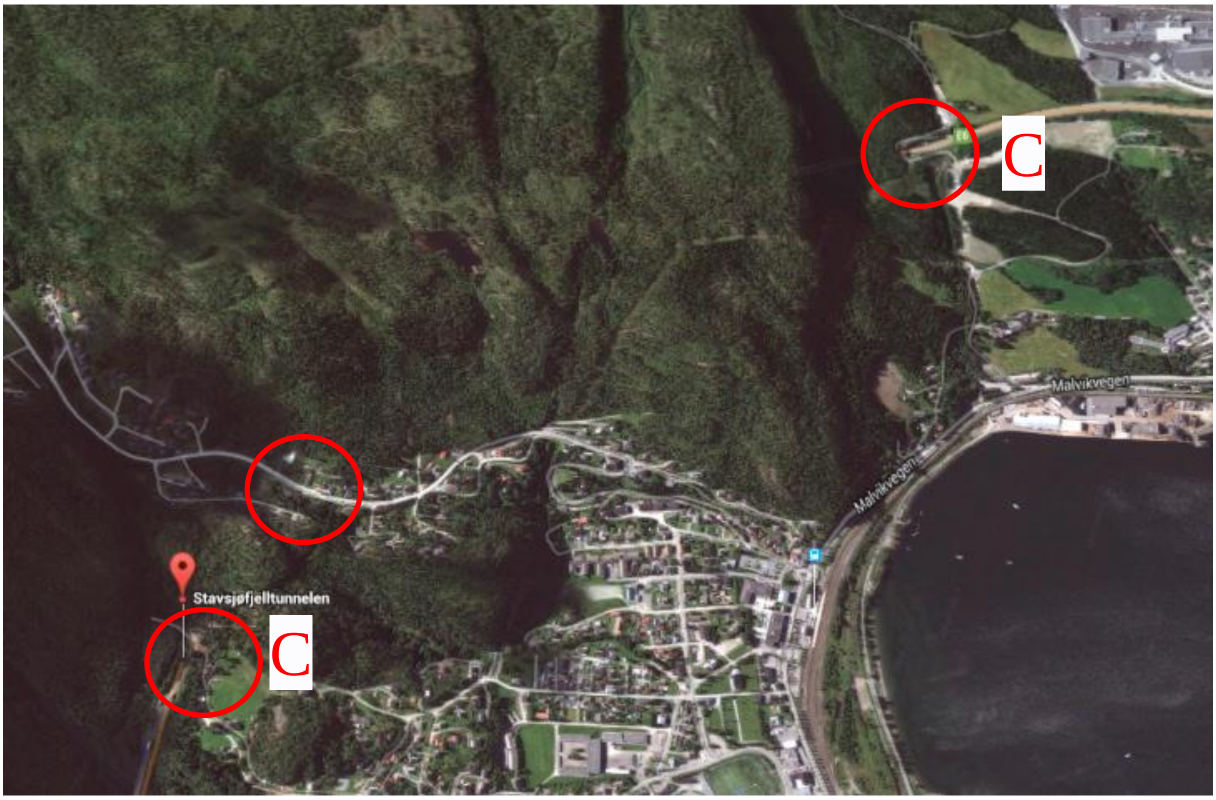

Figure 4 Arial photography of Stavsjøfjelltunnelen - living areas A,B,C (GoogleMaps, 2016)

4. Geology

The chapter describes the geological situation based on available geological reports. The current situation has not been described precisely in the available material. However, it does offer sufficient information in order to provide reliable estimation. Information presented in this chapter was taken from (ReinertsenAS, 2015a).

4.1 Rock mass sections

The rock mass originates from the cambro-silurian formation. The rock consists mostly of metamorphic sediment partly from volcanic origin. In general, metamorphism is relatively low but it varies along the tunnel. The rock mass along the tunnel is: greywacke, phyllite, and conglomerate.

The rock mass will be presented and described starting with the profile number 13518(ReinertsenAS, 2015b):

- Dark grey-black phyllite (13518-13750)
- Massive greywacke with schistose sections (13750-14325)
- Conglomerate partly conglomerate greywacke (14325 – 14420) and (14900 – 14980)
- Highly fractured phyllite with schistose greywacke (14420-14900)
- Greywacke and conglomerate with calcite (14980-15240)

Due to changing rock mass characteristics, drillability and blastability varies. The highest differences will occur in the area where greywacke and phyllite occurs in frequent exchange. The greywacke in the western part of the tunnels and the conglomerate are expected to be the most problematic in terms of the penetration rate and drill bit wear. The phyllite areas are expected to be the most favourable.

The available material presents a general description of the brittleness (42-50) however, does not indicate the exact profile numbers. For the purpose of the calculations, the tunnel was divided into 6 sections depending on rock mass. Rock quality varies along each section and as a result the tunnel was divided into 64 sections with different rock characteristics. Further, in the calculations, distance between two tunnel tubes will be of high importance.

In order to provide precise distance, sections lengths are not longer than 50 meters. In the report, Q-method was used to describe the rock quality, which varies between F and A/B. The percentage distribution of rock mass quality is presented in the Table 3. The total length of the excavation section of the new tunnel tube is assumed to be 1722m.

Table 3 Percentage distribution of rock mass quality in the new Stavsjøfjelltunnelen

Distribution	A/B	C	D	E	F
%	24,1	22,1	42,5	8,7	2,6
m	415	380	732	150	45

4.2 Rock blastability

The rock blastability is defined by rock blastability index SPR: "the amount of explosives (kg/m3) are needed to break the rock to a certain degree of fragmentation, where 50% of the blasted rock size is under 250 mm (d50 = 250 mm)"(Zare, 2007b). The Thesis does not consider the calculation of SPR, however, the most important factors influencing SPR will be presented. SPR values for rock type occurring along the tunnel were assumed based on the database from samples tested in the Engineering Geological Laboratory at NTNU. The values were assumed to correspond to phyllite and had a range from SPR = 0,47 to SPR = 0,57 dependent on rock quality. For the purpose of the Thesis and the Tunnel Excavation Performance Model (TEP) the SPR = 0.47 was assumed for the sections where rock mass quality is not worse than D. For the sections where the rock mass quality is below D, the SPR is assumed to equal 0,54. A value of SPR = 0.47 corresponds to medium blastability in NTNU's Tunsim model, whereas SPR = 0.54 corresponds to poor blastability.

The blastability index, SPR, is however, influenced by the following rock characteristics, which are difficult to measure at this stage of the planning (Zare, 2007b):

- Anisotropy
- Density
- Sonic velocity
- Mineralogy and grain binding
- Charging density of the explosives
- Detonation velocity of the explosive

4.3 Rock mass drillability - Drilling Rate Index (DRI)

The DRI is a combination of two indicators: SJ – Sievers miniature drill-test and S20 – intact rock specimen brittleness value. The SJ miniature drill test is "an indirect measure of rock resistance to tool indentation (surface hardness)" while the S20 " is an indirect measure of rock resistance to crack growth and crushing."(Sandvik, 1999). The testing procedures of the samples will not be presented in the Thesis however, the recorded DRI values for the samples tested in the NTNU Geological Laboratory are presented in Figure 5. For the purpose of the project a DRI value of 49 has been assumed.

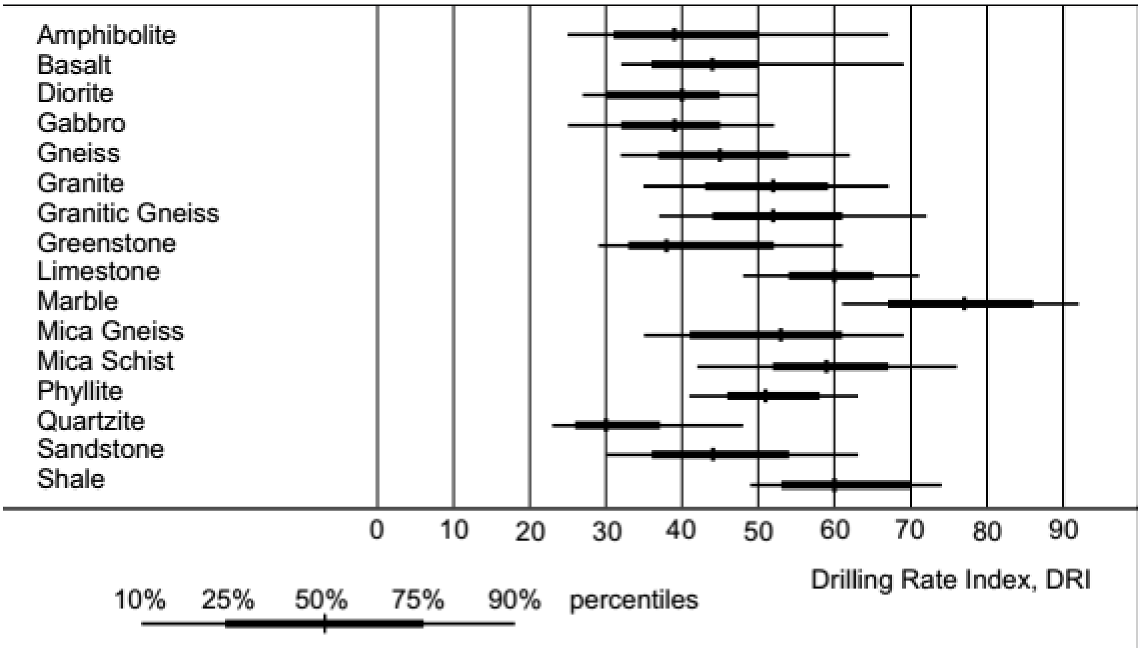

Figure 5 Recorded Drilling Rate Index, DRI for different rock types(Zare, 2007a)

4.4 Vickers Hardness Number Rock (VHNR)

Vickers Hardness Number Rock "A simplified approach to rating rock wear capacity is the use of rock surface hardness or mineral microindentation hardness."…"The hardness number is defined as the ratio of the applied indenter load (kilogram force) to the total (inclined) area of the permanent impression."(Sandvik, 1999) The load is applied to the sample by square based diamond pyramid. In order to define VHNR for this project, the closest possible assumption was taken from the Figure 6. The average for phyllite was assumed to be adequate for this project – VHRN = 550.

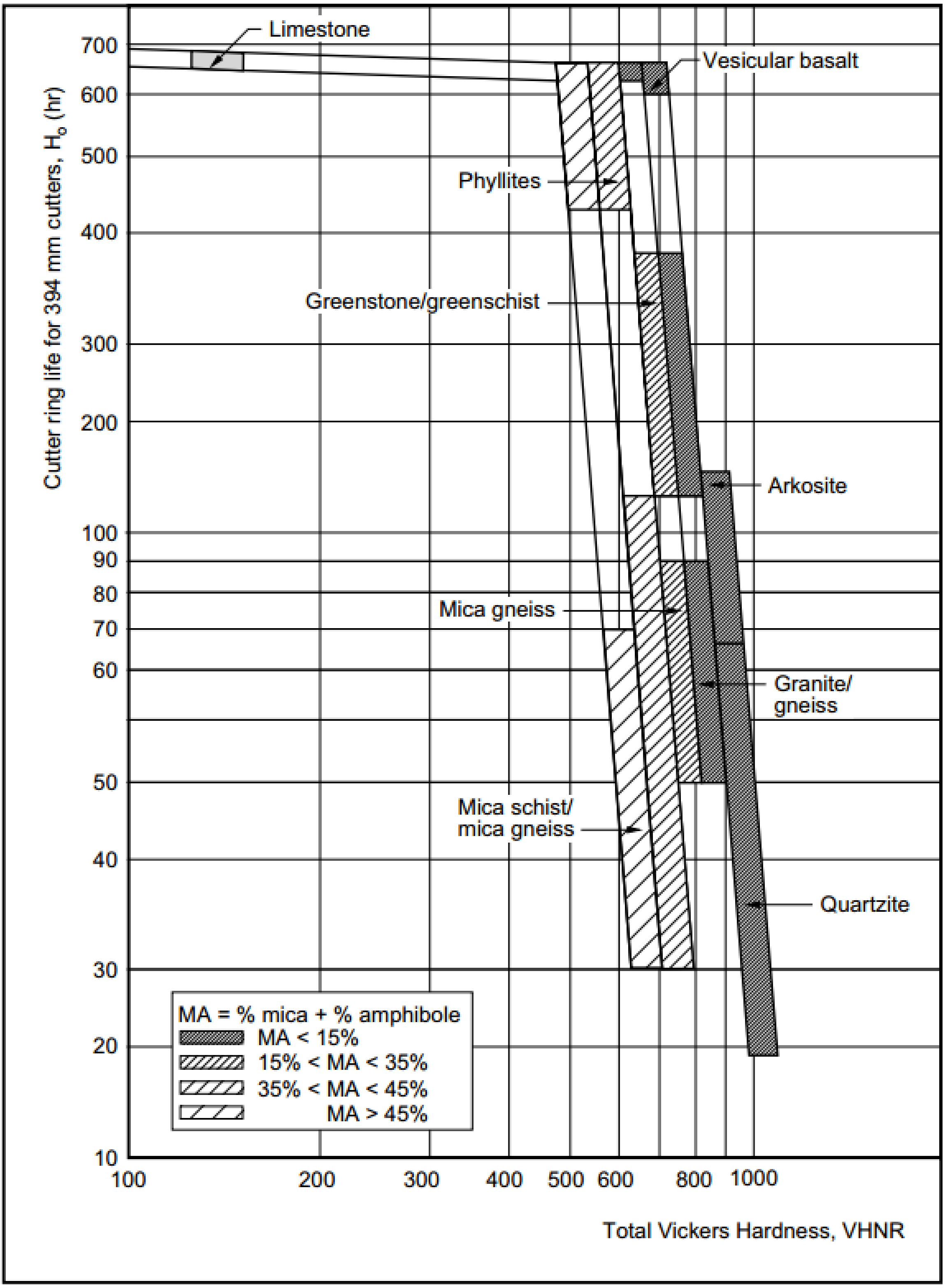

Figure 6 Recorded cutter ring life as a function of Vickers hardness number for rock VHNR (Bruland, 1998)

5. Estimation of the vibration limits

There is a need to place the tunnel as close as possible to the existing tunnel. Such placement reduce costs and the length of cross passages. However, close placement might cause high level of vibration in existing tunnel induced by blasting in the tunnel under construction. There are two vibration limits estimation models that can be used. This chapter presents general information about seismic waves and vibration and explain why and how vibration propagates. Information presented in this chapter together with figures were taken from (NTNU, 1998).

5.1 General information

The vibration level in the existing tunnel depends basically on the distance between the existing tunnel and the blasting source as well as the amount of charge detonated in the same moment. Important factor influencing the wave are characteristics of transmission medium. Potential factors influencing the wave characteristics could be: momentary detonating charge, distance between the source and the sensor, rock characteristics, fracturing and drilling accuracy.

5.2 Frequency

The higher the frequency the lower the danger for damage risk. Frequencies from 5-15 Hz are the natural frequencies for most buildings. Vibrations induced by blasting are in the range of 10 – 100 Hz and the dominating frequencies are around 40 – 70 Hz. Frequency of the vibration wave decrease with the distance increment Figure 7. This indicates that for short distance between the vibration source and the second tunnel tube one can allow higher particle velocity with respect to the recommended values for particle velocity dependent on the ground condition.

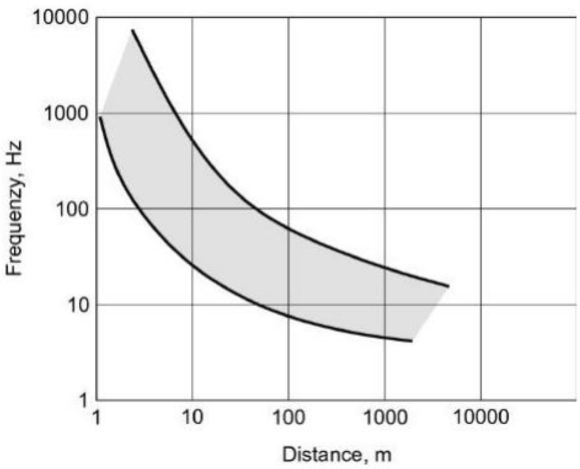

Figure 7 Oriented Values for frequency capacity

5.3 Seismic wave

There are three types of seismic waves created by blast in the rock environment:

- P-wave – pressure wave
- S-wave – shear wave
- Surface wave

P-waves, also called compression waves produce oscillating compressive stress and shear stress. The p-wave propagates in the direction of the wave.

S-waves, also called shear waves produce oscillating shear wave perpendicular to the wave direction.

5.4 Geological influence

Factors like rock type, passing from one media to another, joints or degree of fracturing can influence vibrations transition abilities through the rock mass. Type of the rock can influence the seismic velocity. The velocity might vary from high values in case of rigid competent rock types with high compressive strength to low values in case of soft fractured rocks. High degree of refraction can be experienced while passing through one media to another especially when the density difference is high.

5.5 Fracturing

The higher the degree of fracturing the lower the vibration level achieved. The angle between the direction of the wave and the direction of fracturing plane will influence the wave propagation. Strengthening and focusing of the wave will appear in case of law angle between fracturing plane and the wave direction. When the angle is bigger vibrations will be damped faster.

5.6 Border between two media

In most cases when the wave comes across the border between two media that have different properties the energy of the wave will be spread. One part of the energy will be reflected while the remaining part will pass through the border. There are two mechanisms causing damping:

- Geometrical spreading of the energy
- Absorption (damping due to hysteresis)

When the wave crosses the border between the rock and the air, high degree of refraction will appear and some part of the wave energy might be transmitted along the surface of the rock. In favorable environment presented on Figure 8 focusing of the vibration due to topography and geology might appear.

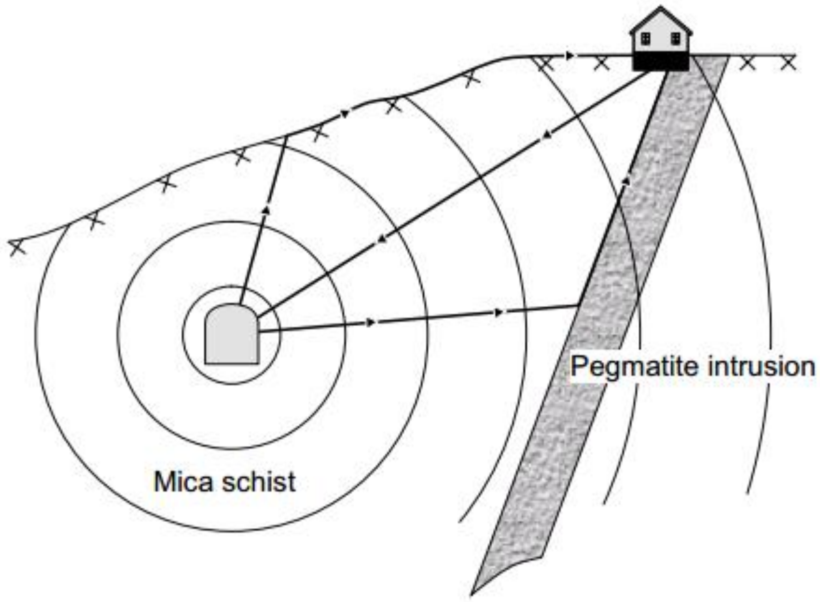

Figure 8 Focusing of vibrations

6. Vibration estimation methods

This chapter will present four well known and broadly used models for blast estimation. Following methods allow the estimation of different parameters considering the rock blasting technique. Different factors can be estimated by the transforming given equations.

In general there is one uniform formula that describes the correlation between vertical particle velocity, charge per delay, distance between blast and vibration sensor. This dependency can be described by following equation:

$$v = K * \frac{Q^{\gamma}}{d^{\alpha}} \tag{1}$$

v - vertical particle velocity (mm/s)

K - site parameter

Q – charge per delay (kg)

d – shortest distance between blast and measuring point (m)

γ - parameter associated with charge utilization

α - parameter associated with damping ratio

This equation is a common denominator for several methods. It was decided to take four methods into consideration.

- The ‘‘Norwegian’’ Model
- NTNU model
- NS 8131-1:2012+A1:2013 model (“New Standard” model)
- ISEE 1998 model

6.1 The "Norwegian" model

The ‘‘Norwegian’’ model is the most often used method in Norway. It is simplified formula given by formula (2). In this case γ =0,5 and α =1. The formula is given by:

$$v = K * \frac{\sqrt{Q}}{d} \Leftrightarrow Q = (\frac{v * d}{K})^2 \tag{2}$$

The model base on site parameter K which is a function of distance and rock mass condition. The correlation is presented below in Figure 9.

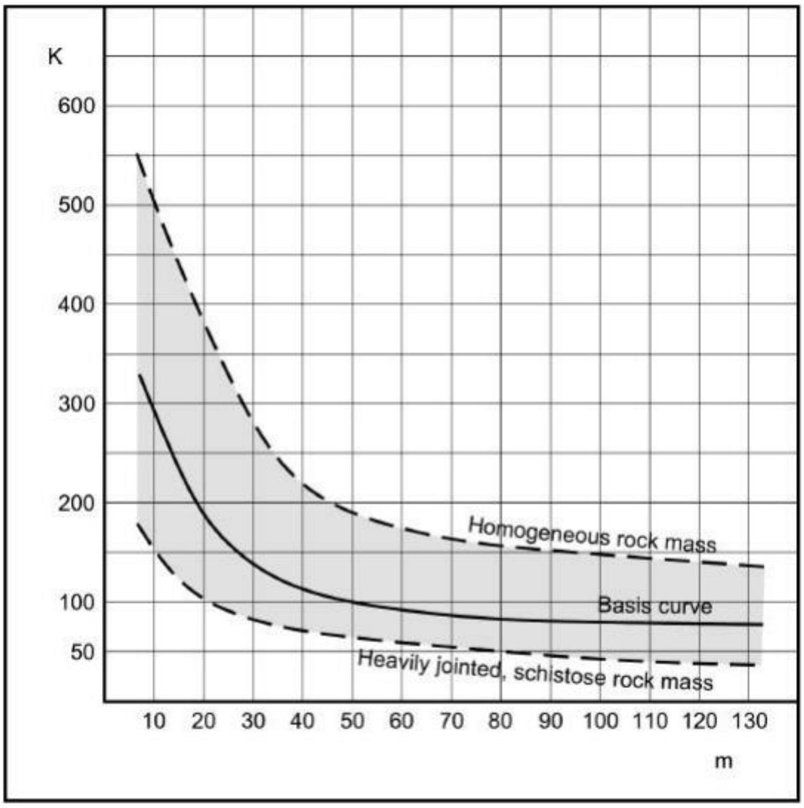

Figure 9 Site parameter K as a function of distance and rock mass conditions (NTNU, 1998)

6.2 NTNU model

The NTNU model was established in order to reduce the uncertainty associated with the ‘‘Norwegian’’ Model. The new equation was created based on NTNU experience and blast testing results and is given by following formula (3):

$$v = 500 * c * \frac{Q^{\alpha i}}{d^{\beta i}} \tag{3}$$

c - rock mass parameter

βi - damping exponent (exponent for geometrical damping, absorption and fracturing)

αi - charge exponent (exponent for explosive utilization)

In order to calculate the amount of explosive, the equation needs to be transformed as below:

$$Q = \frac{(v * d^{\beta i})}{(500 * \alpha i)}^{1/\alpha i} \tag{4}$$

Those factors and exponents can be found in tables and diagrams. The rock mass parameter can be read from Figure 10. The value varies between 0,7 and 1,3 and depends upon the degree of jointing and schistosity. For unknown rock conditions, c is assumed to equal 1. The charge exponent depends upon charging method and drillhole diameter, and is presented in Figure 11. The damping exponent β is solved from the equation (5).

$$\beta = \frac{\ln(500 * c * Q^{\alpha}) - \ln v}{\ln d} \tag{5}$$

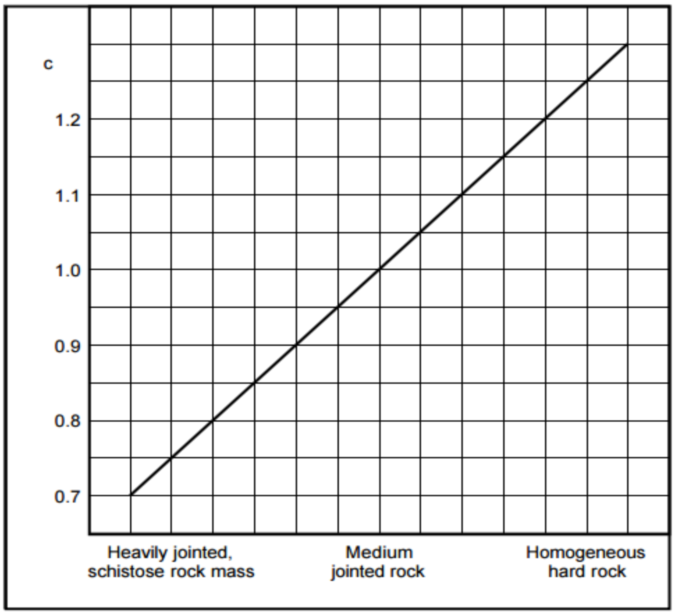

Figure 10 Rock mass parameter c (NTNU, 1998)

Blast type	α
Ordinary blasting works (ANFO/cartridged) drill hole diameter < 127 mm	0,8
Heavy bench blasting (slurry explosives) drill hole diameter 127 - 381 mm	0,5

Figure 11 Exponent for explosive utilization, α (NTNU, 1998)

Figure 11 presents the parameter for explosive utilization α. The value varies dependent on the type of the blasting works. In the case of the tunnel, one should choose a value of 0,8 that corresponds to ordinary blasting works.

Figure 12 presents correlated NTNU results. The β coefficient can be read from the figure. The β exponent value varies between 1,1 and 1,75 and depends upon the blasting degree (heavy, ordinary), as well as the direction of wave propagation (I jointing or II jointing).

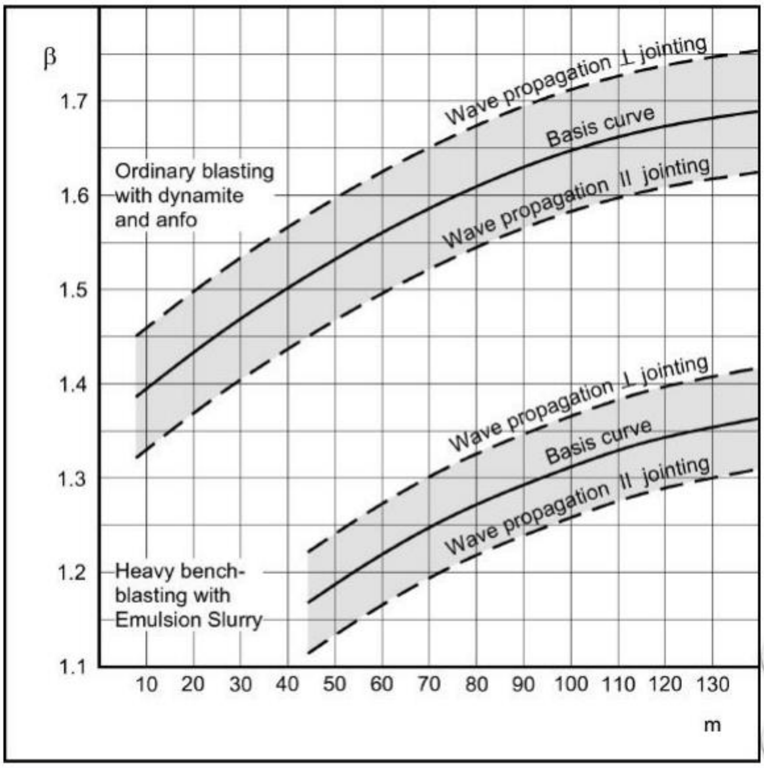

Figure 12 Damping exponent β for distance 5 m to 140 m (NTNU, 1998)

There are two ways of estimating the β coefficient in the NTNU method. The first method uses the information included in Figure 12, the second method uses in-situ ground condition tests and formula (3). In this, project data from site measurements were used. The data comes from the Strindheim tunnel. The data was the main input for the Eirin Fjærtoft Master Thesis in 2013.

6.3 NS 8141-1:2012+A1:2013 model

This model is the newest model and is the result of improving the equations with three separate coefficients. Studies show that models with three separate coefficients will always provide better regression results. The highest correlation was obtained by using formula (6). (Gjengedal, 2013)

$$v = K * \frac{Q^{\gamma}}{d^{\alpha}} \tag{6}$$

Additionally, by using the regression method, Gjengedal ended up with an improved equation and presented guidelines that are currently used in the guidance for NS 8141-1:2012+A1:2013.

$$v = H * \left(\frac{d}{\sqrt{Q}}\right)^{-\alpha} \tag{7}$$

H - rock coefficient dependent on type of excavation and distance (200 – constant for tunnels)

α - coefficient dependent on type of excavation and distance (1,24 – constant for tunnels)

6.4 ISEE 1998 model

This model was established by International Society of Explosive Engineers and uses the equation (8). This model is based on experience data and uses the scaled distance factor β=0,5 (Gjengedal, 2013).

$$v = H * \left(\frac{d}{\sqrt{Q}}\right)^{-1,6} \tag{8}$$

H – Ground response factor

Damping exponent is set to be 1,6 and is described as a typical value, but can vary between 1 and 2. The ground response factor equals 1725 and is said to be the upper-bound line for typical data from downhole blasting.

7. Calculations of the vibration values and results

The aim of this chapter is to present vibration calculations on behalf of Reinertsen AS. In the following, the limit values for vibration velocity will be calculated in order to prevent damages in the existing tunnel while excavating the second tunnel tube. This process was supported by the Department of Civil and Transport Engineering at NTNU using Project report 14A-98 Rock Blasting Technique. Guidance for the calculation process was found in Norwegian Standards. In order to show a broad perspective, limit values obtained through New Standard NS 8141-1:2012+A1:2013 introduced in 2013 will be compared to the old version of the standard NS 8141.E (English version of NS 8141:2001).

7.1 Limit values for vibration velocity due to NS 8141.E

The limit peak value of vibrations from groundwork (v) are determined by the formula.

$$v = v_0 * F_g * F_b * F_d * F_k \qquad (9)$$

v_0 is the uncorrected peak value of the vertical vibration velocity in millimeters per second and is set to 20 mm/s

F_g is a ground condition factor which takes into account the ground conditions at the site of the construction work

F_b is a construction factor which depends on the type and design of the construction works, the construction materials and the type of foundation

F_d is a distance factor which takes into account the distance between the vibration source and the measurement position at the construction works

F_k is a source factor which takes into account characteristics of the vibration source

7.1.1 Ground condition factor, F_g

The ground condition factor, F_g , depends upon rock type and quality, as well as seismic velocity. Reinertsen AS performed tests across the tunnel at the 14800 and 14900 profiles(ReinertsenAS, 2015c). The results shows that seismic velocity reaches 3995m/s. However, since NS 8141.E suggests either F_g = 2,5 or F_g = 3,5 , it was decided to introduce a value of F_g =3,0 or F_g =3,3 in the sections where rock quality was estimated to be of A/B quality, or where the seismic velocity approaches the values around 4000 m/s respectively. The total share of the values is presented in Table 4.

Table 4 Percentage share of the ground condition factor F_g

F_g	Share %
2,5	73,9
3,0	11
3,3	15,1

7.1.2 Construction factor, F_b

$$F_b = k_b * k_m * k_f = 1,7 * 1,0 * 1,0 = 1,7$$

$$F_b = k_b * k_m * k_f = 1,7 * 1,2 * 1,0 = 2,04$$

k_b is a construction factor that depends on the type and design of the construction works. Norwegian standard does not specify the factor for tunnel construction however tunnel construction was classed as heavy construction, e.g. bridges, docks, fortifications and equals 1,70

k_m is material factor which depends on the type of material in the construction works. Old Stavsjøfjelltunnelen is supported by bolts, and reinforced concrete and gives two factors respectively:

k_m=1,00

k_m=1,20

k_b is foundation factor which depends on the method of foundation for the construction works. None of the options presented by Norwegian standard relate to the tunnels however it states that in the case of structures build on rock, the foundation factor should equal k_f=1,00.

7.1.3 Distance factor, F_d

This factor describes the shortest distance between the source and the receiving point. The distance factor depends upon the vibration source, ground condition and distance. In the case of blasting in rock, the distance factor remains constant and is independent of distance.

$$F_d = 1,0$$

7.1.4 Source factor, F_k

The source factor depends on the type of vibration source. In the case of blasting, the factor equals

$$F_k = 1,0$$

As a result of the calculations, 5 different limit values for vibration velocity due to NS 8141.E were specified.

$$v = v_0 * F_g * F_b * F_d * F_k = 20 \text{ mm/s} * 2{,}5*1{,}7*1{,}0*1{,}0 = \mathbf{85mm/s}$$

$$v = v_0 * F_g * F_b * F_d * F_k = 20 \text{ mm/s} * 2{,}5*2{,}04*1{,}2*1{,}0 = \mathbf{102mm/s}$$

$$v = v_0 * F_g * F_b * F_d * F_k = 20 \text{ mm/s} * 3{,}0*1{,}7*1{,}0*1{,}0 = \mathbf{102mm/s}$$

$$v = v_0 * F_g * F_b * F_d * F_k = 20 \text{ mm/s} * 3{,}25*1{,}7*1{,}0*1{,}0 = \mathbf{111mm/s}$$

$$v = v_0 * F_g * F_b * F_d * F_k = 20 \text{ mm/s} * 3{,}25*2{,}04*1{,}2*1{,}0 = \mathbf{132{,}6mm/s}$$

The vibration velocity limits obtained through NS 8141.E is presented in Figure 13, the total percentage share of the vibration velocity limits are presented in Table 5.

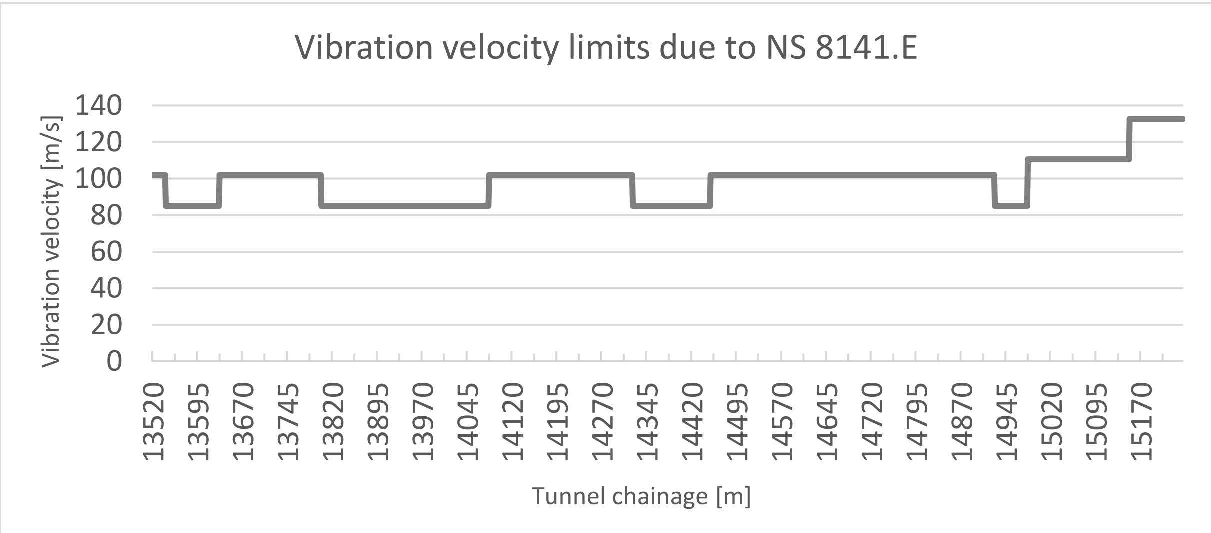

Figure 13 NS 8141.E vibration velocity limits due to NS 8141.E

Table 5 Percentage shear of vibration velocity limits - NS 8141.E

V [mm/s]	Share [%]
85	32,23
102	52,67
110,5	9,87
132,6	5,23

7.2 NS 8141-1:2012+A1:2013 – vibration velocity limits

The limit peak value of vibrations from groundwork (v) are determined by the Table 6. And is dependent on the following factors:

- Rock quality
- Existing rock support
- Quality of rock support
- Tunnel function

As a result, three different values for vibration velocity limits can be obtained: 15, 25 and 45 mm/s. Obtained values can be increased by a factor of 1,25 when the existing tunnel is not in use, and decreased by a factor of 0,5 when, in the tunnel, there is no possibility for inspection due to the presence of concrete lining. In this project, such an option was not considered. For this project it was assumed that none of the [a,b,c] situations will appear, however, it might be considered as a limitation (Table 6).

Table 6 Peak particle velocity estimation (translated to English from NS 8141-1:2012+A1:2013)

Description of the tunnel/cavern condition	**$v_{f,tunnel}$ [a,b,c] [mm/s]**
P1 - Poor rock conditions, spot bolting or no reinforcement or not reinforced shotcrete applied	15
P2 - Poor rock conditions, reinforced shotcrete together with bolts applied	25
P3 - Poor rock conditions, full profile concrete casting applied	45
G1- Good rock conditions, spot bolting or no reinforcement or non-reinforced shotcrete applied	25
G2- Good rock conditions, reinforced shotcrete together with bolts applied	45
[a] If the tunnel/cavern is not in use the v_f value can be multiplied by the factor 1,25 **[b] If there is no possibility for inspection in case of concrete lining the v_f value should be reduced by 0,5** **[c] When it comes to technical installations the v_f value should be estimated by following chapter 8.5 in the NS 8141-1:2012+A1:2013**	

The percentage results of the share of the values for vibration velocity limits due to NS 8141-1:2012+A1:2013 along Stavsjøfjelltunnelen are presented in Table 7.

Table 7 Vibration limits - NS 8141-1:2012+A1:2013

V [m\s]	**Share [%]**
15	32,23
25	52,67
45	9,87

The results shown in Table 7 were obtained by analyzing the guidelines described in Table 6. Over 53,83% of the tunnel chainage was qualified as poor rock condition (from F to D). 46,17% of the chainage has a quality of C to A/B. Along the tunnel, all combinations of rock support types and rock condition types, presented in the Table 6 appeared. The percentage shear of each alternative is presented in the Table 8.

Table 8 Percentage shear of vibration limit alternatives

	P1	P2	P3	G1	G2
%	9,87	42,80	1,16	43,26	2,90

The vibration velocity limits obtained through NS 8141-1:2012+A1:2013 is presented in Figure 14.

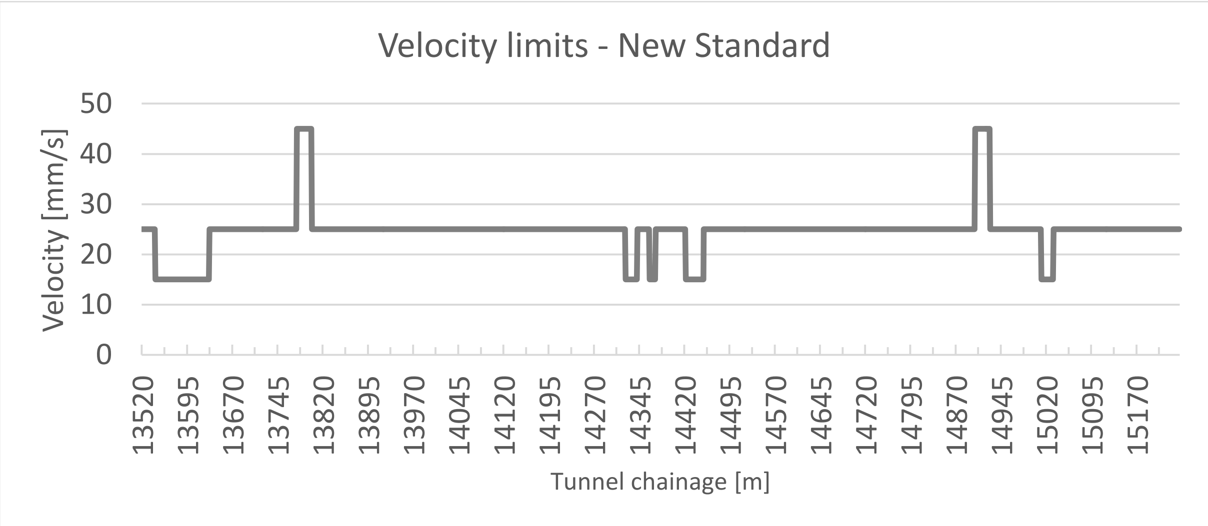

Figure 14 Vibration velocity limits - New Standard

7.3 Comparison

The new version of the standard makes a clear distinction between vibration in construction works other than tunnel work, and vibration during tunnel construction. The biggest difference has been observed when it comes to the final results. The New Standard is much more conservative, sometimes giving over 6 times lower results than those obtained by following the Old Standard.

By using the Old Standard, one can obtain many different limit values. The method depends on many different factors. The maximum result is 64% higher than the minimum result.

By using the New Standard one can obtain three different limit values. The method depends on few factors. The maximum result is 300% higher than the minimal result.

The comparison shows that the results from the Old Standard deviates much less than the results obtained through the New Method.

The two methods do not present significant correlation, as show in Figure 15. The correlation equals R^2=0,035. This result might indicate that the two standards take different factors into consideration when it comes to vibration limit estimation.

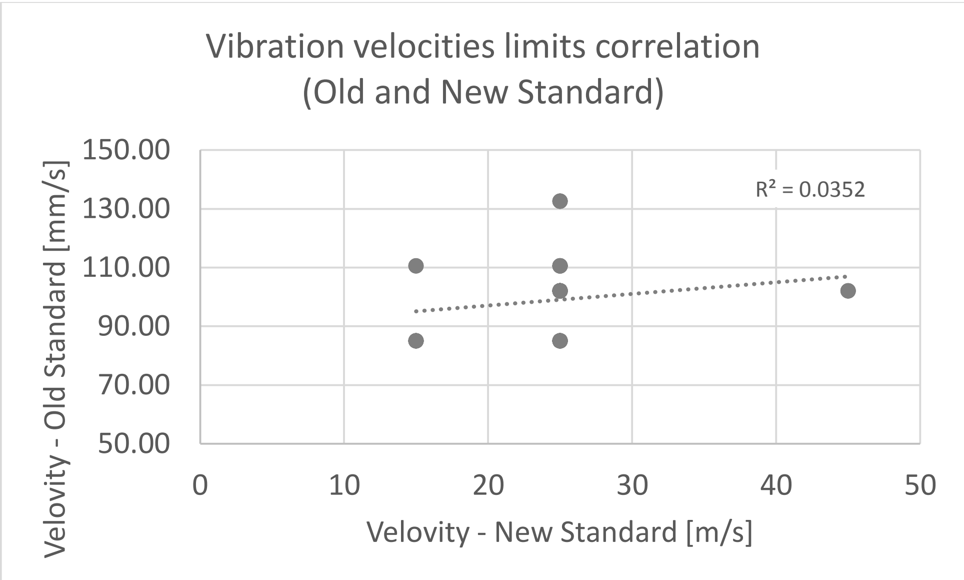

Figure 15 Correlation between vibration velocities limits (Old and New Standard)

8. Charge estimate methods for different vibration limits

In this chapter, four methods will be used in order to estimate the maximum charge amount for different vibration limits. Each method will use the limits obtained by following both the new and old Norwegian standards. The results will be presented on diagrams which show the maximum amount of detonated simultaneously charge. For certain charge estimates methods, maximum borehole length diagrams were presented. Drilling lengths below 2.3 meters and over 5.3 meters were not shown in the figures. Borehole lengths over 5.3 meters are not considered in this project. Areas where the drilling length is below 2 meters have to be further evaluated in order to suggest safe and appropriate charging, and will be further discussed in Chapter 11.5.2. Firstly, the results following NS 8141.E will be presented. Secondly, the results following NS 2012 will be presented. Next, all results will be correlated and compared. Conclusions will be presented at the end of each part of methods analysis. At the end of the chapter, all methods will be compared, and crucial insights and comments will be shown. All the calculation were done according to the equations presented in the Chapter 6 and 7. The calculations were done in the Excel program where the input data could be found (Attachment 1)

8.1 The New Standard Method

a) NS 8141.E limits

Figure 16 presents the charge amount estimated using the New Standard Method for vibration limits obtained through NS 8141.E. The average charge for this model equals 84,5kg. The dashed line presents the maximum unit charge used to charge one 5.3 meters long drill hole in the 5 meters round (8,3 kg).

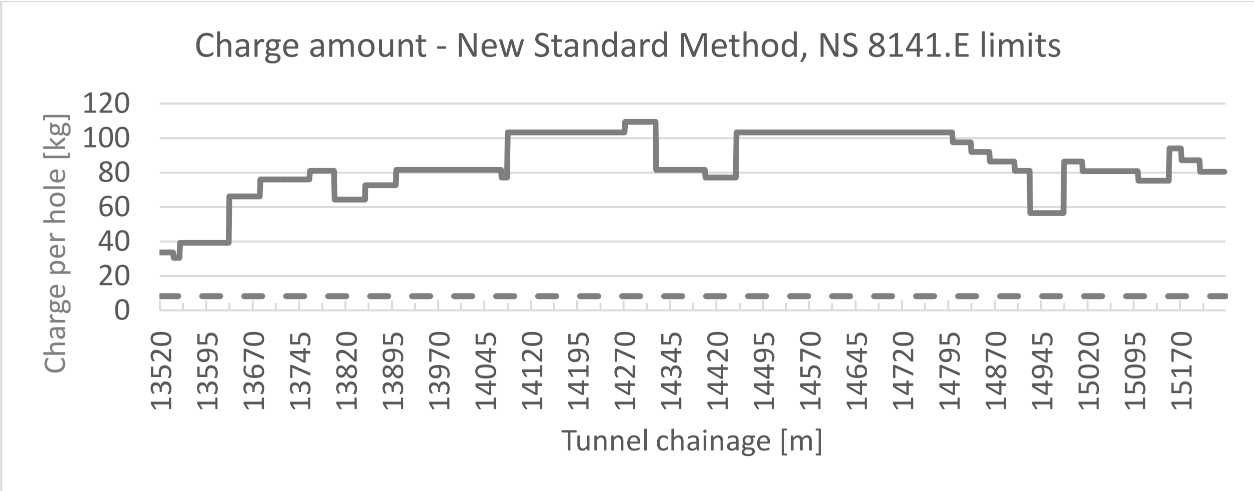

Figure 16 Charge amount - New Standard Method, NS 8141.E limits

b) NS 8141-1:2012+A1:2013 limits

Figure 17 presents the charge amount estimated using the New Standard Method for vibration limits obtained through NS 8141-1:2102+A1:2013. The average charge for this model equals 9,56kg. The dashed line presents the maximum unit charge used to charge one 5.3 meters long drill hole in the 5 meters round (8,3 kg).

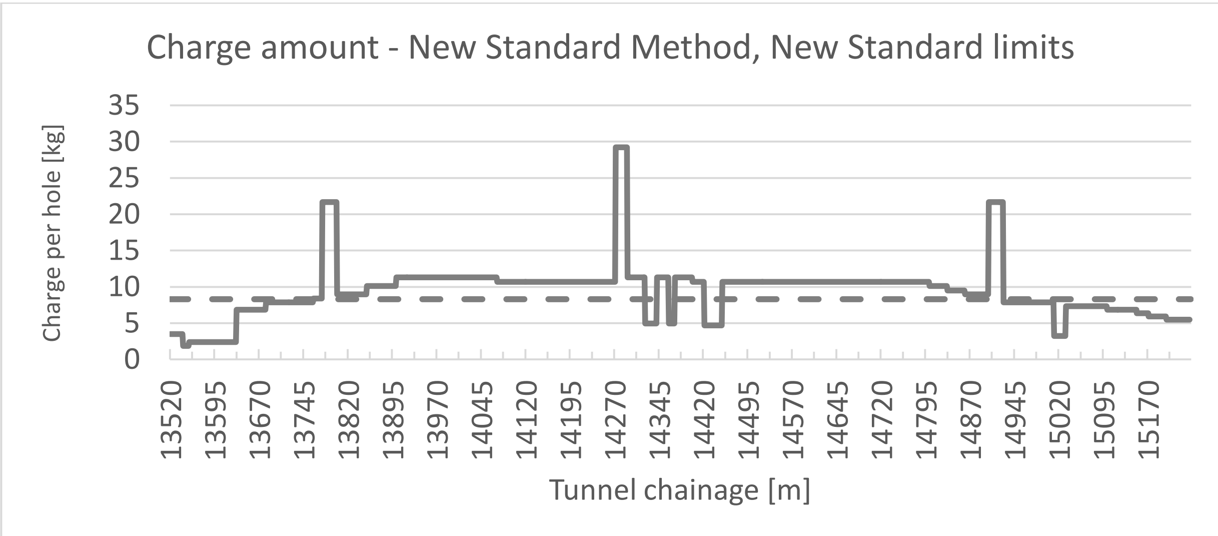

Figure 17 Charge amount - New Standard method, New Standard limits

Figure 18 presents the borehole length estimated using the New Standard Method for vibration limits obtained through NS 8141-1:2102+A1:2013.

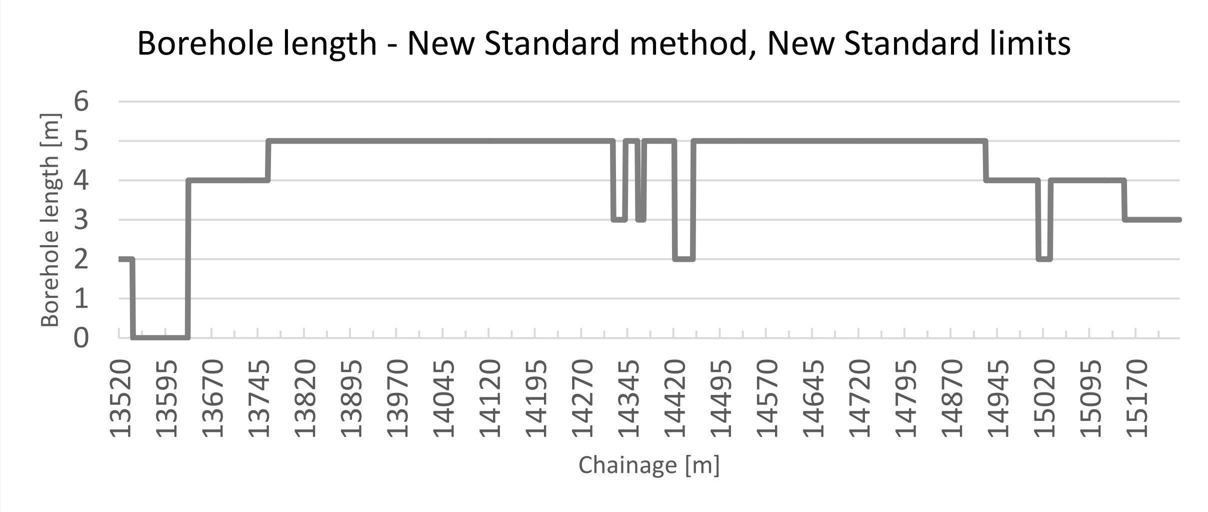

Figure 18 Estimated borehole length - New Standard Method, New Standard limits

Comparison:

Results obtained by following the old limits give the result over 9 times higher than the new limits. The two data sets have a cross correlation of R^2=0,26. The correlation is affected by sections from 13775 to 13800 and 14900 to 14925, where, due to the NS 8141-1:2012+A1:2013 limits, the total loading per hole equals 21,67 kg and 81,10 kg due to NS 8141.E limits. Additionally, the correlation is affected by section from 14270 to 14290 where, due to the NS 8141-1:2012+A1:2013 limits, the total loading per hole equals 29,22kg and 109,37kg due to NS 8141.E limits. After the exclusion of the values above, the cross correlation equals 0,46 (Figure 19)

Conclusions:

Based on the industry experience, a combination of the New Standard Method and Old Limits gives high results that are considered to be out of the scope of most predictable solutions. When it comes to the combination of New Standard Method and NS 8141-1:2012+A1:2013 Limits, results are considered to be incorporated in the scope of most predictable solutions.

The correlation between the two methods was affected by three tunnel sections. This situation is a result of high percentage differences between vibration limit values obtained through NS 8141-1:2012+A1:2013.

The correlation of charge amount results obtained from the two standards can show that the vibration limits influence the final trend of the results only to certain degree, since both methods have similar trends.

However, the vibration limits influence the amount of charge. As a result, the average charge amount considering NS 8141.E is around 8,8 times higher than the result obtained following NS 8141-1:2102+A1:2013.

The main conclusion arising from the analysis is that one should not combine New Standard Method with Norwegian Standard NS 8141.E.

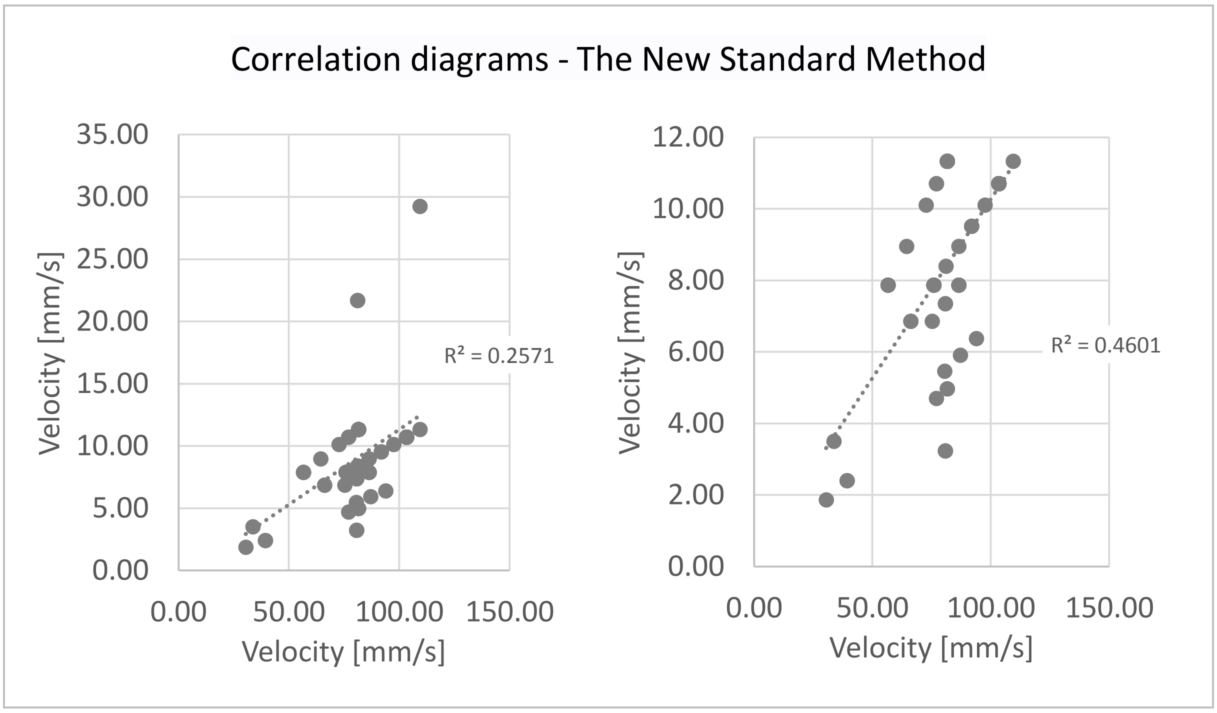

Figure 19 Correlation diagrams - The New Standard Method

a) before improvement b) after improvement

8.1.1 The ‘‘Norwegian’’ Model

a) NS 8141.E limits

Figure 20 presents the charge amount estimated using The ‘‘Norwegian’’ Model for vibration limits obtained through NS 8141.E. The average charge for this model equals 7,8kg. The dashed line presents the maximum unit charge used to charge one 5.3 meters long drill hole in the 5 meters round (8,3 kg).

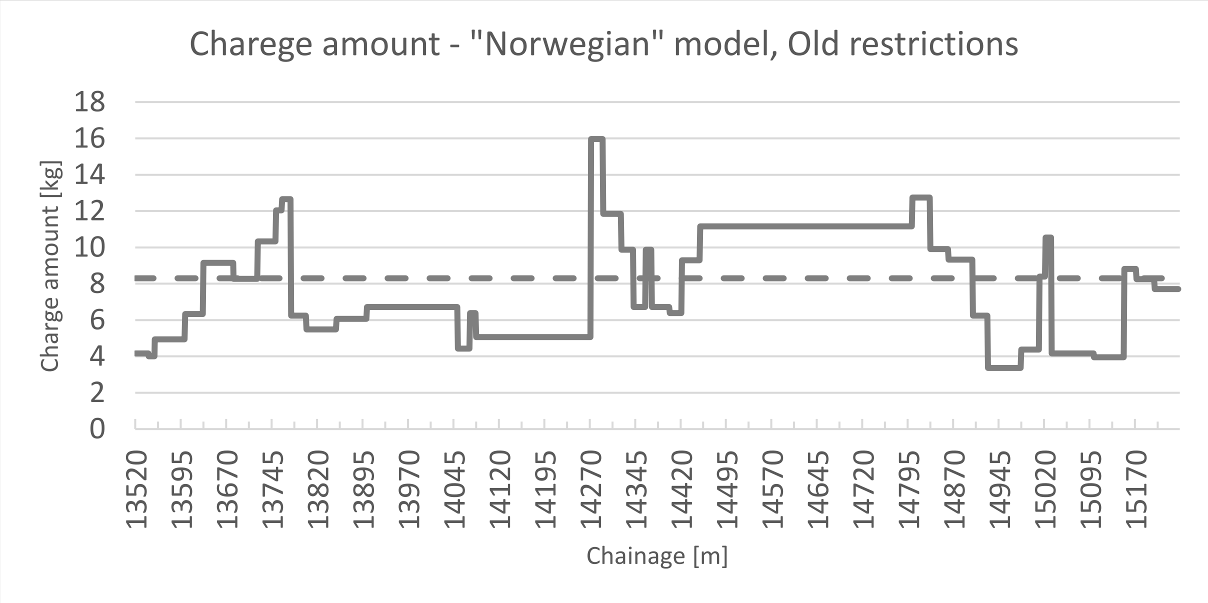

Figure 20 Estimated charge amount - ‘‘Norwegian’’ Model, NS 8141.E limits

Figure 21 presents the borehole length estimated using The ‘‘Norwegian’’ Model for vibration limits obtained through NS 8141.E.

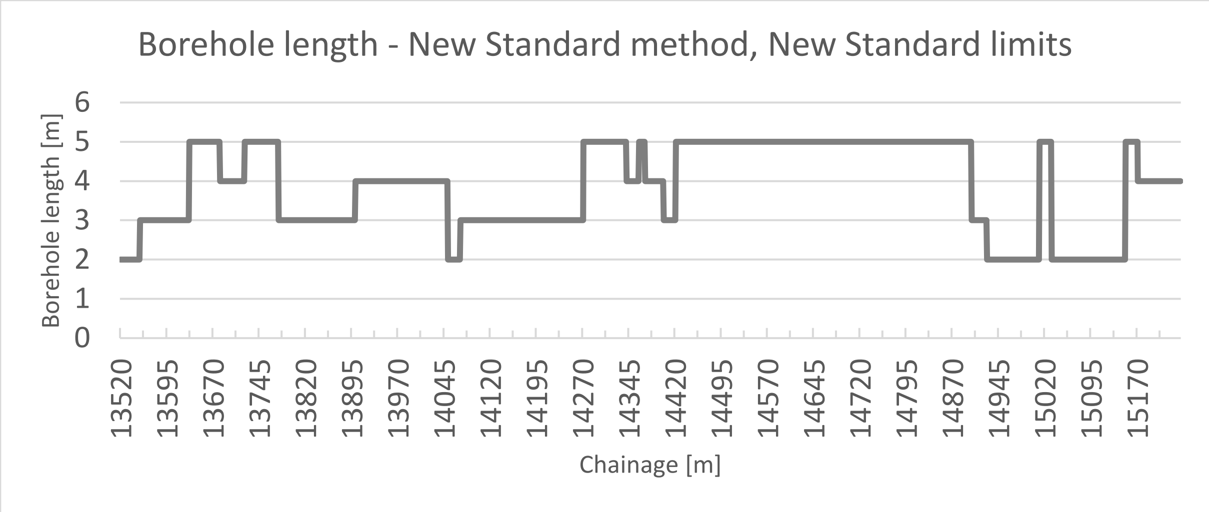

Figure 21 Estimated borehole lengths - ''Norwegian'' Model, NS 8141.E limits

b) NS 8141:NS 8141-1:2012+A1:2013 limits

Figure 22 presents the charge amount estimated using New Standard Method for vibration limits obtained through NS 8141-1:2102+A1:2013. The average charge for this model equals 1,98kg. 2,1kg is equivalent to 2m borehole length.

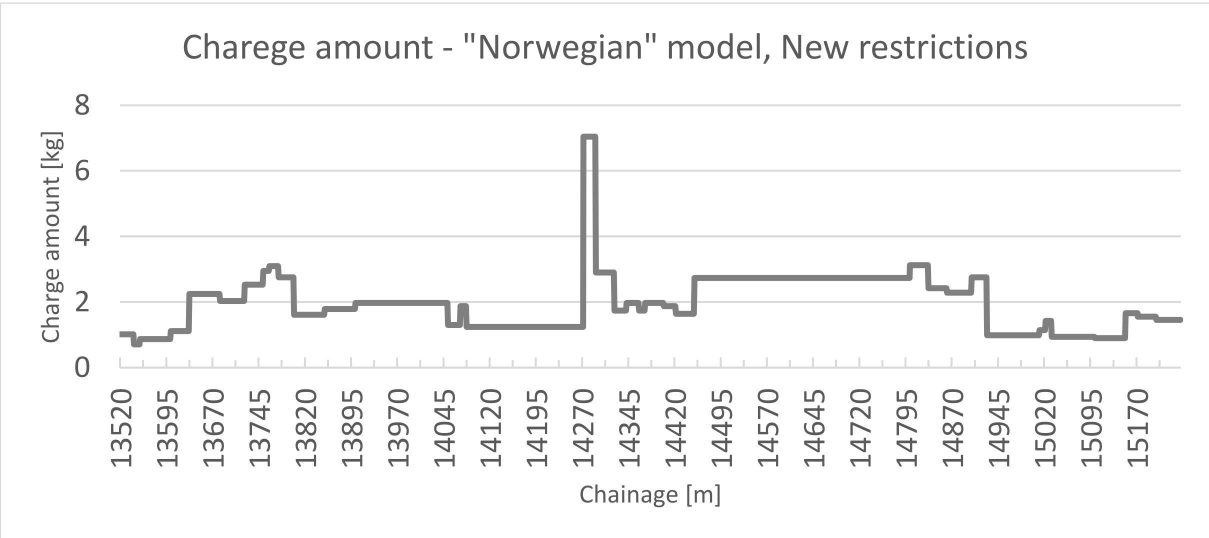

Figure 22 Estimated charge amount - ''Norwegian'' Model, New Standard limits

Comparison:

The results obtained by following the Old Standard limits give the result over 4 times higher than the new limits. Sections from 13775 to 13800, 14900 to 14925 and 14270 to 14290 where due to the Figure 22 vibration limit equals 45mm/s does not influence the correlation strongly. Two data sets have cross correlation of R^2=0,68 (Figure 23).

Conclusions:

Based on industry experience, the combination of The ‘‘Norwegian’’ Model and the NS 8141.E limits gives results that are considered to be incorporated in the scope of most predictable solutions. When it comes to the combination of The ‘‘Norwegian’’ Model and the New Limits, the results are considered to be out of the scope of most predictable solutions.

The correlation between two methods was not affected considerably by three sections where, due to NS 8141-1:2102+A1:2013, the vibration limit equals 45mm/s.

This comparison might indicate that the ‘‘Norwegian’’ Model is not always influenced by single increments of maximum charge limits. Section from 14270 to 14290 represent higher maximum charge amount. Due to formula (6), both site parameter (low value) and vibration limit (high value) increased the final result. In the cases of the sections from 13775 to 13800 and 14900 to 14925, the site parameter was high and reduced the final result of the maximum charge.

The correlation of charge amount results obtained from the two standards can show that vibration limits influence the final trend of the results only to a certain degree, since both methods have similar trends.

However, the vibration limits influence the amount of charge. As a result, the average charge amount considering NS 8141.E is around 3,9 times higher than the result obtained following NS 8141-1:2102+A1:2013.

The main conclusion arising from the analysis is that one should not combine The “Norwegian” Method with the Norwegian Standard NS 8141-1:2102+A1:2013.

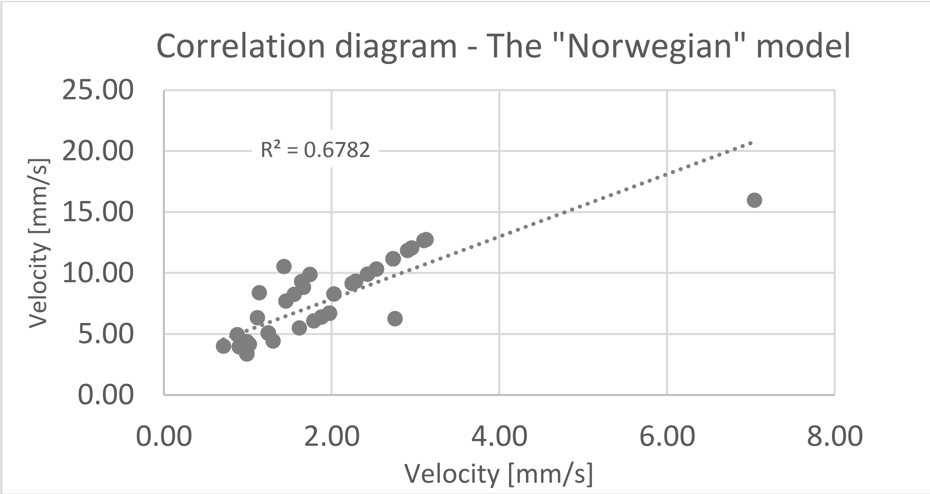

Figure 23 Correlation diagram - The ''Norwegian'' Model

8.1.2 NTNU method

a) NS 8141.E limits

Figure 24 presents charge amount estimated using NTNU Method for vibration limits obtained through NS 8141.E. The average charge for this model equals 11,97kg. The straight line presents the charge used to charge one 5.3 meters long drill hole (8,3 kg).

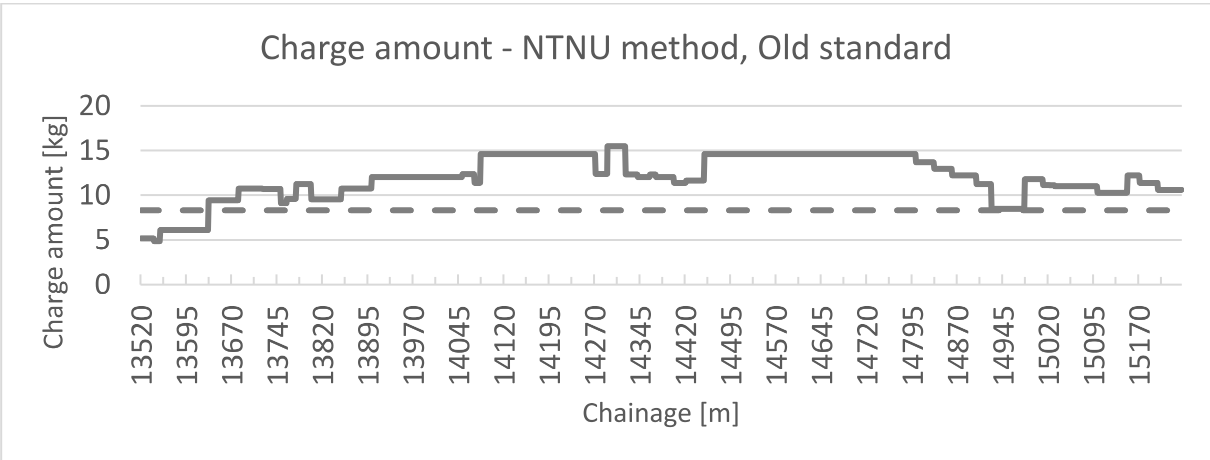

Figure 24 Estimated charge amount - NTNU Method, NS 8141.E limits

b) NS 8141:NS 8141-1:2012+A1:2013 limits

Figure 25 presents charge amount estimated using New Standard Method for vibration limits obtained through NS 8141-1:2102+A1:2013. The average charge for this model equals 2,17kg. 3,1kg is equivalent to 2m borehole length.

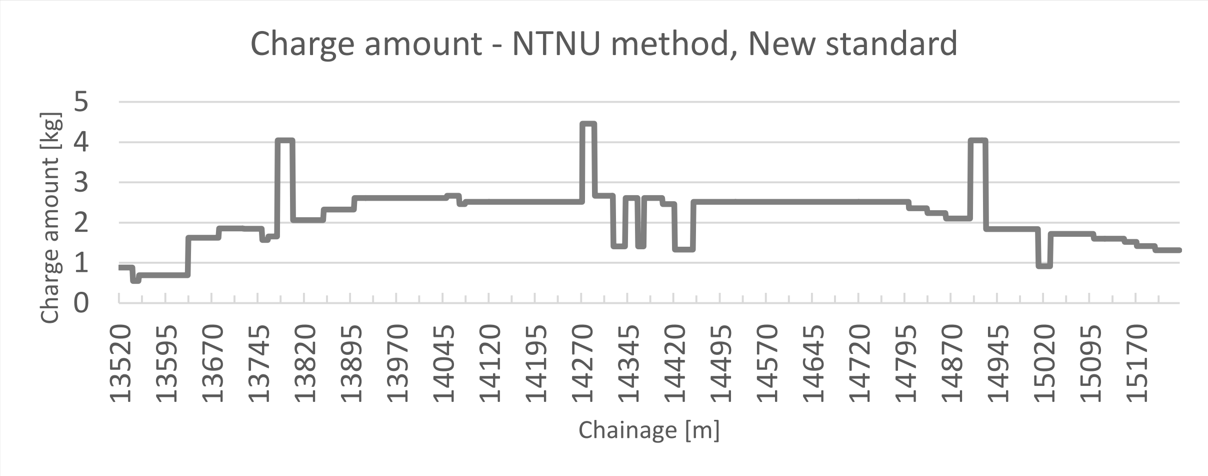

Figure 25 Estimated charge amount - NTNU Method, New Standard limits

Comparison:

The results obtained by following the old limits give a result 5,6 times higher than the new limits. The two data sets have a cross correlation of R^2=0,35.

The correlation is affected much by sections from 13775 to 13800 and from 14900 to 14925 where due to the NS 8141-1:2012+A1:2013 limits total loading per hole equals 4,04kg and 11,25kg due to NS 8141.E limits. Additionally, the correlation is affected much by section from 14270 to 14290 where due to the NS 8141-1:2012+A1:2013 limits total loading per hole equals 4,46kg and 12,40kg due to NS 8141.E limits. After the exclusion of the values above, the cross correlation becomes equal to 0,59 (Figure 26).

Conclusions:

Based on the industry experience, the combination of The NTNU Method and NS 8141.E limits gives the results that are considered to be incorporated in the scope of most predictable solutions. When it comes to the combination of The NTNU Method and New Limits the results are considered to be out of the scope of most predictable solutions.

The correlation between the two methods was affected by 3 tunnel sections. This situation is a result of high percentage differences between vibration limit values obtained through NS 8141-1:2012+A1:2013.

The correlation of charge amount results obtained from the two standards can show that the vibration limits influence the final trend of the result only to a certain degree since both methods have similar trends (Figure 26)

However, the vibration limits influence the amount of charge. As a result, the average charge amount considering NS 8141.E is around 5,6 times higher than the result obtained following NS 8141-1:2012+A1:2013.

The main conclusion arising from the analysis is that one should not combine NTNU Method with the Norwegian Standard NS 8141-1:2102+A1:2013.

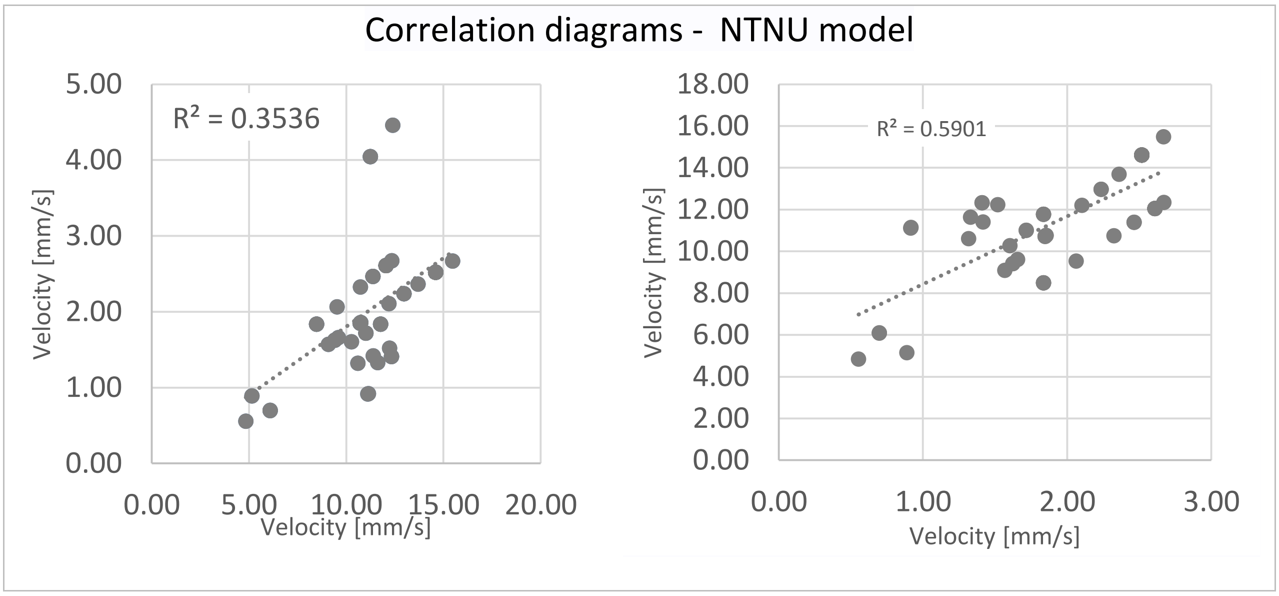

Figure 26 Correlation diagrams - NTNU model, Standard Method a) before improvement b) after improvement

8.1.3 ISEE 1998 method

a) NS 8141.E limits

Figure 27 presents the charge amount estimated using the ISEE 1998 method for vibration limits obtained through NS 8141.E. The average charge for this model equals 7,4kg. The straight line presents the charge used to charge one 5.3 meters long drill hole (8,3 kg).

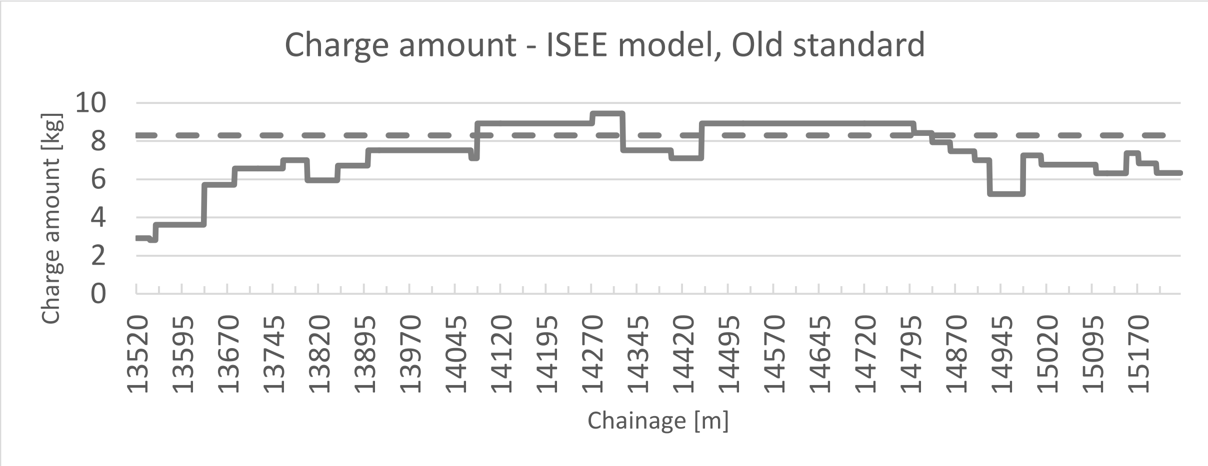

Figure 27 Estimated charge amount - ISEE 1998 Method, NS 8141.E limits

b) NS 8141-1:2012+A1:2013 limits

Figure 28 presents the charge amount estimated using ISEE 1998 Method for vibration limits obtained through NS 8141-1:2012+A1:2013. The average charge for this model equals 1,34kg. 3,1kg is equivalent to a 2m borehole length.

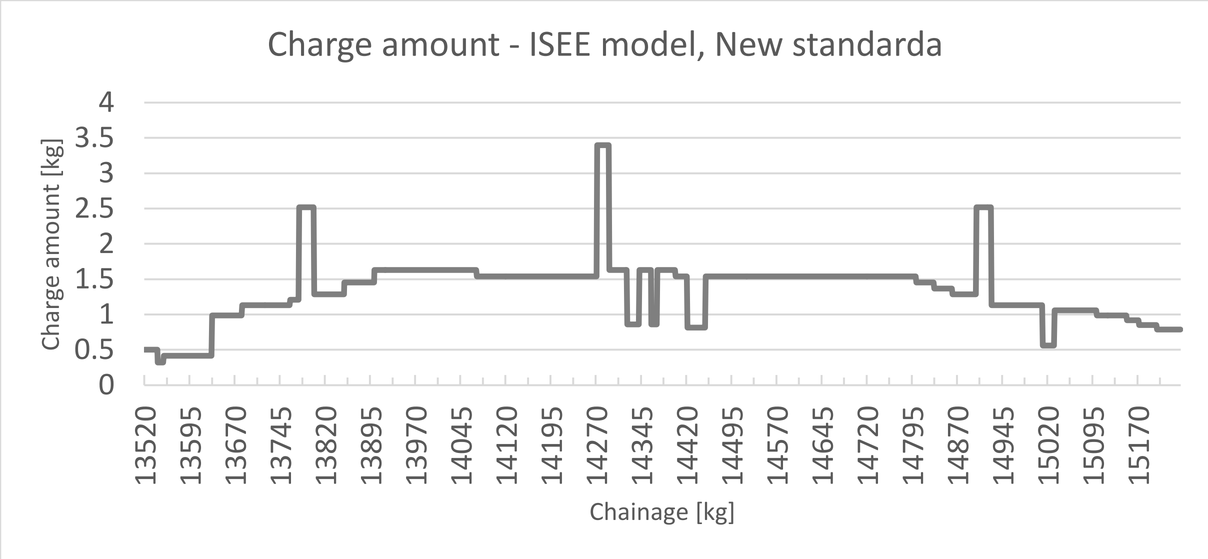

Figure 28 Estimated charge amount - ISEE 1998 Method, New Standard limits

Comparison:

Results obtained by following the NS 8141.E limits give the result 5,45 times higher than the NS 8141-1:2012+A1:2013 limits. The two data sets have a cross correlation of R^2=0,4. The correlation is affected much by sections from 13775 to 13800, 14900 to 14925 where due to the NS 8141-1:2012+A1:2013 limits total loading per hole equals 2,52kg and 7,01kg due to NS 8141.E limits. Additionally, the correlation is affected much by section from 14270 to 14290 where due to the NS 8141-1:2012+A1:2013 limits total loading per hole equals 4,46kg and 3,40kg due to NS 8141.E limits. After the exclusion of the values above, the cross correlation becomes equal to 0,6 (Figure 29).

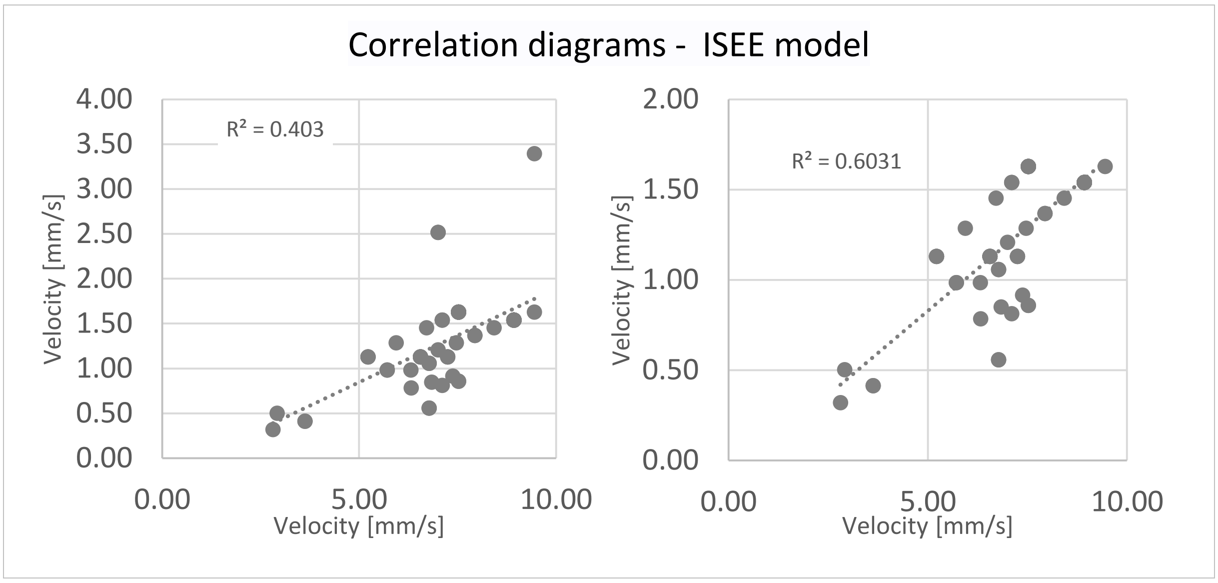

Figure 29 Correlation diagrams - ISEE model, a) before improvement b) after improvement

Conclusions:

Based on industry experience, the combination of The ISEE 1998 Method and Old limits gives results that are considered to be incorporated in the scope of most predictable solutions. When it comes to the combination of The NTNU Method and New Limits the results are considered to be out of the scope of most predictable solutions.

The correlation between the two methods was affected by 3 tunnel sections. This situation is a result of high percentage differences between the vibration limit values obtained through NS 8141-1:2102+A1:2013.

The correlation of the charge amount results obtained from the two standards can show that the vibration limits influence the final trend of the result only to a certain degree, since both methods have similar trends.

However, the vibration limits influence the amount of charge. As a result, the average charge amount considering NS 8141.E is around 5,45 times higher than the result obtained following NS 8141-1:2102+A1:2013.

The main conclusion arising from the analysis is that one should not combine ISEE 1998 Method with the Norwegian Standard NS 8141-1:2102+A1:2013.

9. Analysis and findings - vibrations

In this chapter, all the findings will be combined and analysed together. It was found to be clearer to describe and analyse the combined results in one chapter. Additionally, the most important findings will be presented.

9.1 Results

Values obtained by following the Old Standard vibration velocity limits are from 4 to 9 times higher than the values obtained by using the New Standard. Based on the experience of NTNU and Reinertsen AF, it was noticed that the Norwegian Model, New NTNU method and ISEE 1998 method produce the most possible results if the NS 8141.E limits were applied. The values obtained by using the NS 8141-1:2012+A1:2013 method were considered to be too high.

Values obtained by following the New Standard limits are more conservative than the values obtained by using the New Standard. Based on the NTNU and Reinertsen experience, it was noticed that the New Standard Method gives reliable results if the NS 8141-1:2012+A1:2013 limits were applied. The values obtained by using the Old Standard method were considered to be too high.

When it comes to charge amount estimation, the NTNU model gives the highest values. Previously in the Semester Project of the same author it was concluded that high results might be caused by inappropriate β coefficient estimation. However, the β coefficient estimation was done according to the Master Thesis of Eirin Fjaertoft (Fjærtoft, 2013) and is based on the same in-situ measurements. Thesis of Eirin Fjaertoft was supervised by professionals from NTNU as well as Public Road Administration. Since then it was found highly improbable that the β coefficient estimation was inappropriate.

If an appropriate charge amount estimation method is applied to the corresponding standard (Table 9 Charge amount estimation models and corresponding standards), both the ISEE 1998 method and the ‘‘Norwegian’’ Model presents conservative results in comparison to the New Standard method. The average charge amount for both methods is below 8,1 kg (5 m long borehole). The New Method presented in the NS 8141-1:2012+A1:2013 gives higher results than those achieved by using ISEE 1998 and ‘‘Norwegian’’ Model.

Table 9 Charge amount estimation models and corresponding standards

Model	Standard
New Standard Model	New Standard (NS 8141-1:2012+A1:2013)
‘‘Norwegian’’ Model	Old Standard (NS 8141.E)
NTNU Model	Old Standard (NS 8141.E)
ISEE Model	Old Standard (NS 8141.E)

The selection of an appropriate method it is up to the designer preferences. All the methods are permitted, and give more or less the same results.

9.2 Findings

There are two possible ways for particle velocity value limit estimation in Norway. One can follow either the Old Standard NS 8141.E or NS 8141-1:2012+A1:2013. The New Standard is more conservative when it comes to particle velocity value limits. However, in order to estimate the explosive amount and round length, one needs to apply a proper estimation model. In this project, four estimation models were used and combined with values of particle velocity limits obtained by following two available standards.

The most important conclusions are the following:

1) It is recommended to use Old Methods and Old Standards
2) It is recommended to use New Method and New Standard
3) It is not recommended to mix New Standards and Old Methods as well as Old Standards and New Methods

9.2.1 Recommended approach

As the result of the analysis above, it was decided to follow the Old Methods and Old Standards. Since the NTNU Model was broadly used in Norway, it will be used further in this Thesis. The Model will use the Old Standard (NS 8141.E) as a reference point. Additionally, the analysis uses the same data set as used by Eirin Fjaertoft. The data set has been found reliable since her Thesis was supervised by professionals from both NTNU and Norwegian Public Road Administration.

10. Recommended method

In this chapter the results obtained by following the Old Standard NS 8141.E and the NTNU Model will be presented with respect to buildings and residential areas around the construction area.

10.1 Living areas included in the analysis

In order to select the proper sections of the tunnel considering residential areas, attachment (ReinertsenAS, 2015b) was used. The vibration limits have to be considered in three areas along the tunnel: both portals and the section between 14700 -14950. The vibration limit values are the result of the calculations below. Firstly, the vibration limits had to be established. Secondly, the comparison of the results obtained from the analysis including residential areas and not including residential areas had to be done in order to select more conservative option.

10.2 Limit values for vibration velocity due to NS 8141.E

The limit peak values of vibrations from groundwork (v) are determined by the same formula used in chapter 7.1

$$v = v_0 * F_g * F_b * F_d * F_k \tag{9}$$

When it comes to the buildings, the construction factor, F_b had to be reevaluated. The new calculation of the value of the construction factor, F_b will be presented below.

10.2.1 Construction factor, F_b

$$F_b = k_b * k_m * k_f = 1{,}0 * 1{,}0 * 0{,}7 = 1{,}7$$

k_b is a construction factor depends on the type and design of the construction works. The Norwegian standard specifies the factor for average dwellings and equals 1,0

k_m is a material factor which depends on the type of material in the construction works. The Norwegian standard specifies the factor for unreinforced concrete, brick, hollow concrete blocks, masonry, lightweight aggregate concrete and similar and equals 1,0

k_m=1,0

k_f is foundation factor which depends on the method of foundation for the construction works. A value of k_f=0,7 was applied since the Norwegian standard specifies the factor for embankment, shear wall and column foundation. The Norwegian standard allows the factor k_f=1,0 if the structure is built directly on rock however, this information is not available. Because of this, the safe solution was taken into consideration, i.e. k_f=0,7

As the result of the calculation, three different limit values for vibration velocity due to NS 8141.E were specified.

10.2.2 Limit values for the residential areas

Tunnel portal, direction Trondheim:

$v = v_0 * F_g * F_b * F_d * F_k$= 20 mm/s * 2,5*0,7*1,0*1,0=**35mm/s**

Tunnel portal, direction Stjørdal:

$v = v_0 * F_g * F_b * F_d * F_k$= 20 mm/s * 3,25*0,7*1,0*1,0=**45,5mm/s**

Residential area between PEG 14700 -14950:

$v = v_0 * F_g * F_b * F_d * F_k$= 20 mm/s * 3,0*0,7*1,0*1,0=**42mm/s**

10.3 Application of the new limit values

10.3.1 (PEG 15240):

Figure 30 presents tunnel portal, direction Stjørdal. In this area, the closest building is located 15m over the existing tunnel. The distance between the tunnels at PEG 15200 is 13.0 m. As a result, the distance between the new tunnel tube and the closest existing building can be assumed to be 20m. In this case, the maximum amount of explosive detonated at one moment can be calculated as shown below:

$$v = 500 * c * \frac{Q^{\alpha i}}{d^{\beta i}} \tag{3}$$

In order to calculate the amount of explosive, the equation needs to be transformed as below:

$$Q = \frac{(v * d^{\beta i})}{(500 * \alpha i)}^{1/\alpha i} \tag{4}$$

The explosive amount restriction due to the residential area:

$$\frac{(45.5 * 20.0^{1.2865})^{(\frac{1}{0.8})}}{(500 * 0.9)} = 7.05\ kg$$

The explosive amount restriction due to existing tube of the tunnel:

$$\frac{(132.6 * 13.0^{1.2353})^{(\frac{1}{0.8})}}{(500 * 0.9)} = 11.4\ kg$$

The location of other buildings does not affect the explosive amount since the next building is located 25m from the tunnel:

$$\frac{(45.5 * 25.0^{1.2865})^{(\frac{1}{0.8})}}{(500 * 0.9)} = 11.75\ kg$$

As a result, it is recommended to excavate the first 50m with special respect to the explosive amount = 7.05 kg. The round length should be reduced down to 4 meters since the unit charge per hole for 4m blast is 6,5 kg.

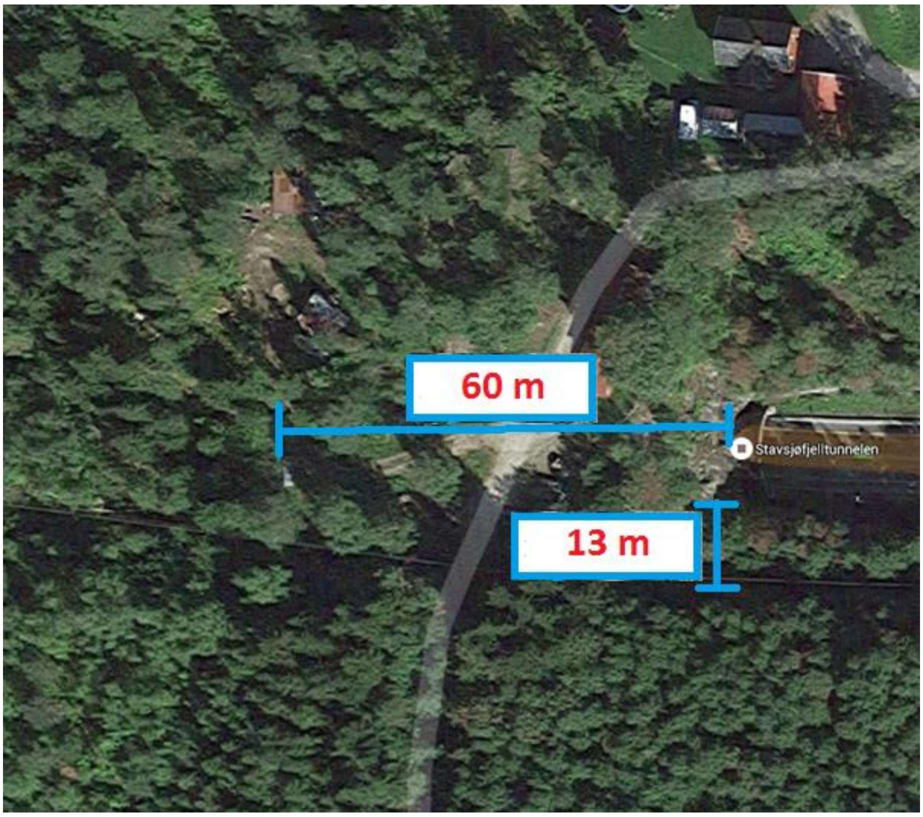

Figure 30 Arial photo of the tunnel portal, direction Stjørdal

10.3.2 (PEG 14700 -14950):

The residential area is located 15m over the new tunnel tube and is around 250m long (Figure 31). In order to calculate the amount of explosive, procedure from 10.3.1 was used with additional help of a MS Excel Sheet (Attachment 1). Figure 32 shows the amount of explosive per hole along the residential area. Figure 33 shows the correlated round length restriction along the residential area.

The explosive amount restriction due to the residential area:

$$\frac{(35 * 15.0^{1.2497})^{(\frac{1}{0.8})}}{(500 * 0.9)} = 2.82\ kg$$

The explosive amount restriction due to the existing tube of the tunnel PEG 14850:

$$\frac{(102.0 * 16.5^{1.2607})^{(\frac{1}{0.8})}}{(500 * 0.9)} = 12.96kg$$

Figure 31 Arial photo of the residential area over the Stavsjøfjelltunnelen PEG 14700 - 14950

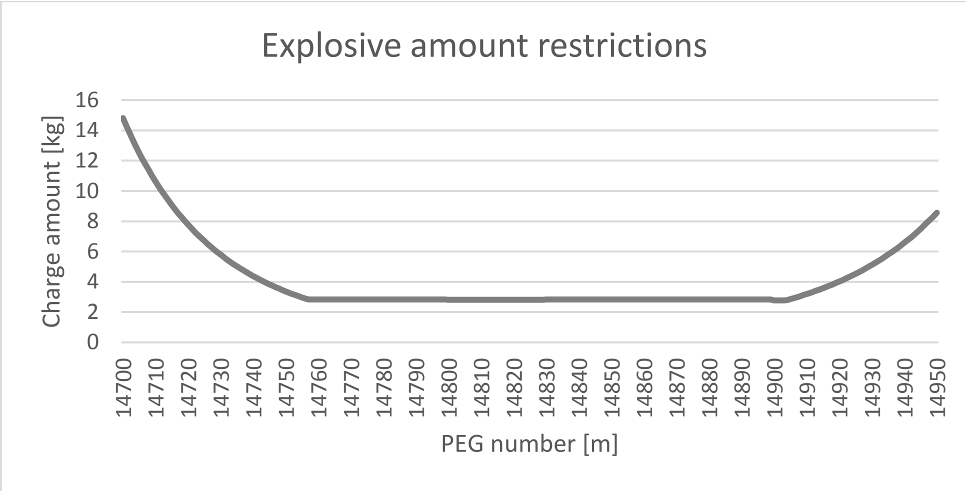

Figure 32 Explosive amount restrictions PEG 14700 - 14950

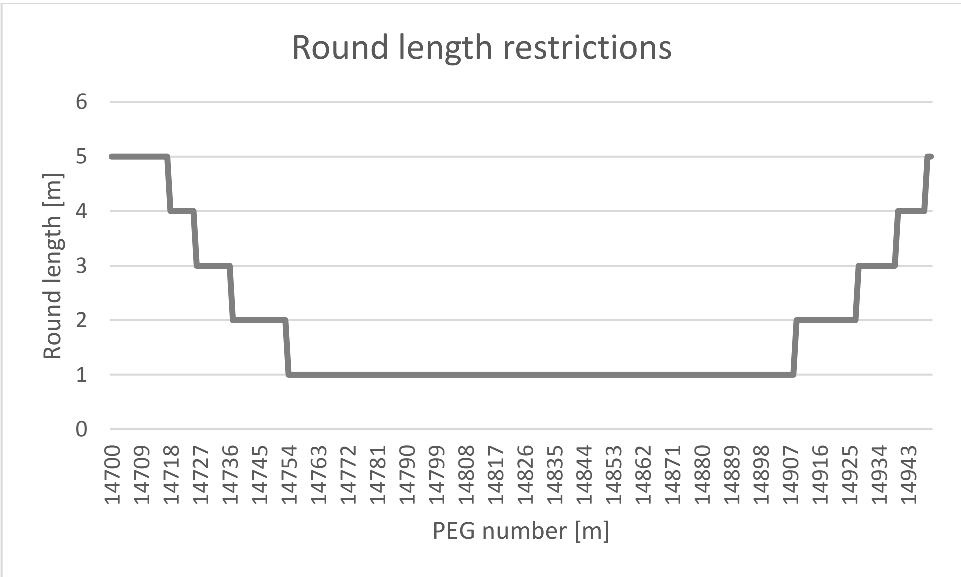

Figure 33 Round length restrictions PEG 14700 - 14950

As a result, it is recommended to excavate the tunnel under the residential area with special respect to the explosive amount presented in Figure 32. The round length should be reduced down to the values presented in Figure 33.

10.3.3 (PEG 13540):

Figure 34 shows the tunnel portal, direction Trondheim. In this area the closest building is located 150m from the existing tunnel. In this case the maximum amount of explosive detonated at one moment can be calculated as shown below:

The explosive amount restriction due to residential area:

$$\frac{(35 * 150^{1.67})^{(\frac{1}{0.8})}}{(500 * 0.9)} = 1432.72\ kg$$

The explosive amount restriction due to existing tube of the tunnel PEG 13520:

$$\frac{(102.0 * 10.0^{1.214})^{(\frac{1}{0.8})}}{(500 * 0.9)} = 5.15 kg$$

The explosive amount restriction due to existing tube of the tunnel PEG 13630:

$$\frac{(85.0 * 13.5^{1.1855})^{(\frac{1}{0.8})}}{(500 * 0.9)} = 6.09 kg$$

At the same time PEG no 13630 is the last meter where F_b was set to equal 1.7 due to the rock support in the existing tunnel. From that moment F_b changes to 2.04 and at the same time the maximum charge equals:

$$\frac{(102.0 * 14.0^{1.1973})^{(\frac{1}{0.8})}}{(500 * 0.9)} = 9.41 kg$$

Figure 34 Arial photo of the tunnel portal, direction Trondheim

The analysis shows that the explosive amount restriction concerning the residential area around the tunnel portal, direction Trondheim, are way over the restricted level. While blasting, one should pay special attention to explosive amount restrictions being a result of the close distance between the two tunnels. As a result, it is recommended to excavate the first 90m with special respect to the explosive amount presented in Figure 35. The round length should be reduced down to the values presented in Figure 36.

10.4 Final charge amount restriction and round length

Figure 35 shows three sets of data. The red dashed line is equal to 8.3 kg, which corresponds to the biggest amount of the unit charge used while drilling 5.3m length rounds. The green dotted line, the charge recommendation when not taking the residential areas into consideration. The filled blue area, represents the charge recommendation when taking the residential areas into consideration. One should note that the lowest value on the chart is

2,3kg. It is assumed that the most vibration sensitive area should be extensively monitored when it comes to the vibration levels while blasting. The process of vibration measuring should be externally evaluated. Because of this, precise recommendation concerning the charge amount and exact round length was not suggested.

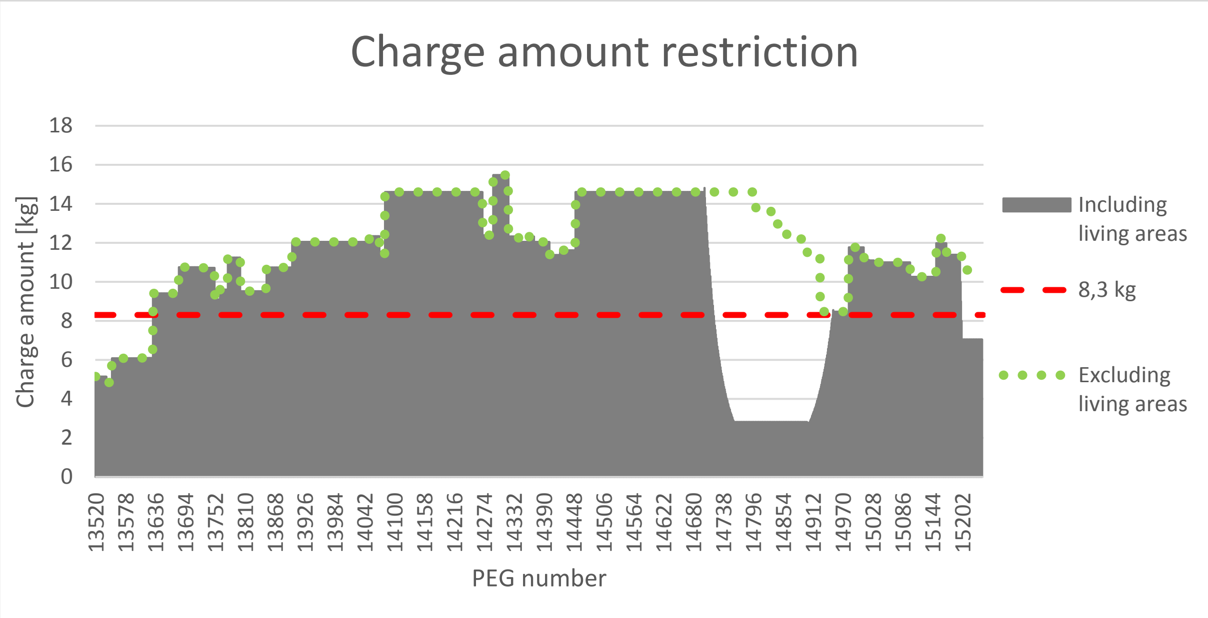

Figure 35 Charge amount restriction along the Stavsjøfjelltunnelen

As the result of the charge amount restriction analysis, the suggestion for the round length has been done and presented in Figure 36.

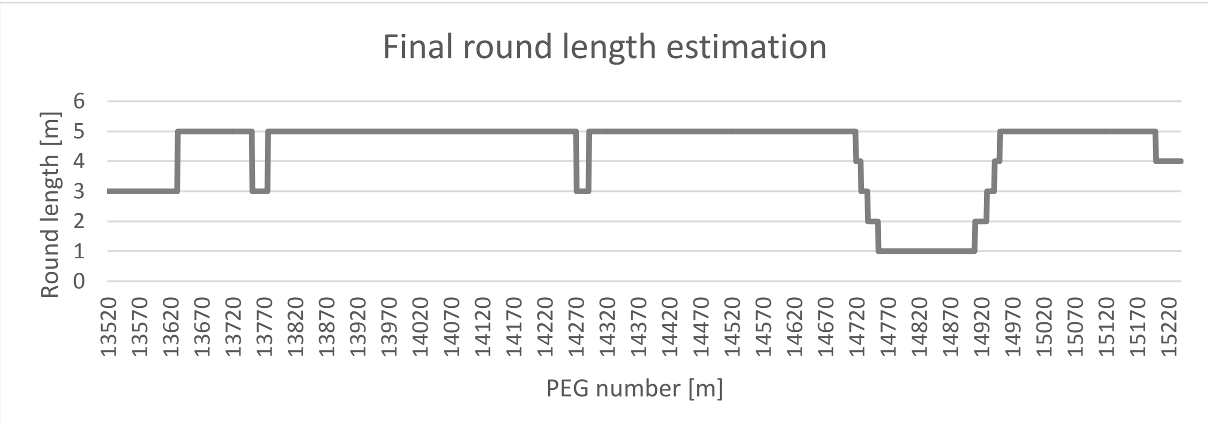

Figure 36 Round length estimation for Stavsjøfjelltunnelen

One should notice that the area between PEG 14700 – 14950 where the charge amount restriction is below 3,1 kg and the round length estimation is below 2,3 m should be additionally evaluated. Evaluation and adjustment should be done during estimation based on the vibration control. However, for the calculation purposes round length was set to be 1 m in the areas below 3,1 kg of charge and the minimum unit charge amount was set to be 2,3 kg using interpolation.

11. Drill and Blast design

For decades Norway has developed general rules for Drill and Blast design. The Drill and Blast design rules and their application with respect to the explosive amount restrictions will be presented in this chapter. The experience considers such topics as cut design, hole diameter, hole length, face shape. This chapter presents general information about the currently used Drill and Blast technique in Norway. The information presented in this chapter comes from personal experience, contacts with the General Road Administration, NTNU technical report (Zare, 2007b) and contact with SANDVIK Company.

11.1 Tunnel blasting

Tunnel blasting differs from surface blasting where the drilling is done parallel to the break-out or free surface. In order to excavate the tunnel, the need for an opening in the surface appears. Such an opening is called a cut. Located around the cut are holes that belong to the easer area. The easer area has the lowest density of holes per m^2 since the only rule of the easers is to continue tunnel opening. Cut holes are fully loaded and located close to each other to secure the proper opening. The bottom holes are called invert and are fully loaded. In order to achieve good contour quality the fill out of the holes with the explosive in the contour holes as well as helpers (row nearest to contour) is reduced to respectively 25% and 60%. The distance between the rows - B (burden) as well as S (spacing) is reduced. Figure 37 presents the hole placement over the tunnel face. (Zare, 2007b)

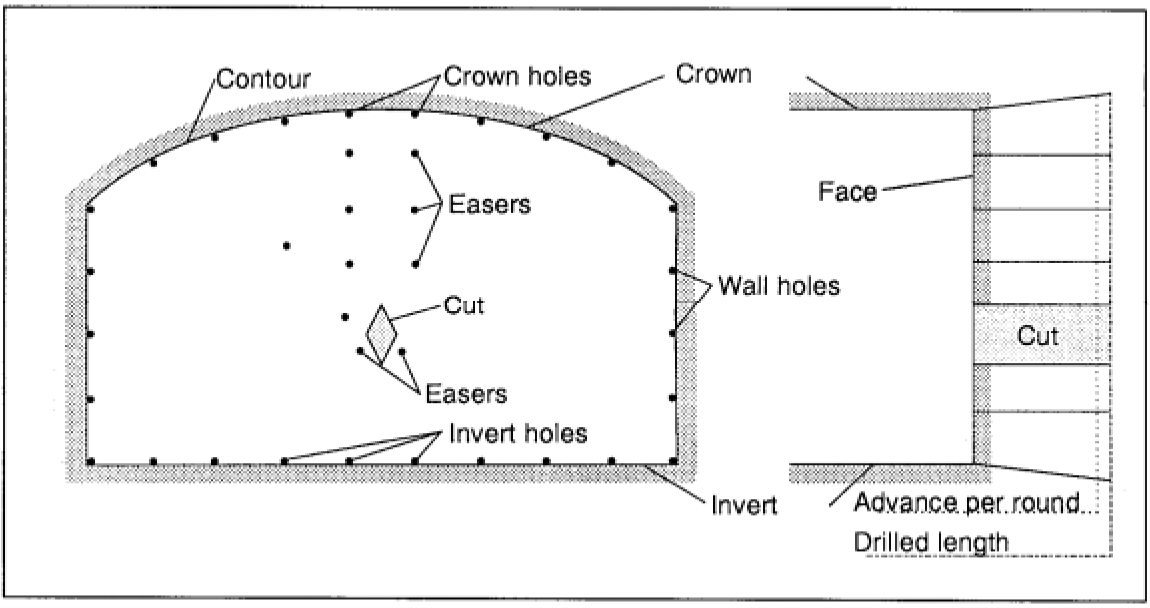

Figure 37 Holes placement on the tunnel face

11.2 Cut design

The most common drill bit diameters used currently in Norway are 48 and 64mm. Almost 100% of the cut designs are parallel cut designs. With increasing round lengths the fan cuts and V-cut designs become ineffective. (Olsen, 2014)

There are three factors influencing the cut design: geology, machinery, and the individual experience of the blaster. When it comes to the diameter of the riming holes, standard 102mm diameter holes are mostly used, however, it is possible to increase the diameter to 127mm if the geology and the round length requires such adjustments. The blast holes located closest to the riming holes are usually placed in a distance 1.5- 2.0 times the reaming holes' diameter (between 15 – 25 cm). The amount of necessary rimming holes is presented in Figure 38 (Zare, 2007b).

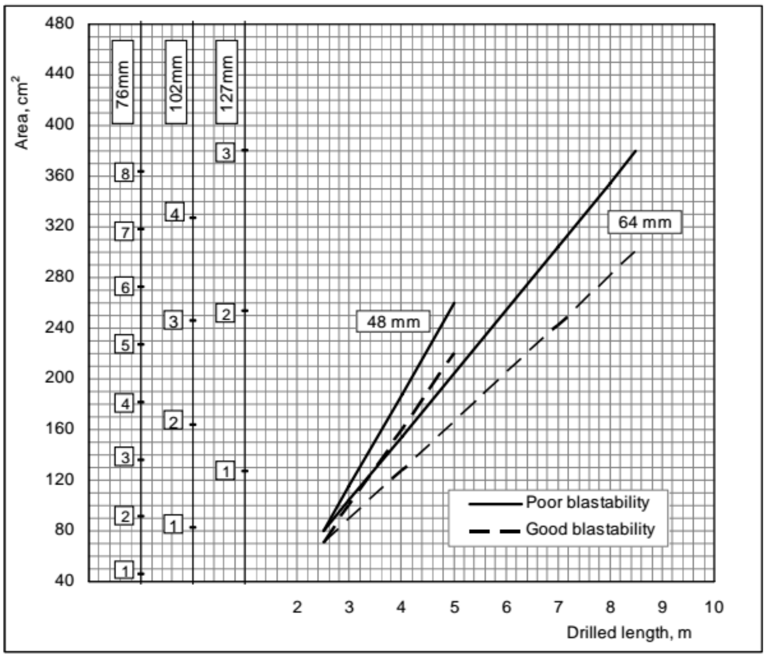

Figure 38 Estimation of rimming holes dimensions and amount

While designing the cut, one should consider the necessary expansion area. The expected area of the rock to be blasted has to have a minimum of 80% free space that is created by the riming holes at the beginning, and open itself as the explosion continues. The cut design procedure used in the D&B proposition for this master Thesis will not be shown, however, the design follows main rules included in the NTNU report (Zare, 2007b).

11.3 Easer, Invert, Contour and Row nearest contour design

Standard drilling length in the Norwegian tunnelling industry is 5.3m and 6.2m has been used in larger railway and road tunnels (Olsen, 2014). The distance between holes as well as density of the holes location are presented in Equation 10.

Table 10 Guiding values for burden, spacing and stopping area for 48 mm drill holes

Type of hole	Burden, V	Spacing, E
Contour		
Good blastability	0.8 – 1.0 m	0.7 – 1.0 m
Poor blastability	0.7 – 0.9 m	0.6 – 0.9 m
Row nearest contour		
Good blastability	1.0 m	1.1 m
Poor blastability	0.9 m	1.0 m
Invert hole		
Good blastability	1.0 m	1.0 m
Poor blastability	0.8 m	0.8 m
Easer		
Good blastability	$F_s = 1.8\ m^2$	
Poor blastability	$F_s = 1.3\ m^2$	

F_s is a stopping area, as function of burden and spacing. Usually the ratio between spacing and burden is set to be 1.2. However, this value can vary based on the rock conditions and the geometry of the cross section. (Zare, 2007b)

$$F_s = \frac{E_s * V_s}{k_{bl}} = \frac{(1.2 * V_s) * V_s}{k_{bl}} \qquad (10)$$

E_s= spacing of stoping holes

V_s=burden of stoping holes

11.4 Face shape and holes direction

When it comes to the tunnel face shape, the “tilbaketrukket kontur” or “D-shape” - pulled face shape, is most widely used in Norway (Figure 39). The point is to obtain the face shape where the cut is placed 1-1,5m in front of the contour. In this case, pull out and contour quality is improved by decreasing the confinement of the explosives. As a result, applying this tunnel face shape reduces the round time up to one hour due to reduced required hole amount (influence on drilling time and charging time) as well as scaling. It is easier to scale the rounded face than have 90 degree corners where the access for the hydraulic hammer is limited (Hendenberg, 2016). In order to improve the contour quality, the second contour line can be drilled parallel to the contour holes. The longitudinal distance between the two following holes should be the biggest when it comes to the contour and the helpers. The easer holes are designed in a way to have inclination upwards. Additionally, the horizontal angle could be set to be 0 in order to reduce the collaring time. In order to provide sufficient water irrigation one site or two sites ditch could be blasted.

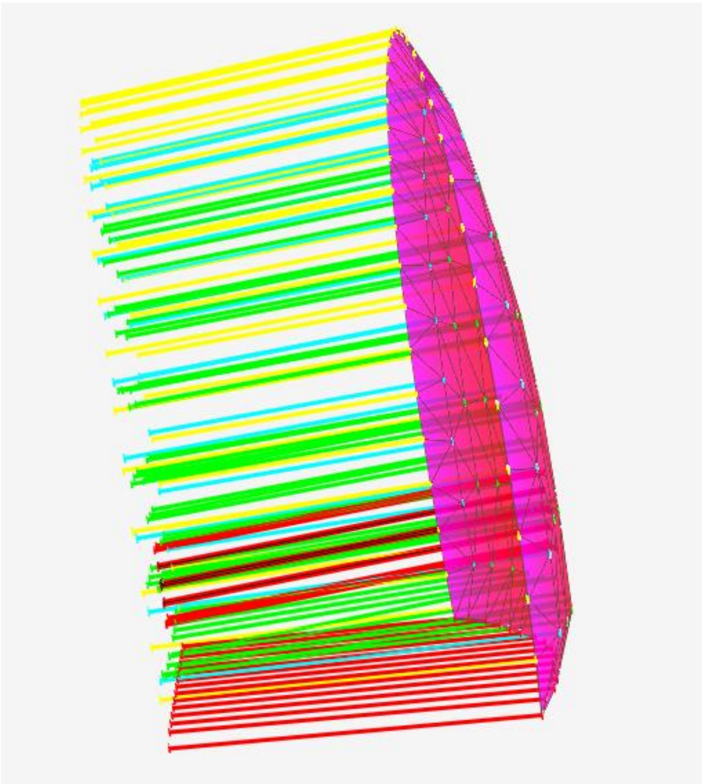

Figure 39 D-shaped tunnel face - iSURE - D&B design for poor blastability T10,5 - 134 detonators

11.5 D&B design for Stavsjøfjelltunnelen

In order to calculate the stopping area, F_s, it is necessary to estimate the skill level and round length dependent factor - K_{bl}. In the current project, it was estimated that the skill level is closer to the high level and the drilled length equals 5,3m. Therefore, the skill level was set to be 1,04 when using Figure 40.

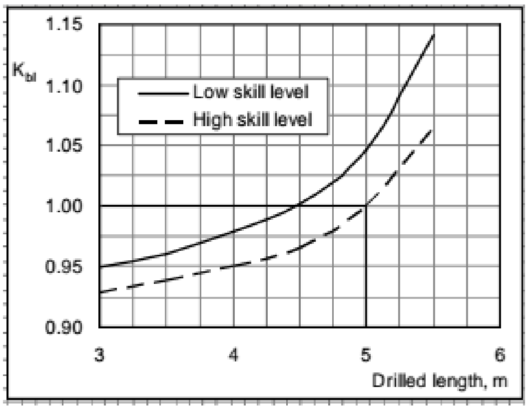

Figure 40 Correlation factor for drilled length K_{bl} (Zare, 2007b)

As a result, the following values of burden and spacing, according to blastability, were obtained and suggested in the Table 11. Based on these values, two blast designs for both medium and bad blastability are presented. The D&B designs can be considered as a suggestion and start point in order to achieve the individual D&B design for this project.

Table 11 Guiding values for burden, spacing and stopping area for Stavsjøfjelltunnelen

Contour:			Row nearest contour:		
Blastability	**Bad**	**Middle**	**Blastability**	**Bad**	**Middle**
E_s [m]	0,75	0,85	**E_s [m]**	1,0	1,1
V_s [m]	0,8	0,9	**V_s [m]**	0,9	1,0
Easer:			**Invert:**		
Blastability	**Bad**	**Middle**	**Blastability**	**Bad**	**Middle**
E_s [m]	1,3	1,45	**E_s [m]**	0,8	1,0
V_s [m]	1,1	1,2	**V_s [m]**	0,8	1,0

The Drill and Blast input parameters presented in the Table 11 have been used while creating the D&B pattern in the iSURE program developed by SANDVIK. Figure 41 and Figure 42 present the suggested Drill and Blast design respectively for medium and poor blastability of the Stavsjøfjelltunnelen. As the result of using the input data from Table 11 in the iSURE program the D&B design for the medium blastability results in 113 detonators and the D&B design for the poor blastability results in 134 detonators.

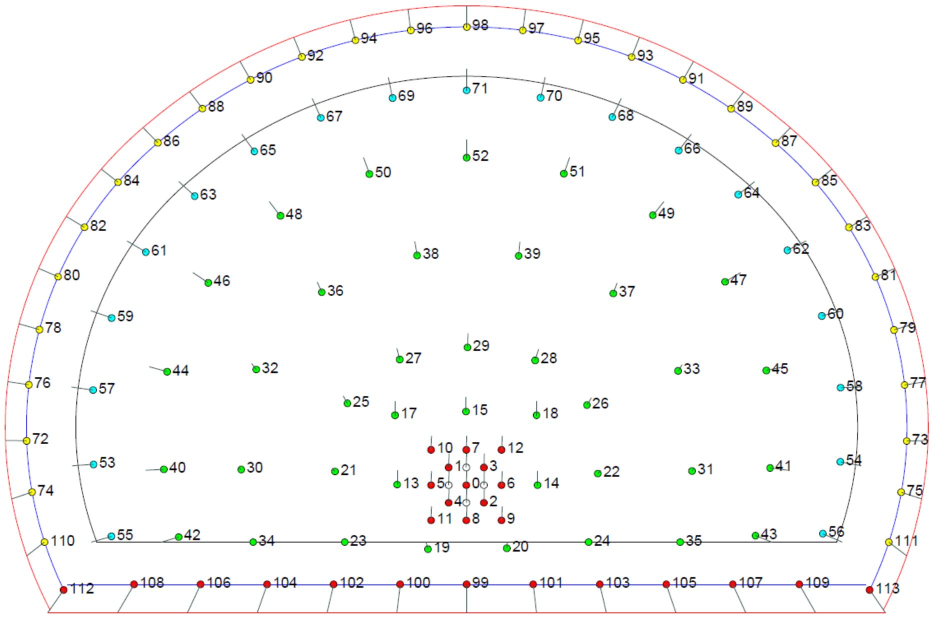

Figure 41 D&B design for medium blastability T10,5 - 113 detonators

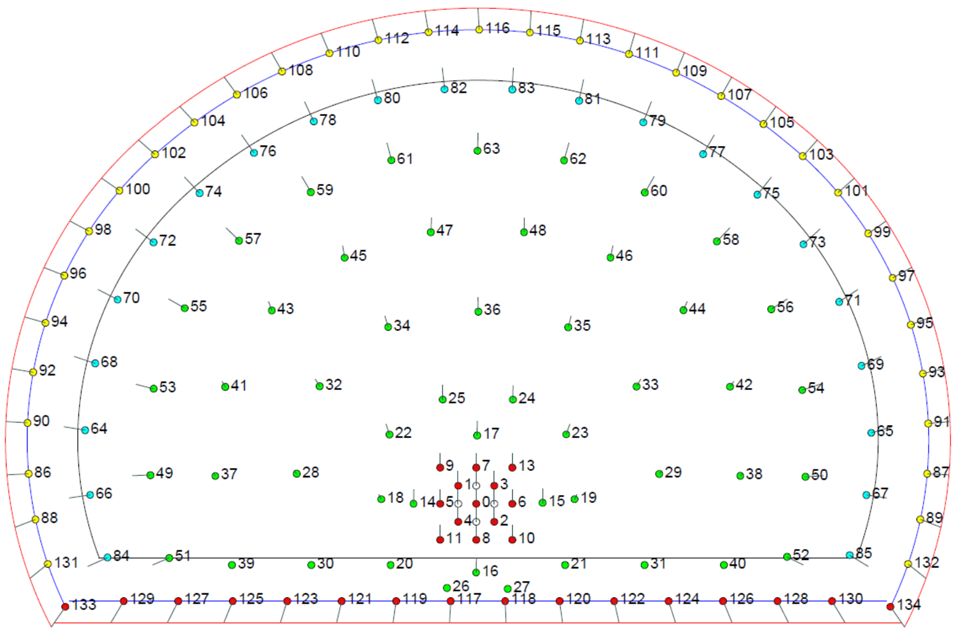

Figure 42 D&B design for poor blastability T10,5 - 134 detonators

11.5.1 Detonators

In order to meet the vibration restrictions, it is suggested to use the Austin Electronic detonators in the Stavsjøfjelltunnelen. The timing of these electronic detonators can be programmed in order to fulfill the users need. The delay can be set between 1 – 10 000 ms with the minimum delay of 1 ms (Austin, 20016). In the case of the Stavsjøfjelltunnelen, the delay could be set to 25ms in the cut, 50ms around the cut and 75 ms closer to the contour. (Bruland, 2016)

11.5.2 Applied hole depths, column lengths, stemming lengths and total explosive weight

Table 12 and Table 15 show the recommended charging amounts, column lengths and stemming lengths for different hole types and hole depths. The hole depth was adjusted based on the iSURE program recommendations. Additionally, the column length and stemming length was presented. The information included in the tables can be used as a starting point for the charging technique in the Stavsjøfjelltunnelen and can be adjusted if needed.

Table 12 Recommended hole depth - 5.3m drilled length

5.3 m				
	Hole depth [m]	**Column length [m]**	**Stemming length [m]**	**Total weight [kg]**
Contour	5.3	4.6	0.7	2.3
2nd contour	5.3	4.6	0.7	5
Field	5.3	4.05	1.25	7.3
Invert	5.3	4.6	0.7	8.3
Cut	5.3	4.6	0.7	8.3

Table 13 Recommended hole depth - 4.3m drilled length

4.3 m				
	Hole depth [m]	**Column length [m]**	**Stemming length [m]**	**Total weight [kg]**
Contour	4.3	3.6	0.7	1.8
2nd contour	4.3	3.6	0.7	3.9
Field	4.3	3.05	1.25	5.5
Invert	4.3	3.6	0.7	6.5
Cut	4.3	3.6	0.7	6.5

Table 14 Recommended hole depth - 3.3m drilled length

3.3 m				
	Hole depth [m]	**Column length [m]**	**Stemming length [m]**	**Total weight [kg]**
Contour	3.3	2.65	0.65	1.3
2nd contour	3.3	2.65	0.65	2.9
Field	3.3	2.1	1.2	3.8
Invert	3.3	2.65	0.65	4.8
Cut	3.3	2.65	0.65	4.8

Table 15 Recommended hole depth - 2.3m drilled length

2.3 m				
	Hole depth [m]	**Column length [m]**	**Stemming length [m]**	**Total weight [kg]**
Contour	2.3	1.7	0.6	0.85
2nd contour	2.3	1.7	0.6	1.85
Field	2.3	1.15	1.15	2.1
Invert	2.3	1.7	0.6	3.1
Cut	2.3	1.7	0.6	3.1

Based on the Chapter 10.4 - Final charge amount restriction and round length, the areas along the tunnel where the recommended round length is below 2.3m should be evaluated based on experience gained at the project. An adjustment should be made with respect to the vibration measurements. However, for the calculation purposes round length where the minimum unit charge is below 3,1 kg was set to be 1 m in and the minimum unit charge amount was set to be 2,3 kg using interpolation.

12. Calculation of tunnel construction performance

This chapter will focus on all the activities related to the tunnel excavation process: Drilling, charging, blasting, ventilation, loading, hauling, scaling and rock support. However, the NTNU model considers rock support in a separate report. In order to estimate the total time consumption on tunnel excavation, two NTNU models needed to be combined by the prototype Excel program named Tunnel Excavation Performance (TEP) Model. At first, the results for time consumption of excavation excluding the rock support will be presented. The results are based on TUNSIM program (tunnel construction cost and performance program) which is based on NTNU reports: 2A-05 Drill and Blast Tunnelling Blast Design, 2B-05 Drill and Blast Tunnelling Advance Rate. Secondly, the results of time consumption of rock support based on NTNU report 2F-99 Tunneldrift, Enhetstidsystem for driving, sikring, og innredning will be shown.

12.1 Round cycle

The conventional round cycle was assumed to be used while constructing the Stavsjøfjelltunnelen. The cycle considered by NTNU models consists of:

1. Drilling, charging, blasting
2. Ventilation
3. Loading and hauling
4. Scaling and rock support

When it comes to the three first stages, the time usage can be divided into three following categories of time (Zare, 2007a):

A. Fixed Lost Time

Fixed Lost Time is understood as the time consumption spent on the activities almost independent with regard to variations in round length, the number of crew members and the equipment. For example: Moving the equipment.

B. Proportional Operational Time

Proportional Operational Time is understood as time consumption spent on the activities dependent with regard to variations in round length, the number of crew members and the equipment performance. For example: Face drilling.

C. Incidental Lost Time

Incidental Lost Time is understood as time consumption spent on the random operations, breakdowns or shift changes. A lost time of 10% of the total time consumption is considered to be normal by the prediction model.

12.2 Tunsim input data

In this chapter, the assumptions, input data, equations and charts used by Tunsim are shown. An example of the calculations was not presented, as Tunsim analyses the data automatically. The input data and combined results are presented in the tables in section 12.5.

12.2.1 Drilling

12.2.1.1 Assumptions and input data

The tunnel will be excavated from one side and will have a length of 1730m. The assumed profile is T=10,5 (Figure 43) which gives a theoretical blasting area of 78,83 m^2. As assumed in Chapter 11, a parallel cut will be used.

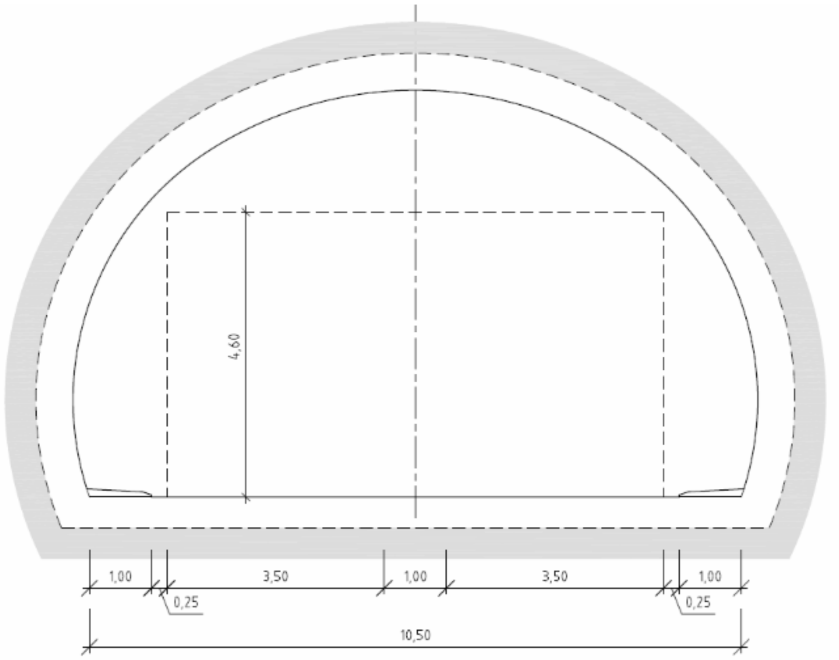

Figure 43 Tunnel profile T 10,5 (StatensVegvesen, 2014)

In the project, 5.3m long 48 mm diameter boring holes were assumed based on SANDVIK recommendations. Due to the NTNU report 2A-05 (Zare, 2007b), 122 holes for poor blastability and 112 for medium blastability should be considered. However, while using iSURE, the amounts of 134 and 113 respectively were obtained. The input data for iSURE were the values shown in Table 11. However, the proposed Drill and Blast design was presented in the Chapter 11.5 The number of rim holes was chosen due to the NTNU recommendation and equals 4x102mm (Zare, 2007b). The calculation will also consider reduced round lengths due to the poor rock condition and the close proximity of residential areas. The prediction assumes rock wear quality described by VHNR parameters = 550, DRI = 49, Drillability set to 'medium' and two blastability settings: poor and medium. More information about the geology can be found in the chapter 4 Geology. The skill level of the tunnel crew was set to be high and the excavation method was chosen to be trackless. The drill rig was assumed to be produced by AtlasCopco with the series number of COP 1838. The number of drilling hammers is 3.

12.2.1.2 Equation presentation following NTNU report: 2B-05 (Zare, 2007a)

Drilling time:

Due to the model, the drilling time has to be calculated for two type of the holes:

Drilling time for the charged holes:

$$T_h = \frac{l_h * N_h}{v_h * N_m} \quad (11)$$

Where:

l_h – drilled length

N_h – number of charged holes

v_h – net penetration rate for charged holes

N_m – number of drilling hammers

Drilling time for rimming holes:

$$T_g = \frac{l_h * N_g}{v_g * N_m} * 1.25 \quad (12)$$

Where:

l_h – drilled length

N_g – number of charged holes

v_g – net penetration rate for charged holes

N_m – number of drilling hammers

Time for Moving:

$$T_g = \frac{N_h + 2 * N_g}{N_m} * t_f \qquad (13)$$

t_f – unit time for moving per hole, dependent on hole length (Figure 44)

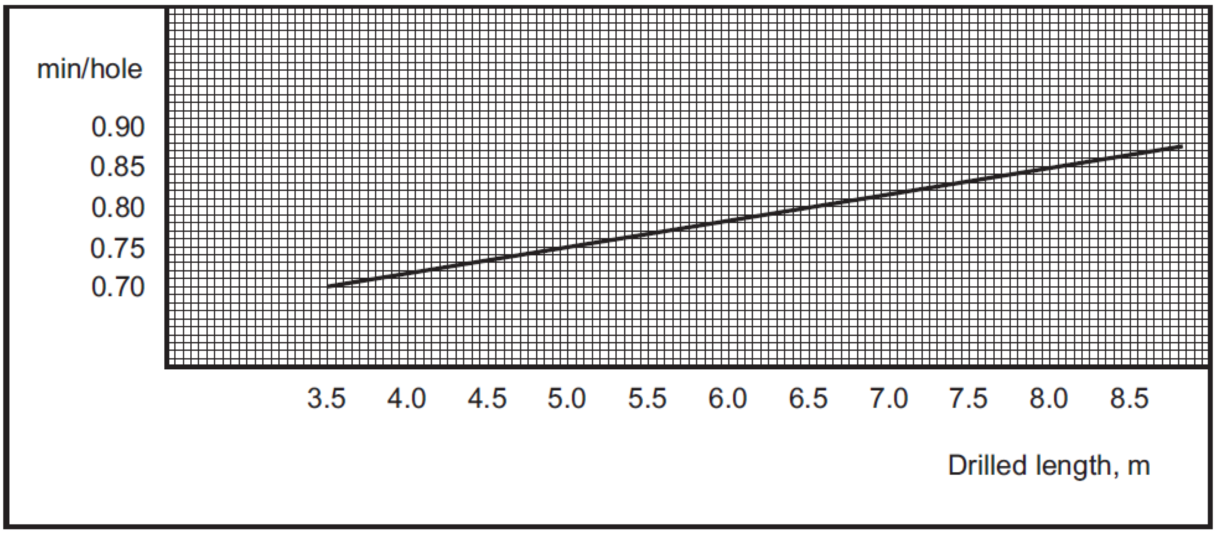

Figure 44 Time for moving t_f, per hole (Zare, 2007a)

Changing of Bits:

$$T_k = \frac{l_h * (2 * N_g + N_h) * f_k * t_k}{N_m} \qquad (14)$$

f_k = frequency of bit changing per drilled meter, dependent on VHNR (Figure 45)

t_k = unit time for bit changing (3min according to regulations)

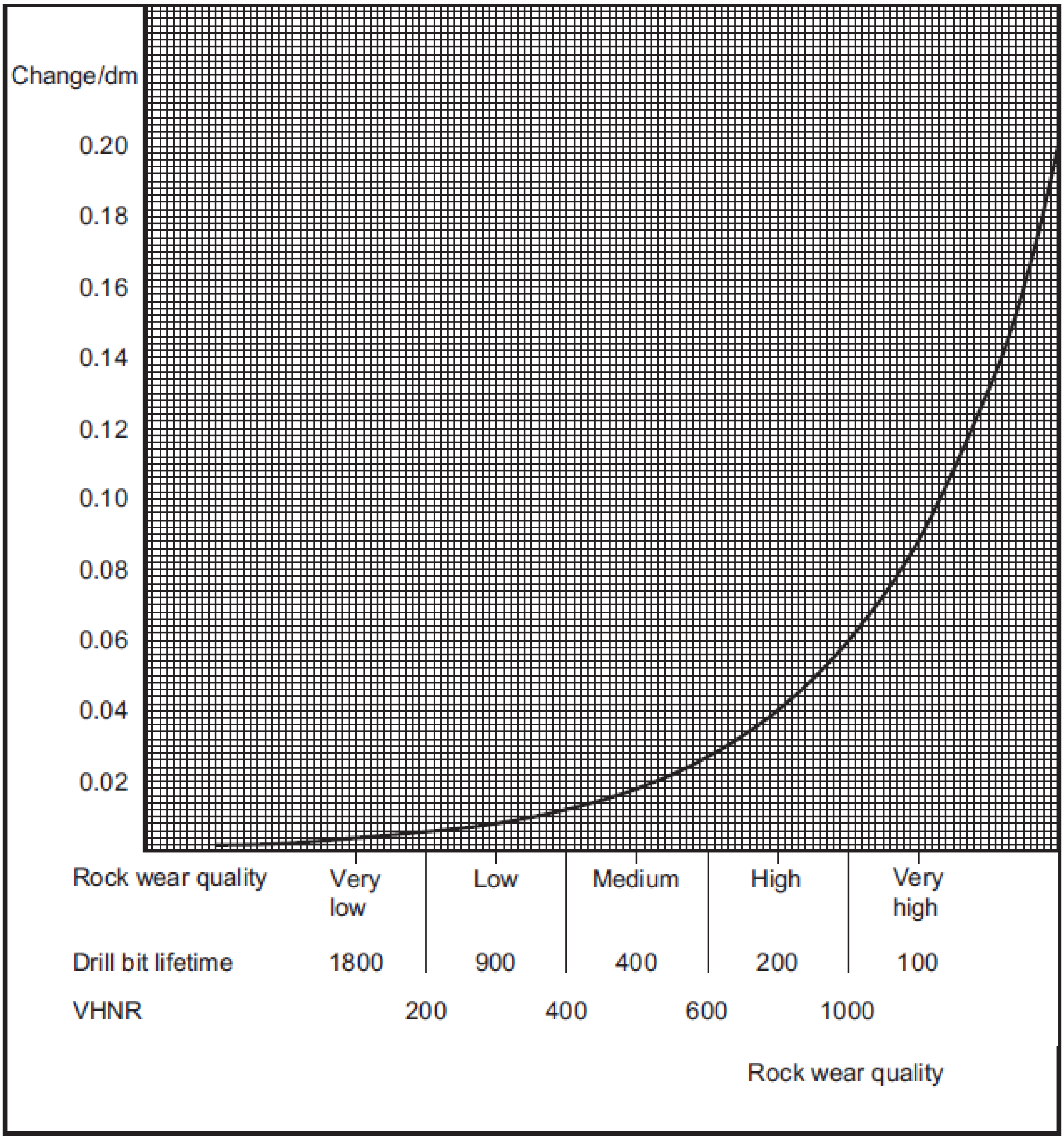

Figure 45 Bit-changing factor f_k, as a function of rock wear quality (Zare, 2007a)

Lack of Simultaneousness:

$$T_{sa} = \left(T_h + T_g + T_f\right) * f_{sa} \tag{15}$$

f_{sa}= correction factor for lack of simultaneousness is dependent on the cross section, net penetration rate, and number of drilling hammers and operators (Figure 46)

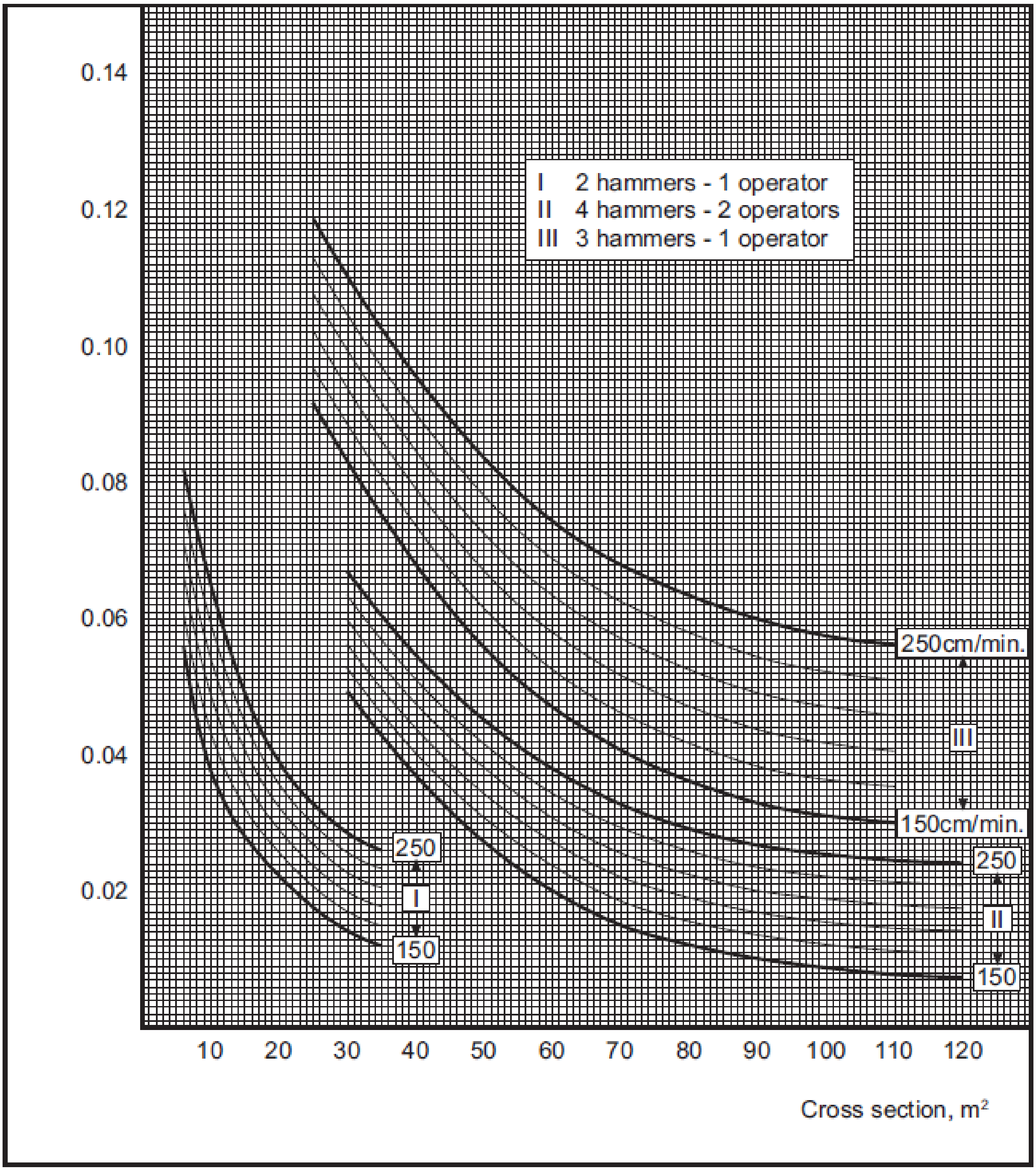

Figure 46 Factor for lack of simultaneousness, f_{sa} (Zare, 2007a)

12.2.2 Charging

The model gives basic recommendations, however, the final charging time might change due to following factors, as mentioned in the NTNU report (Zare, 2007a):

- Amount of the explosives used
- Type of explosives
- Charging capacity
- Number of charging lines
- Number of drillholes
- Drillhole diameter
- Drilled length

The charging time is estimated by the model based on the assumptions mentioned above as well as the current low regulations. It was assumed that the charging process can be carried out after drilling. The charging time depends on the number of holes, the type of explosive, the charging method, as well as the round length. The distinction between ANFO and cartridge explosives has been made. For ANFO, the distinction between 48 mm and 64 mm drill hole diameter has been made. The charging capacity can be regulated when it comes to Emulsion explosives. As such, the charging time is hole diameter independent while using the emulsion explosives. The model includes a distinction in time consumption for 2 and 3 charging lines for ANFO. All factors were included and compared against each other in the Figure 47 where time-determinant charging time, T_{lb}, is presented as a function of number of drillholes, drillhole diameter, number of charging lines and type of explosives. ANFO represents cartridged explosives.

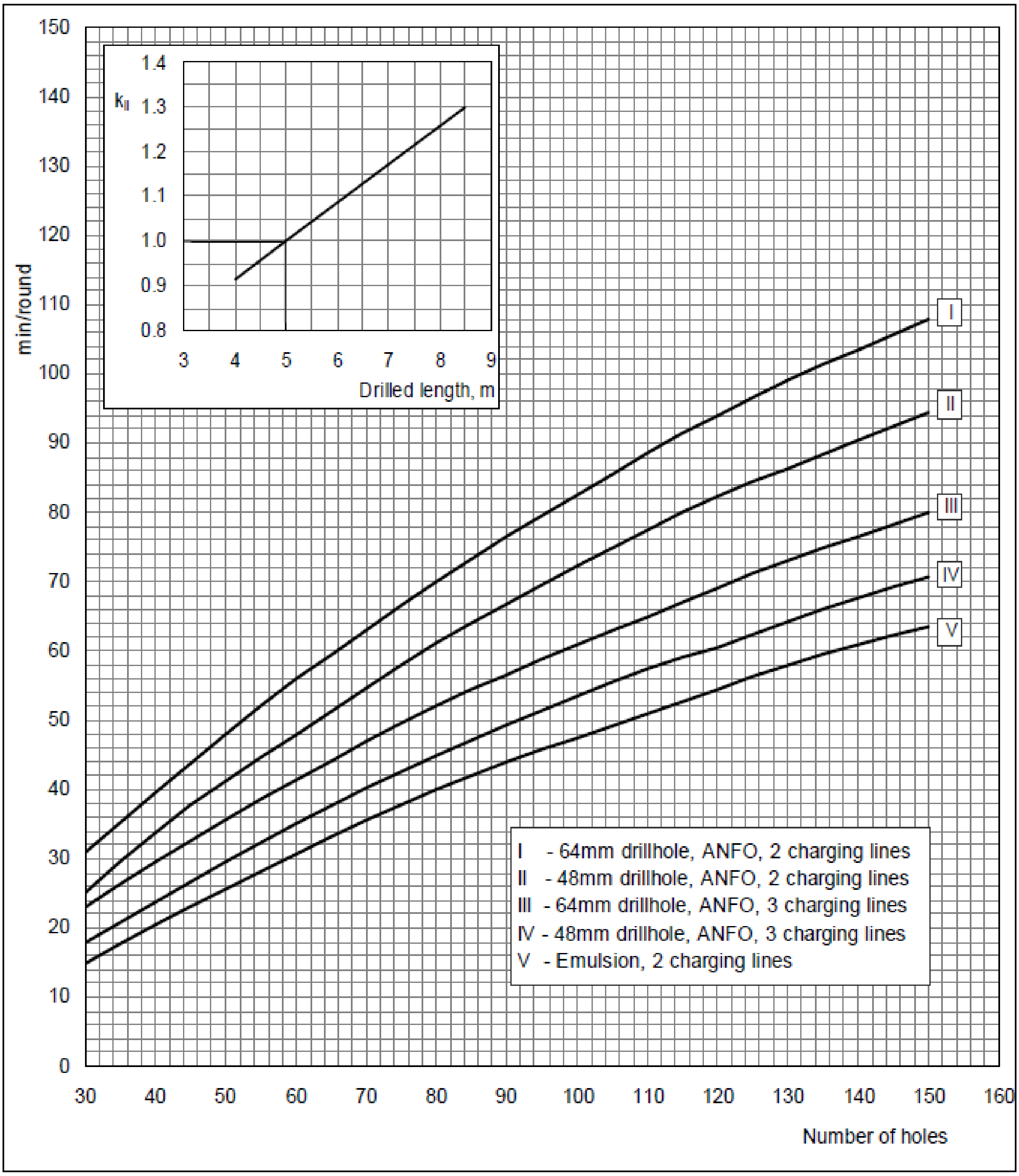

Figure 47 Time-determinant charging time, T_{lb} (Zare, 2007a)

12.2.3 Ventilation break

The aim of a ventilation brake is to reduce the concentration of nitrous gases (NOx) at the tunnel face to below a TLV (Threshold Limit Value) = 2 ppm (parts per million). The estimated time is presented in Figure 48 as a function of explosive type and tunnel cross-section. Either blowing or exhausting ventilation is recommended for tunnels longer than 1000m and larger than 32m^2. It is recommended to keep the distance between the face and the ventilation duct between 30-70m.

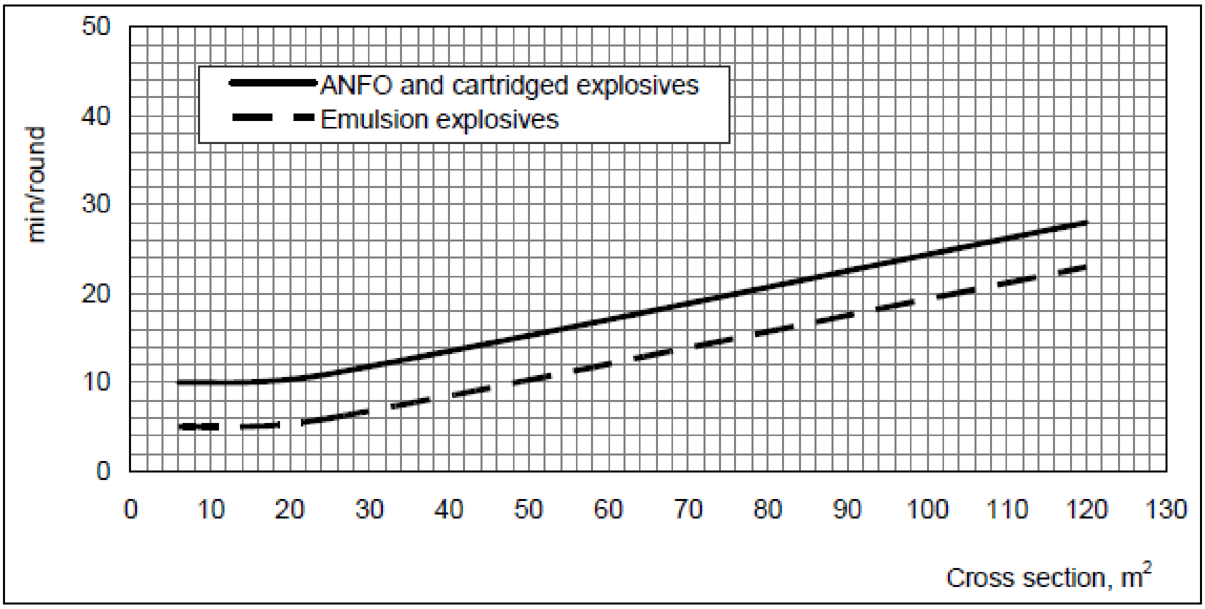

Figure 48 Time-determinant ventilation break as a function of tunnel cross section area. (Zare, 2007a)

12.3 Mucking

While considering the time used for mucking, one should consider that the actual solid cubic meter (asm^3) and planned solid cubic meter (psm^3) will differ. The ratio between asm^3 and psm^3 is called the overbreak factor. The overbreak factor depends on the tunnel cross-section and tunnelling method. The main cause of the difference is the look-out angle and drilling deviation.

The overbreak factor, f_0, is cross section dependent and does not consider niches, as it is considered separately in the Thesis. According to Figure 49 presenting the factor of overbreak f_0, as a function of tunnel cross section area and tunnelling method, f_0=1.148.

While calculating the time for loading, one should first calculate the normalized gross loading capacity, QI. The main assumption is that the equipment is fully utilized. The loading is continuous, however, the model includes the "necessary brakes for dumper change, trimming of the pile and levelling of the floor." When it comes to longer tunnels, it is the transport capacity that influences the mucking capacity. It was assumed that for Stavsjøfjelltunnelen, 80-100% of the loading capacity can be utilized.

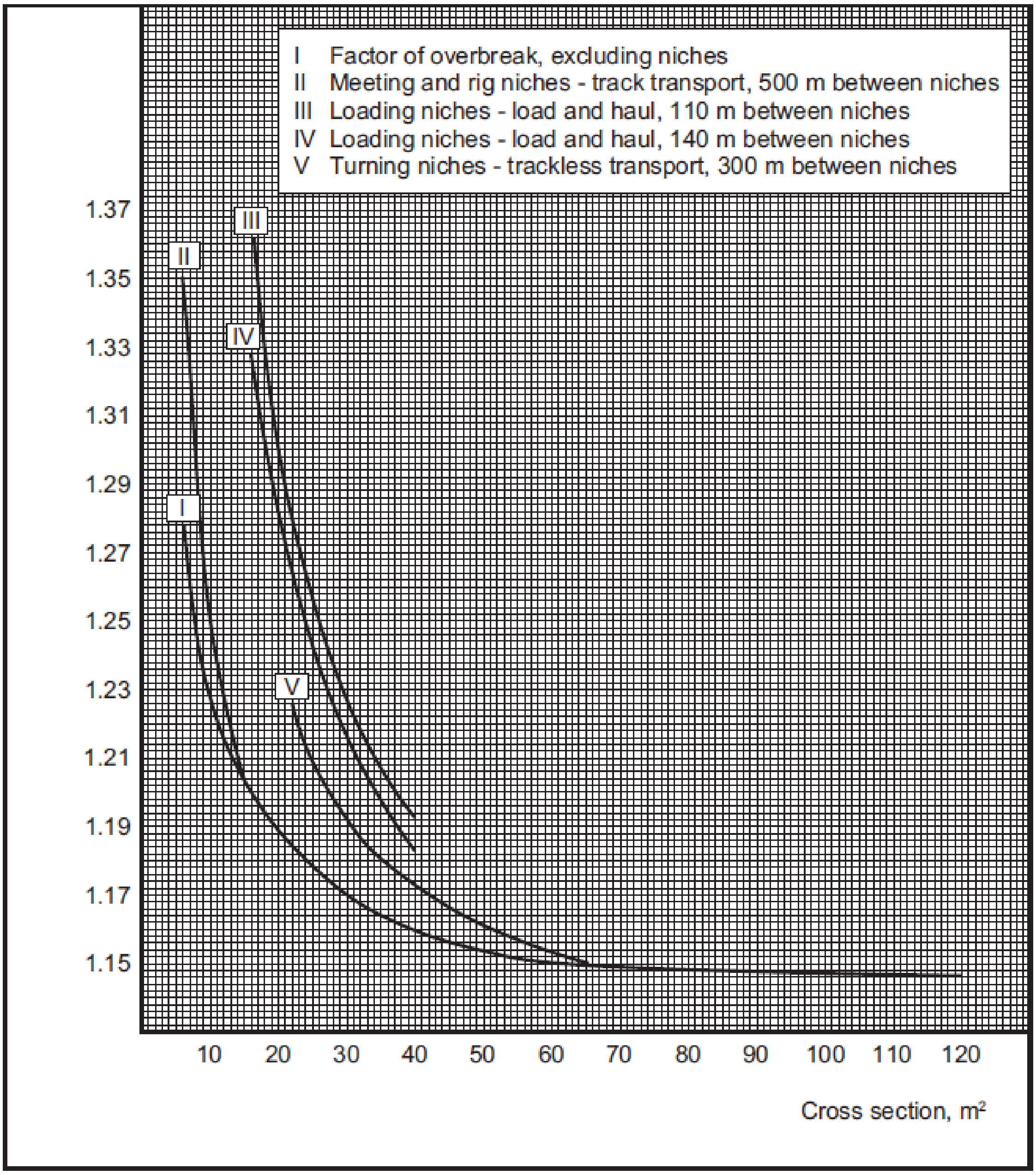

Figure 49 Factor of overbreak f_0 (Zare, 2007a)

In order to calculate the total loading time the model uses the following equations:

The volume of blasted rock per round:

$$V_r = A_s * l_h * p_r * f_0 \qquad (16)$$

Where:

A_s= tunnel cross section area

l_h= drilled length

p_r= pull

f_0= overbreak factor

Loading Time:

$$T_k = \frac{V_r}{Q_l} * 60 \qquad (17)$$

V_r= volume of blasted rock per round

Q_l= normalized gross loading capacity

Q_l= $Q_g * f_{ul}$

Q_g= gross loading capacity

f_{ul}= utilization factor

The model also considers the rig time, T_{rl}. The time depends on the transport type and is a function of the tunnel cross section area and tunnelling method. See Figure 50.

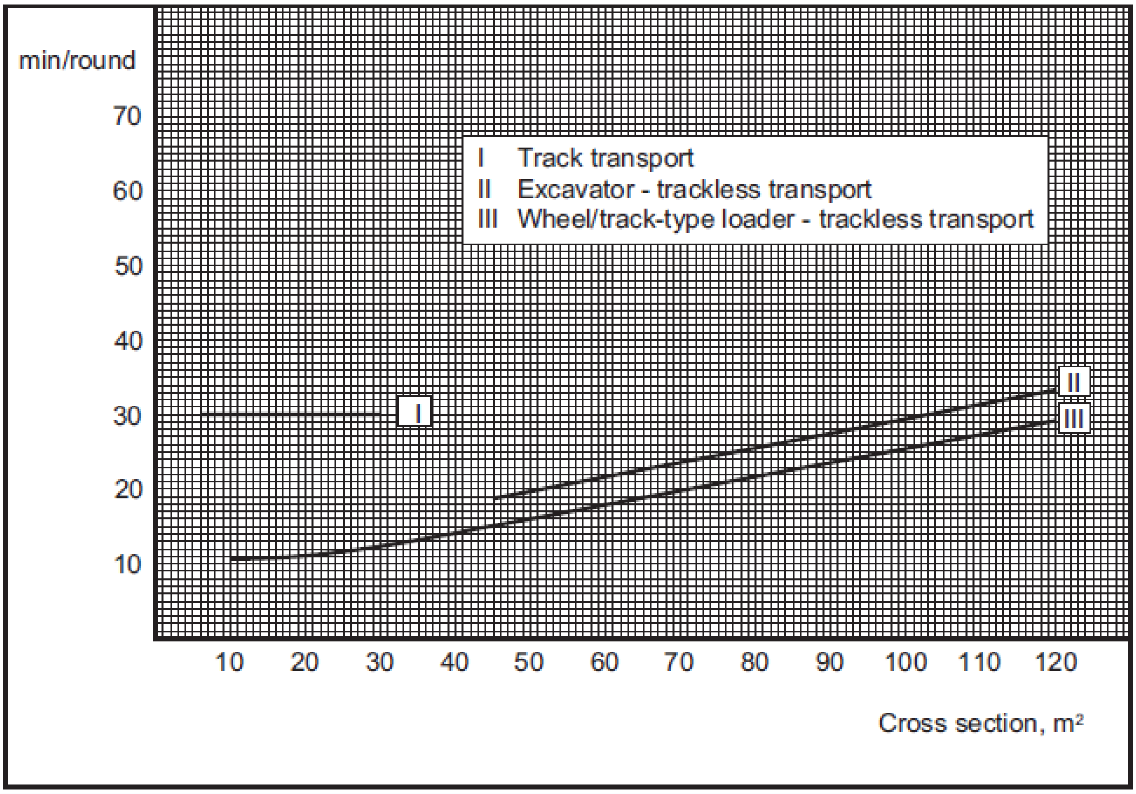

Figure 50 Rig time for loading and hauling, T_{rl} (Zare, 2007a)

12.4 Scaling

After each blast, it is common to use scaling hammers for tunnels with a profile area > 20 m^2. The aim of the scaling is to remove loose blocks and stones from tunnel surface. The time used for the scaling is dependent on the blastability, scaling method and tunnel cross section, and can be read from Figure 51.

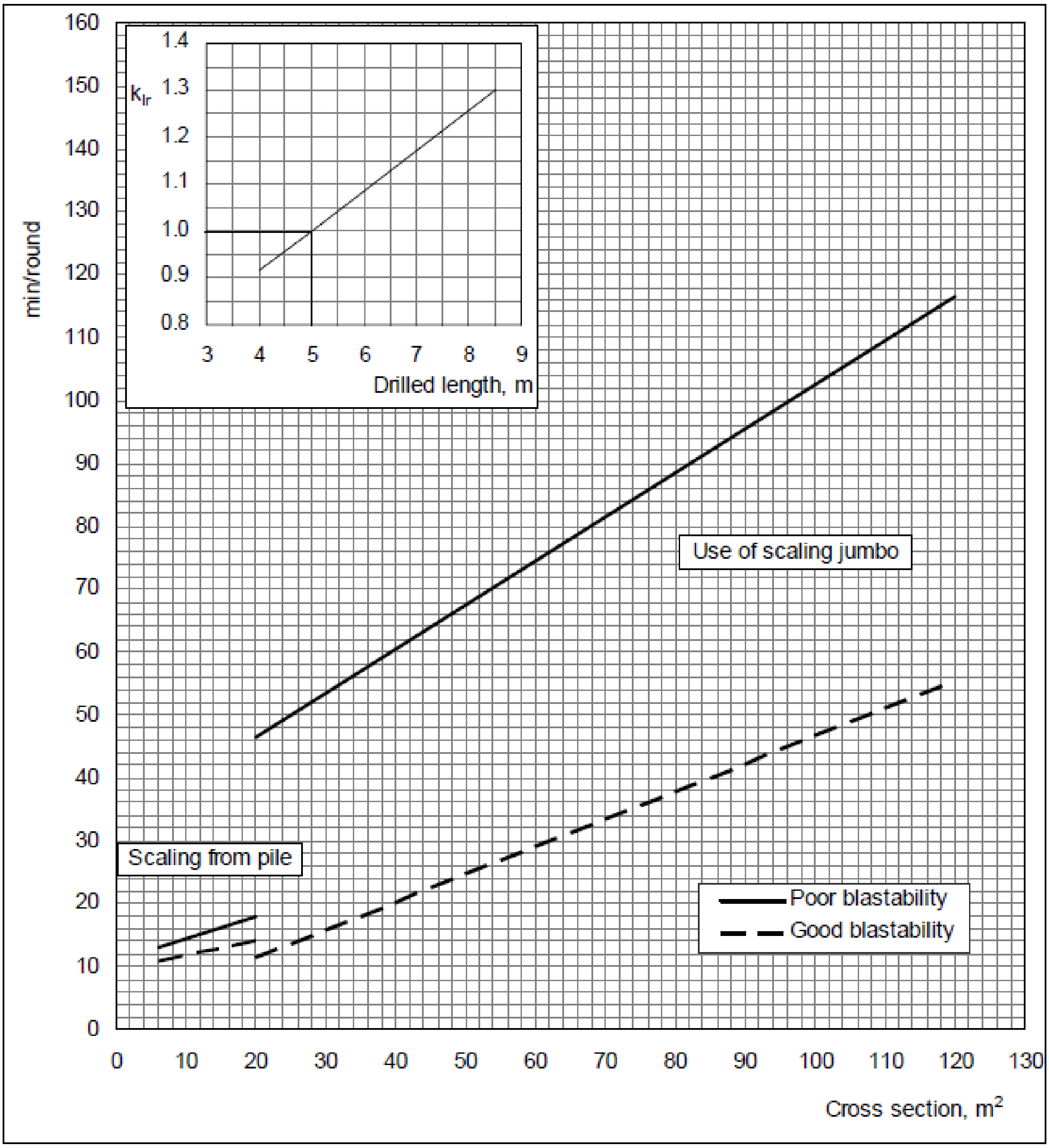

Figure 51 Scaling time per round (Zare, 2007a)

12.5 Tunsim simulation input data and results

The Tunsim algorithm uses the two NTNU reports: *2A-05 Drill and Blast Tunnelling Blast Design, 2B-05 Drill and Blast Tunnelling Advance Rate*. Basic formulas and figures used by the program were presented earlier in the chapter. This section will present the input data for the program and the results of the tunnel construction performance for the tunnel and cross passages. Factors like round length, blastability and tunnel cross sectional area were taken into consideration. Tunsim does not include tunnel support in the performance predictability. The time consumption of tunnel support installation is presented in the next chapter.

The input data for the Tunsim simulation, as well as the corresponding results, will be presented in two separate sections of this chapter. This separation is a result of the special need for niche input data. The minimum input drill length in the program is 2,8m. Since the results for 2,3m as well as 1,3m were needed, the values were interpolated with high accuracy over R^2=0,99.

12.5.1 Input data and results for the main tunnel T10,5

12.5.1.1 Main tunnel

The input data for T10,5 was presented in the Table 16. The results of the analysis were presented in Table 17 and Table 18 below. The results present time consumption per each activity, as well as the gross round time consumption. Since the round lengths of 2,3m and 1,3m were not analyzed in the Tunsim simulation, there are no results for separate activities corresponding to these round lengths. However, the interpolated gross round time consumption has been presented.

Table 16 Tunsim input data for the Main tunnel T10,5

INPUT POOR/MEDIUM					
		Main tunnel			
	m	5.3	4.3	3.3	2.8
Area	m^2	78.83			
Length	m				
Drill length	m	5.3	4.3	3.3	2.8
Charged holes					
Drill diameter	mm	48			
Rim diameter	mm	102			
Blastability	poor/medium				
DRI	medium				
VHNR	550				
SKILL	high				
Excavation	trackless				
Rig	COP 1838				
hammers	3				
Explosive	emulsion				
Loading	Volvo L 330E				
Transport	Dump track				
Weeks/year	46				
h/day	20				
Shift amount	2				

Table 17 Tunsim analysis results - Medium Blastability for the Main tunnel T10,5

Tunsim analysis results - Medium Blastability T10,5							
		Main tunnel					
						interpolated	
Drill length	m	5,3	4,3	3,3	2,8	2,3	1,3
Drilling holes	min	153	126,1	100,3	88,7		
Charging	min	53,2	48,7	44,3	42,1		
Ventilation	min	13,1					
Loading	min	129,1	109,2	89,4	50		
Scaling	min	64,1	58,5	53,4	50,7		
Net round	min	460,8	400,7	341,9	314		
Net week	min	62,8	58,6	52,7	48,7		
Gross round	min	481,8	419	357,5	328,3		
Gross week	m	60,1	56	50,4	46,6		
Gross round	h	8,03	6,98	5,96	5,47	4,95	3,92

Table 18 Tunsim analysis results - Poor Blastability for the Main tunnel T10,5

Tunsim analysis results - Poor Blastability T10,5							
		Main tunnel					
						interpolated	
Drill length	m	5,3	4,3	3,3	2,8	2,3	1,3
Drilling holes	min	179	148	118,1	104,5		
Charging	min	59,8	54,8	49,8	47,3		
Ventilation	min	13,1					
Loading	min	129,1	109,2	89,4	79,4		
Scaling	min	89,9	82,4	74,9	71,1		
Net round	min	522,8	455,5	389,3	357,7		
Net week	min	55,3	51,5	46,3	42,7		
Gross round	min	546,7	476,2	407	374		
Gross week	m	52,9	49,3	44,3	40,9		
Gross round	h	9,11	7,94	6,78	6,23	5,64	4,49

12.5.1.2 Cross passages

The excavation of the cross passages was assumed to be done at the same time as the main tunnel excavation. The drilling process will take place when the drill rig is not needed in the main tunnel. That can be while scaling or injecting. The blasting of the cross passages can take place at the same time as the face blasting. The same approach could be applied to other processes in the cross passages. As a result, the main tunnel excavation performance would not be disturbed.

When it comes to the vibration restriction, the experience from the Gevingåsen project shows that the first round could be 3m long. The rest of the blasts are recommended to be 2m long. The last blast should be 1m long (Fjærtoft, 2013). According to the (ReinertsenAS, 2015b), there are seven planned cross passages in the tunnel. According to Figure 53 three cross passages are located in the A/B support class areas, one in C, and three in D. The average value of the C support class was assumed for calculation purposes. However the T4 profile was assumed in the Chapter 3.6 - New tunnel tube, it was decided to estimate the excavation performance assuming the tunnel profile of T5,5 as for escape tunnels. The cross section area would be A=42,59 m^2 (StatensVegvesen, 2014). According to the (ReinertsenAS, 2015b), the average length of the cross passage is around 17m.

Six out of the seven cross passages are located in the areas without round length restrictions (13690, 14190, 14440, 14690, 14940, 15140). The excavation process would then be 3m round + 6x2m rounds + 2x1m round.

According to the Tunsim analysis, 3m rounds could be excavated in 254 min. The values for the 2m rounds and 1m rounds were interpolated. A 2m long round could be excavated in 200 min, and a 1m long round could be excavated in 146 min.

The total excavation time consumption per cross passage T_{CP1}, in the area without vibration restrictions, would be:

$$T_{CP1} = 1x254min + 6x200min + 2x146min = 1746min$$

$$T_{CP1} = 29h\ 7min$$

The cross section on PEG 14940 is located under the residential area where the maximum round length is 1m. The recommended excavation process would be then 17x1m. The total excavation time consumption per cross passage T_{CP2}, in the area width vibration restrictions, would be:

$$T_{CP2} = 17\ x\ 146min = 2482\ min$$

$$T_{CP2} = 41h\ 22min$$

12.5.2 Input data and results for niche T13,5

Niche sizes and amounts are decided based on the tunnel class. As mentioned earlier, the tunnel class is E. For tunnel class E, a niche should be located every 500m and have a profile of T12.5 (StatensVegvesen, 2014). However, it was decided earlier that the tunnel tube will be T10.5, which corresponds to a T13.5 profile for the niche. In order to simplify the calculation, it was decided to use the average cross sectional area along the sections of the niche. Figure 52 presents the niche dimensions.

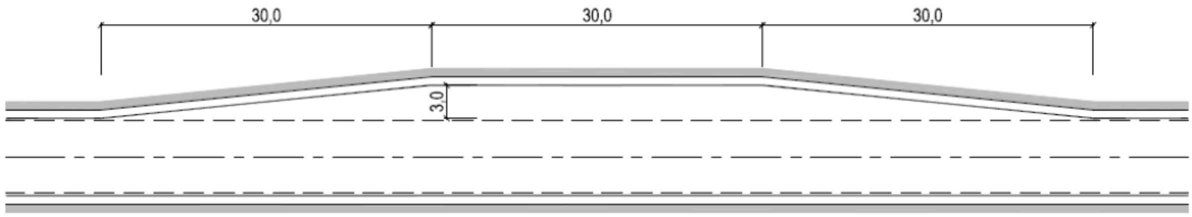

Figure 52 Niche T13,5 (StatensVegvesen, 2014)

Total length of the niche is 90m. First 30m and last 30m have a varying profile. In order to simplify the calculation and estimate the niche excavation performance, the average profile area was calculated.

$$A_{T10.5} = 78.83m^2$$

$$A_{T13.5} = 105.66m^2$$

$$A_{average} = (A_{T10.5} + 2 * A_{T13.5}) * \frac{1}{3} = 96{,}72m^2$$

The tunnel excavation performance estimated in the Tunsim program for the niche in the poor and medium blastability areas is presented by Table 20 and Table 19 respectively.

Table 19 Niche excavation performance - Medium Blastability

Tunsim analysis results - Medium Blastability T13,5							
Drilled length	[m]	5,3	4,3	3,3	2,8	2,3	1,3
Gross round	[min]	550,60	463,40	391,60	359,50	330,13	278,44
	[h]	9,18	7,72	6,53	5,99	5,50	4,64

Table 20 Niche excavation performance - Poor Blastability

Tunsim analysis results - Poor Blastability T13,5							
Drilled length	[m]	5,3	4,3	3,3	2,8	2,3	1,3
Gross round	[min]	599,50	505,80	429,00	395,70	363,61	307,94
	[h]	9,99	8,43	7,15	6,60	6,06	5,13

13. Rock support

The calculations included in the previous chapters do not take the time consumption used for rock support into consideration. The rock support installation will change the excavation performance and has to be considered. The necessary rock support was chosen by following Handbook 021 from the Norwegian Public Road Administration (NPRA), as well as Publication no. 19 from NFF. The time consumption was estimated by following the Project rapport 2F-99 Tunneldrift – Enhetstidsystem for driving, sikring og innredning. The analysis was divided by rock support type into 4 parts: Bolting, Shotcreting, Spilling and Shotcrete girders. For each part, an example of the calculation is presented. At the end of each part, combined results are presented. Additionally, the time consumption analysis of Probe and Core drilling was performed, however, not considered in total consumption time.

In order to select the required rock support, one should establish the geological and rock engineering condition. The geological condition has been described in Chapter 4. In Norway, rock support estimation is mostly based on the NGI model, Q-model. The Q-value determines the rock mass class. NFF propose the support class division from I – VI as shown in Figure 54.

The percentage distribution of the rock mass quality is shown in Table 21. The support class distribution along the tunnel is presented in Figure 53. In the Figure symbols: A/B, C, D, E, F were converted into numbers correspondingly: 1, 2, 3, 4, 5. The figure shows that rock support class E might require a shorter round length as well as spilling and shotcrete girders installation. The additional rock support would be agreed with the client on the construction site, however, such scenario is analyzed in this Thesis.

Table 21 Percentage distribution of rock mass quality in the Stavsjøfjelltunnelen

Distribution	**A/B(1)**	**C(2)**	**D(3)**	**E(4)**	**F(5)**
%	24,1	22,1	42,5	8,7	2,6
[m]	415	380	732	150	45

In the case of very poor rock mass, as well as extremely poor rock mass, the NPRA inspector often orders a widened tunnel profile (+50cm) in order to apply spilling bolts. Due to this approach, the circuit of class E, where the shotcrete girders support is ordered, and whole class F, was set to be 24.9m. Based on the mathematical calculation the circuit is around 24.4 m. However, poor geological condition might cause unforeseen rockfalls and larger than expected profile. In order to be on the safety site, the circuit was estimated to be 24.9 m. The estimation was done in cooperation with Iwona Rumbuc, Quality Head Engineer in Marti Norge. (Rumbuc, 2016a)

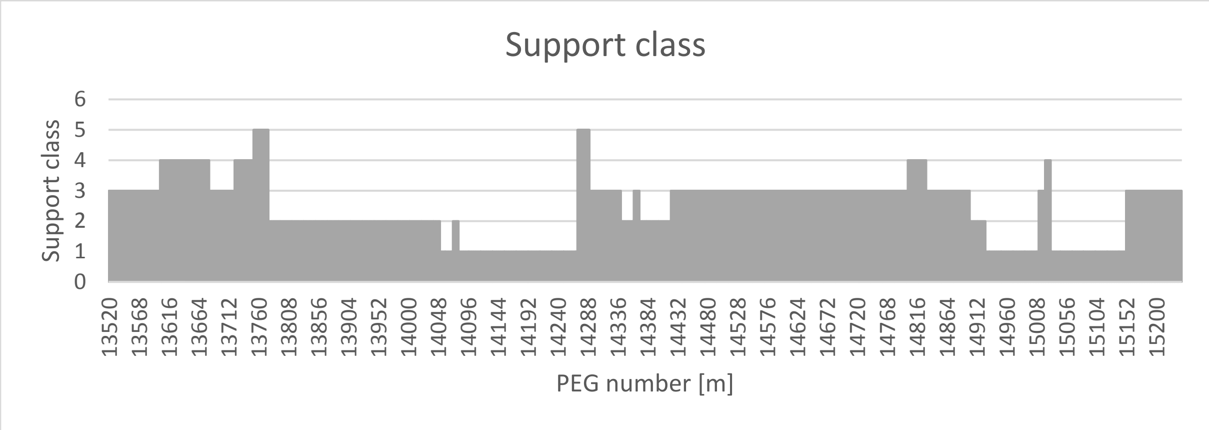

Figure 53 Support class distribution along the tunnel

Rock mass class	Geology Q-value	Rock support class Permanent support
A/B	Competent rock Average joint spacing > 1m Q = 10 - 100	**Support class I** - Occasional bolting. Sprayed concrete B35 E700, 80 mm, roof and walls to 2m above invert
C	Moderately competent rock Average joint spacing 0.3 – 1 m Q = 4 – 10	**Support class II** - Systematic bolting (c/c 2 m), end anchored, pre stressed and grouted. Sprayed concrete B35 E700, 80 mm on roof and walls
D	Densely cracked or chisty rock mass, Average joint spacing < 0.3 m. Q = 1 – 4	**Support class III** - Sprayed concrete B35 E1000, ≥ 100 mm - Systematic bolting (c/c 1,5 m), end anchored, grouting (timing to be considered)
E	Very poor rock mass Q = 0.1 – 1	**Support class IV** -Sprayed concrete B35 E1000, 150 mm - Systematic bolting (c/c 1,5 m), end anchored, grouting - If Q < 0,2spiling ø25 mm, c/c 300 mm or less - If Q < 0,2 reinforced sprayed concrete ribs E30/6 ø20 mm, c/c 2 – 3 m, -Systematic locking of the ribs by bolts, L= 3 – 4 m - Cast invert to be considered
F	Extremely poor rock mass Q = 0.01 – 0,1	**Support class V** - Spiling, c/c 200 – 300 mm, ø32 mm bolts or "self boring anchors" - Sprayed concrete B35 E1000, 150 – 250 mm - Systematic bolting, c/c 1,0 – 1,5 m, grouted - Reinforced concrete ribs D60/6+4, ø20 mm, c/c 1,5 – 2 Systematic bolting of the ribs c. 1,0 m, L 3 – 6 m - Reinforced concrete invert
G	Exceptional poor rock mass, basically loose material. Q < 0.01	**Sikringsklasse VI** - Special design required. Not suitable for blasting.

Figure 54 Rock mass class and Q-value (Berggren, 2014)

13.1 Bolting

a) Required support

The rock bolting will be performed by following a systematic net pattern. According to Figure 54, in support class I, occasional bolting should be installed. However, personal experience of the author shows that the client often agrees to implement 2.5m x 2.5m net. Such approach will be used while estimating the time consumption for this project.

An example of the calculation will be presented on support class C. The results of all the calculations are presented in Table 22 and Table 23.

Support class C with c/c=2,0m:

Circuit:	$\frac{22,71}{2,0m} - 1 \ = 10,38 \, bolts/row$
Total:	$10,4 * 0,5 \ = 5,18 \, bolts/m$

b) Time consumption

The time consumption includes the drilling of the bolt holes, as well as installation of the polyester bolts after drilling. The bolt length, in the case of support class C, is 3m. An example of the calculation will be presented for support class C. The results of all the calculations are presented in Table 22 and Table 23.

Time per meter (diagram):	$0,28 \, h/m$
Correction factor for 3m length bolts:	1,0
Correction factor for time consumption:	1,0
Time consumption for 1m of the tunnel:	$0,28 \, \frac{h}{m} * 1,0 * 1,0 = 0,28 \frac{h}{m}$

Table 22 Required bolt support and time consumption in the Main tunnel

BOLTING									
Required support and time consumption									
SK	length [m]	circuit [m]	net [m]	bolt/row	row/m	bolt/m	length factor	h/m	h/m corrected
A/B	3	22.71	2.5	8.1	0.40	3.23	1	0.18	0.18
C	3	22.71	2	10.4	0.50	5.18	1	0.28	0.28
D	3	22.71	1.5	14.1	0.67	9.43	1	0.53	0.53
E	4	22.71	1.5	14.1	0.67	9.43	1	0.53	0.53
E	4	24.9	1.5	15.6	0.67	10.40	1.3	0.58	0.75

Table 23 Required bolt support and time consumption in the Niche

BOLTING									
Required support and time consumption									
SK	length [m]	circuit [m]	net [m]	bolt/row	row/m	bolt/m	length factor	h/m	h/m corrected
A/B	3	24.9	2.5	9.0	0.40	3.58	1	0.21	0.21
C	3	24.9	2	11.5	0.50	5.73	1	0.33	0.33
D	3	24.9	1.5	15.6	0.67	10.40	1	0.58	0.58
E	4	24.9	1.5	15.6	0.67	10.40	1	0.58	0.58
E	4	28.2	1.5	17.8	0.67	11.87	1.3	0.66	0.86

13.2 Spilling bolts

a) Required support

Spilling bolt support will be required in E and F support classes. According to Figure 54, spilling bolt support should be applied in SK E when the Q-value drops below Q = 0,2, and in SK F. An example of the calculation will be presented on support class E. The results of all the calculations are presented in Table 24 and Table 25.

Support class E with c/c=300 mm:

Circuit: $\frac{24{,}9}{0{,}3\ m} - 1 \ = 79{,}67\ bolts/row$

Bolts will have a length of 8 m overlap. As the result the amount of bolts per meter is:

$$79{,}67 \frac{bolt}{row} * \frac{1}{3m} = 26{,}56 \frac{bolts}{1m\ length}$$

b) Time consumption

The time consumption includes the drilling of the bolt holes, as well as the installation of the 8m bolts after drilling. The bolt length, in the case of support class E, is 8m. Very poor rock mass requires 50 cm profile widening what results in 24,9m circuit. An example of the calculation will be presented on support class E. The results of all the calculations are presented in Table 24 and Table 25.

Time per round (diagram): $3{,}37\ h/round$

Correction factor for 8m length bolts: 1,0

Correction factor for time consumption: 1,0

Total time consumption: $\frac{3{,}37\frac{h}{round}}{3m} = 1{,}12h/m$

Table 24 Required spilling bolt support and installation time consumption in the Main tunnel

Spilling								
Required support							time consumption	
SK	circuit [m]	distance between rows	distance between bolts	bolts /row	bolt /meter	bolt length factor	h /round	h/m
E	24.9	3m	0.3m	79.67	26.56	1	3.37	1.12
F	24.9	3m	0.25m	95.60	31.87	1	3.98	1.33

Table 25 Required spilling bolt support and installation time consumption in the Niche

Spilling								
Required support							time consumption	
SK	circuit [m]	distance between rows	distance between bolts	bolts /row	bolt /meter	bolt length factor	h /round	h/m
E	28.2	3m	0.3m	90.67	30.22	1.00	3.79	1.26
F	28.2	3m	0.25m	108.8	36.27	1.00	4.48	1.49

13.3 Shotcrete

a) Required support

Shotcrete support will be required in all support classes. In the case of very poor rock mass, and extremely poor rock mass, the NPRA inspector often orders a widened tunnel profile (+50cm) in order to apply shotcrete girders. Due to this approach, the circuit of class E where the shotcrete girders support is ordered, and whole class F, was set to be 24.9 m. The recommended thickness of the shotcrete with respect to the support classes was presented on Figure 54:

A/B: 80 mm

C: 80 mm

D: 100 mm

E: 150 mm

F: 250 mm

An example of the calculation will be presented on support class C. The results of all the calculations are presented in Table 26 and Table 27.

Support class C with 80 mm of shotcrete:

Amount of the shotcrete per meter: $22{,}71m * 1m * 0{,}08m = 1.82\ m^3/m$

b) Time consumption

Time consumption, in the case of shotcreting, depends on the number of tunnel meters that are covered by shotcrete at the same time. An example calculation will be presented on support class C where it was assumed that 15m of the tunnel can be shotcreated during one shotcrete spraying round. Additionally, it is recommended to apply a time consumption factor considering the fact that shotcrete spraying can be performed during drilling or other activities such as injection. All the results will be presented in the Table 26 and Table 27.

Correction factor for length:	0,87
Shotcrete performance (diagram):	0,14
Shotcrete performance:	$0{,}87 * 0{,}14 \frac{h}{m^3} = 0{,}12 \frac{h}{m^3}$
Amount of shotcrete per 15m:	$1{,}82\ m^3/m * 15m = 27{,}3\ m^3$
Time consumption per 15m:	$27{,}3\ m^3 * 0{,}12 \frac{h}{m^3} = 3{,}32\ h$
Time consumption factor:	0,5
Hour consumption per meter:	0,11
Total time consumption per 380m:	$\frac{3.32h}{15m} * 0{,}5 = 0.11\ h/m$

Table 26 Time consumption of Shotcrete installation - Main tunnel

Shotcrete										
Required support and time consumption										
SK	thickness [m]	circuit [m]	volume per meter [m^3]	meters per spraying	length factor	h/m3	h/m3 factor	h per section	cons. factor	h/m
A/B	0.08	22.71	1.82	20	0.75	0.14	0.11	3.82	0.00	0.00
C	0.08	22.71	1.82	15	0.87	0.14	0.12	3.32	0.50	0.11
D	0.1	22.71	2.27	10	1	0.12	0.12	2.73	0.80	0.22
E	0.15	22.71	3.41	5	2	0.1	0.20	3.41	1.00	0.68
E	0.15	24.9	3.74	3	3	0.08	0.24	2.69	1.00	0.90
F	0.2	24.9	4.98	3	3	0.08	0.24	3.59	1.00	1.20

Table 27 Time consumption of Shotcrete installation - Niche

Shotcrete										
Required support and time consumption										
SK	thickness [m]	circuit [m]	volume per meter [m^3]	meters per spraying	length factor	h/m3	h/m3 factor	h per section	cons. factor	h/m
A/B	0.08	24.9	1.99	20	0.75	0.13	0.10	3.88	0.00	0.00
C	0.08	24.9	1.99	15	0.87	0.13	0.11	3.38	0.50	0.11
D	0.1	24.9	2.49	10	1	0.125	0.13	3.11	0.80	0.25
E	0.15	24.9	3.74	5	2	0.09	0.18	3.36	1.00	0.67
E	0.15	28.2	4.23	3	3	0.07	0.21	2.66	1.00	0.89
F	0.2	28.2	5.64	3	3	0.07	0.21	3.55	1.00	1.18

13.4 Shotcrete girders

a) Required support

The analysis of shotcrete girders installation was not offered in the report 2F-99. In order to estimate the required support, as well as the installation time consumption, personal experience of the author and the results of a discussion with Tunnel Production Manager Martin Adamkovic from Marti IAV Solbakk was applied (Adamkovic, 2016).

The recommended shotcrete girders support for rock class E and F is:

Rock support class E:

- 1 shotcrete girder per 2,5m = 0,4 girder / 1 m

Rock support class F:

- 1 shotcrete girder per 2,0m = 0,5 girder / 1 m

The example of the calculation will be presented on support class E. The results of all the calculations are presented in Table 28 and at the end of this chapter.

b) Time consumption

The time consumption includes the drilling of the bolt holes and the installation of the 4m SN bolts for the shotcrete girders, as well as the reinforcement installation and shotcreting. In order to install the shotcrete girders, profile widening has to be used. The circuit of the widened profile is 24,9m. The example of the calculation will be presented on support class E. The results of all the calculations are presented in Table 28 and Table 29.

Time per one girder: $0{,}16\frac{h}{m} * 24{,}9m = 3.98\ \ h/girder$

Correction factor for time consumption: 1,0

The number of girders per meter: $\frac{0{,}4 \text{ girder}}{\text{m}}$

Time consumption per meter: $\frac{0{,}4girder}{m} * 3{,}98\frac{h}{girder} = 1{,}59\frac{h}{m}$

Table 28 Time consumption of Shotcrete girders installation - Main tunnel

Shotcrete girders						
Required support and time consumption						
SK	circuit [m]	meters between girders	girder/meter	h/ girder meter	h/girder	h/m
E	24.9	2.5	0.4	0.16	3.98	1.59
F	24.9	2	0.5	0.24	5.98	2.99

Table 29 Time consumption of Shotcrete girders installation – Niche

Shotcrete girders						
Required support						
SK	circuit [m]	meters between girders	girder/meter	h/ girder meter	h/girder	h/m
E	28.2	2.5	0.4	0.16	4.51	1.80
F	28.2	2	0.5	0.24	6.77	3.38

13.5 Probe drilling

a) Required activities

In order to check the rock condition and characteristics in front of the tunnel face, probe hole drilling can be applied. The probe drilling has to be performed with 10 meters overlap. The hole length is usually 25m or 30m. While drilling the MWD (Measure While Drilling) data can be taken from the drill rigs. MWD data includes all drilling parameters such as drilling direction, drilling performance, drilled length etc. While using SANDVIK machines, the iSURE program can be used to analyse the data and see the data on 3D models.

b) Time consumption

The time consumption will be equal for E and F rock class:

$$\frac{hole\ length * amount\ of\ hoels}{hole\ length - overlap} = \frac{30m * 4}{30m - 20m} = \frac{6\ drilled\ m}{m}$$

Time consumption for drilling 1m is 0,0172 h/drilled meter:

$$0{,}0172\,\frac{h}{drilled\ meter} * \frac{6\ drilled\ m}{m} = 0{,}103\,\frac{h}{m}$$

13.6 Core drilling

a) Required activities

Core drilling will be required in the poorest geological condition as, by taking the real sample, it can give the clearest picture of actual geological condition in front of the face. The core drilling can drill up to 50m. In this project, the F geological condition zone was estimated to be 45m.

b) Time consumption

Time consumption per drilled meter: 0,74 h/drilled meter

13.7 Injection

a) Required activities

Injection is performed in order to prevent leakage into the tunnel. The leakage depends on the rock characteristics, especially the crack amount and its filling. According to available information the injection class of the rock mass has been found to be class C (ReinertsenAS, 2015b). This kind of the rock mass has low hydraulic conductivity and is difficult to inject. Since the injection class is C it is recommended to make the probe drilling in the weakness zone areas in order to decide whether the injection is needed or not.

According to the available information there might be a need to implement systematic injection between Modalen (profile no. 14550-15050) and Svartdalsdammen (profile no. 14050-14300)(ReinertsenAS, 2015b). In order to estimate the duration of each injection cycle interview with Iwona Rumbuc, the quality engineer from Marti Norge has been made (Rumbuc, 2016b).

Standard injection could be 25 meters long and comprise of 25 holes (4 probe holes + 21 additional injection holes). The injection should have minimum 8 meters overlap what means that the injection should take place every 3 blasts of 5 meters.

b) Time consumption

Drilling for the injection could take 6 hours in average. Injection process could take 8 hours in average.

Total amount of hours spend on the injection:	$14h * \frac{750m}{15m} = 700h$
Total amount of working days spend on the injection:	$\frac{700h}{20h} = 35\ days$
Total amount of working weeks spent on the injection:	$\frac{35}{7} = 5\ weeks$

The calculation shows that total project duration would be 5 working weeks longer.

14. Tunnel excavation performance – model

There is a need for closer estimation models for time scheduling in Drill and Blast. (Yangkyun Kim, 2009). It has been noticed that there are many factors influencing the construction time of D&B tunnels. Stavsjøfjelltunnelen passes through an area where several factors have to be taken into consideration. The excavation will be performed through changing geological condition and will be restricted by the vibration limits due to the existing tunnel, as well as the residential areas in close proximity to the tunnel. Additionally, tunnel blasting has been restricted by time windows for blasting. In order to deal with all those factors, a model for the Stavsjøfjelltunnelen project has been created. The Model estimates the total duration of the project, together with the performance distribution along the tunnel.

14.1 Model description

14.1.1 Round duration influencing factors

The model combines two existing NTNU models presented in previous chapters. The first model used for estimating the rock support time consumption 2F-99 and the second model used for excavation time estimation without the rock support 2B-05. The program look up for the proper round duration based on the round length, blastability, geological condition, and the changing tunnel profile when it comes to the niches.

a) Rock support classes

In case of the Stavsjøfjelltunnelen project, six different rock support classes have been assumed. The rock support class varies from A/B to F. Since the Figure 54 considers two different extent of the rock support in case of rock class E, two options were considered in the Thesis. The first option (E*) assumes no need for spilling and shotcrete girders. While the second option (E**) assumes the spilling bolt and shotcrete girders support Table 30 and Table 31.

b) Vibration limits

The round length is usually dependent on the geological condition, however, in the analysed case, the round length is also vibration limit dependent. Due to the vibration limits, the drilled length was assumed to be from 1.3m to 5.3m with 1m steps in between, resulting in five different drilled lengths.

c) Blastability

In case of the Stavsjøfjelltunnelen, it was assumed that the blastability is geological class dependent. The standard blastability has been set to medium, however, in case of rock class E and F the blastability has been assumed to be poor.

d) Profile change

Along the tunnel there are three niches located 500m from each other. Standard tunnel profile is T10.5, and T13.5 for the niches sections. Big profile sections have been analysed with respect to points a), b), and c). Table 30 shows the excavation and tunnel support time consumption in hours based on Support Class (SK), round length and blastability for the Main tunnel - T10,5. Table 31 shows the excavation and tunnel support time consumption in hours based on Support Class (SK), round length and blastability for the Niche - T13,5.

e) Round length

Despite the fact that the drilled lengths are 5,3m; 4,3m; 3,3m; 2,3m and 1,3m during the performance estimation the assumption of 5m, 4m, 3m, 2m and 1m round length was done. This means that the pull-out would be equal from around 94% in case of 5,3m drilled length down to 77% in case of 1,3m drilled length. The precise estimation is not possible to achieve, however, such assumption should place the estimation on the safety side.

Table 30 Excavation and tunnel support time consumption - Main tunnel T10,5

MEDIUM BLASTABILITY						POOR BLASTABILITY					
SK	5,3m	4,3m	3,3m	2,3m	1,3m	SK	5,3m	4,3m	3,3m	2,3m	1,3m
A/B	8.93	7.70	6.50	5.31	3.79	A/B	10.01	8.66	7.32	6.59	5.82
C	9.98	8.55	7.13	5.73	4.00	C	11.06	9.50	7.96	7.01	6.03
D	11.77	9.98	8.20	6.44	4.36	D	12.85	10.93	9.03	7.73	6.39
E*	14.09	11.83	9.59	7.37	4.82	E*	15.17	12.78	10.42	8.66	6.85
E**	29.87	24.46	19.06	13.68	7.98	E**	30.95	25.41	19.89	14.97	10.01
F	35.57	29.02	22.48	15.96	9.12	F	36.66	29.97	23.31	16.66	9.65

E* - Support class E without the need for spilling bolts and shotcrete girders installation

E** - Support class E with the need for spilling bolts and shotcrete girders installation

Table 31 Excavation and tunnel support time consumption - Niche T13,5

MEDIUM BLASTABILITY						POOR BLASTABILITY					
SK	5,3m	4,3m	3,3m	2,3m	1,3m	SK	5,3m	4,3m	3,3m	2,3m	1,3m
A/B	10.23	8.56	7.16	5.92	4.62	A/B	11.04	9.27	7.78	7.02	6.27
C	11.39	9.49	7.85	6.39	4.85	C	12.20	10.20	8.48	7.48	6.50
D	13.32	11.04	9.01	7.16	5.24	D	14.14	11.75	9.64	8.25	6.89
E*	15.44	12.73	10.28	8.01	5.66	E*	16.25	13.44	10.91	9.10	7.31
E**	33.25	26.98	20.97	15.13	9.22	E**	34.06	27.69	21.59	16.22	10.87
F	39.48	31.97	24.71	17.62	10.47	F	40.3	32.67	25.33	18.18	10.94

E* - Support class E without the need for spilling bolts and shotcrete girders installation

E** - Support class E with the need for spilling bolts and shotcrete girders installation

14.1.2 Time windows for blasting and construction work

Based on a recommendation from Reinertsen AS (Chapter 3.2), it was assumed to allow blasting in the time windows presented in the Table 2. Additionally, the construction work is assumed to be performed 20 hours per day (assumed from 4:00 to 24:00) excluding Saturday and Sunday. This gives a production time of 100 h/week. In the model, it was assumed that the blasting moment is the beginning of the process. The model automatically locates the blast in the closest time window allowed by the assumptions.

14.1.3 Error check

In order to check the Excel program for errors, the blast execution time was simulated to take place in several moments of the week and day.

- 4:00
- After 5:00, before 6:30
- 6:30
- After 6:30, before 10:00
- 10:00
- After 10:00, before 14:00
- 14:00
- After 14:00, before 19:30
- 19:30
- After 19:30, before 23:30
- 23:30
- After 23:30, before 24:00
- 24:00
- After 24:00, before 1:00
- 1:00
- 3:00

The blasts were checked during the week days as well as on the weekend. The analysis did not result in any mistakes and might indicate that the model does not contain a serious mistake. The maximum round duration analysed by the model without mistakes is 40h, which is more than enough when it comes to the Stavsjøfjelltunnelen analysis. Despite the fact that no serious program mistake have been found, one should notice that the model and the program is a prototype that requires further development and testing.

14.1.4 Limitations of the Model

The maximum round duration that can be analysed in the program is 40h. A round duration over 40h is not likely to happen, however, further development in that area could improve the model.

The program analyses the tunnel excavation performance with the assumption that the cycle begins with blast. Therefore, the drilling time for the first blast is not taken into consideration. The assumption might also cause a mistake when the geological condition or the round length changes, meaning that the round duration changes. The mistake is the result of the fact that the drilling and charging is then calculated for the round corresponding to the geological condition, or the round length of the round before. It was assumed that the mistake is negligible, however, gives the possibility for improvement for further work with the Model.

The Model does not include national holidays, nor the summer break applied in the Norwegian Tunnelling industry.

14.2 Results

It was decided to run the model based on five different input data sets in order to show the influence of rock condition, vibration limits, time window restrictions and weekly working hours.

A. Real estimation A (rock class E supported with spilling bolts and shotcrete girders)

Real estimation reflects the real rock condition with the assumption that E rock class requires the spilling bolts and shotcrete girders installation, as well as takes the actual vibration limits into consideration. Another assumption is that the work can be performed 20h per day 5 days in a week with respect to the time windows.

According to the analysis the tunnel excavation would take 393 days including weekends.

B. Real estimation B (rock class E supported with shotcrete and bolts)

Real estimation B reflects the real rock condition with the assumption that E rock class does not require the spilling bolts and shotcrete girders installation as well as takes the actual vibration limits into consideration. Another assumption is that the work can be performed 20h per day 5 days in a week with respect to the time windows.

According to the analysis the tunnel excavation would take 368 days including weekends.

C. No blasting time windows

No blasting time windows prediction reflects the real rock condition with the assumption that E rock class does not require the spilling bolts as well as takes the actual vibration limits into consideration. Another assumption is that the work can be performed 20h per day 5 days in a week without any blasting windows restrictions meaning that the blast can happen at any time between 4:00 and 24:00.

According to the analysis the tunnel excavation would take 323 days including weekends.

D. No vibration limits

No vibration limits prediction reflects the real rock condition assuming that the E class rock does not require the spilling bolts or shotcrete girders as well as assumes that there are no vibration limits that influence the round length. The only factor influencing the round length in this case is the geological condition of the rock mass. Another assumption is that the work can be performed 20h per day 5 days in a week with respect to the time windows.

According to the analysis the tunnel excavation would take 316 days including weekends.

E. 24/5 extended daily working hours

Extended daily working hours prediction reflects the real rock condition with the assumption that E rock class does not require the spilling bolts and shotcrete girders as well as takes the actual vibration limits into consideration. While running this model it was assumed that the work is performed 24h per day 5 days a week.

According to the analysis the tunnel excavation would take 312 days including weekends.

F. Perfect geological condition

Perfect geological condition prediction assumes A/B geological condition along the tunnel as well as real vibration limits. Another assumption is that the work can be performed 20h per day 5 days in a week with respect to the time windows.

According to the analysis the tunnel excavation would take 297 days including weekends.

14.3 Comparison and analysis

Thanks to the TEP model, not only the total duration of the project can be estimated, by taking into consideration the rock condition, vibration limits, time window restrictions and weekly working hours. The model gives the possibility to run an analysis on different input data and to check their influence on the project duration.

Based on the TEP analysis, the total duration of the project can be estimated to take between 368 days (Real estimation B) and 393 days (Real estimation A). The difference of 25 days in the estimated project duration would be the result of applied spilling bolts and shotcrete girders rock support in support class E. The option B without additional support is set as the reference point in the analysis. The results of the duration of the project based on the analyzed option together with percentage difference of the project duration taking the Option B as a reference point were shown in the Table 32. In case of the Stavsjøfjelltunnelen project, time windows for blasting were assumed. The analysis, ran with the assumption that no time windows are required, shows that the total project duration could be reduced by 45 days down to 323 days (88% of the reference duration) (Table 32).

The analysis shows that the vibration limits have stronger influence on the total project duration then the time windows restriction. If the time vibration restrictions were omitted, the project duration would be reduced by 52 days to 316 days (86% of the reference duration) (Table 32).

When it comes to working hours, the standard approach in the Norwegian industry for non-critical projects is 100h per week. However, one should consider extended working hours in order to finish the project earlier. In the case of extended working hours from 20h per day to 24h per day, 5 days a week, the total project duration would be reduced by 56 days to 312 days (85% of the reference duration). This means that 20% more working hours gives a reduction in total duration of 15% (Table 32).

The strongest influence on the project duration has the geology. The assumption that the geological support class is A/B along the tunnel results in reduced project duration by 71 days down to 297 days reducing the duration of the project by 19% (Table 32).

Figure 55 presents the excavation duration dependent on the tunnel chainage for all of the analysed options. The steeper diagram line, the lower performance is. In most cases, the diagram follows a similar trend. The biggest difference can be seen when it comes to Option D analysis. In the area between PEG 14700 – 14950, the performance does not decrease, as it happens in other cases. Decreased weekly performance is the result of vibration restrictions and limited round lengths as well as maximum charge amount per hole (Figure 35). Decreased weekly performance as a result of the limited round lengths does not apply to the D Option analysis. The sections between PEG 13750 - 13775 and PEG 14270 – 14290 show decreased performance due to bad rock conditions. Since the perfect geology analysis is not sensitive to changes in geology, the performance does not change along these sections. One can see that the Option C diagram flattens between PEG 14700 – 14950. This section could be excavated 24 days faster if the time windows were not applied, reducing the section excavation time from 96 to 72 days.

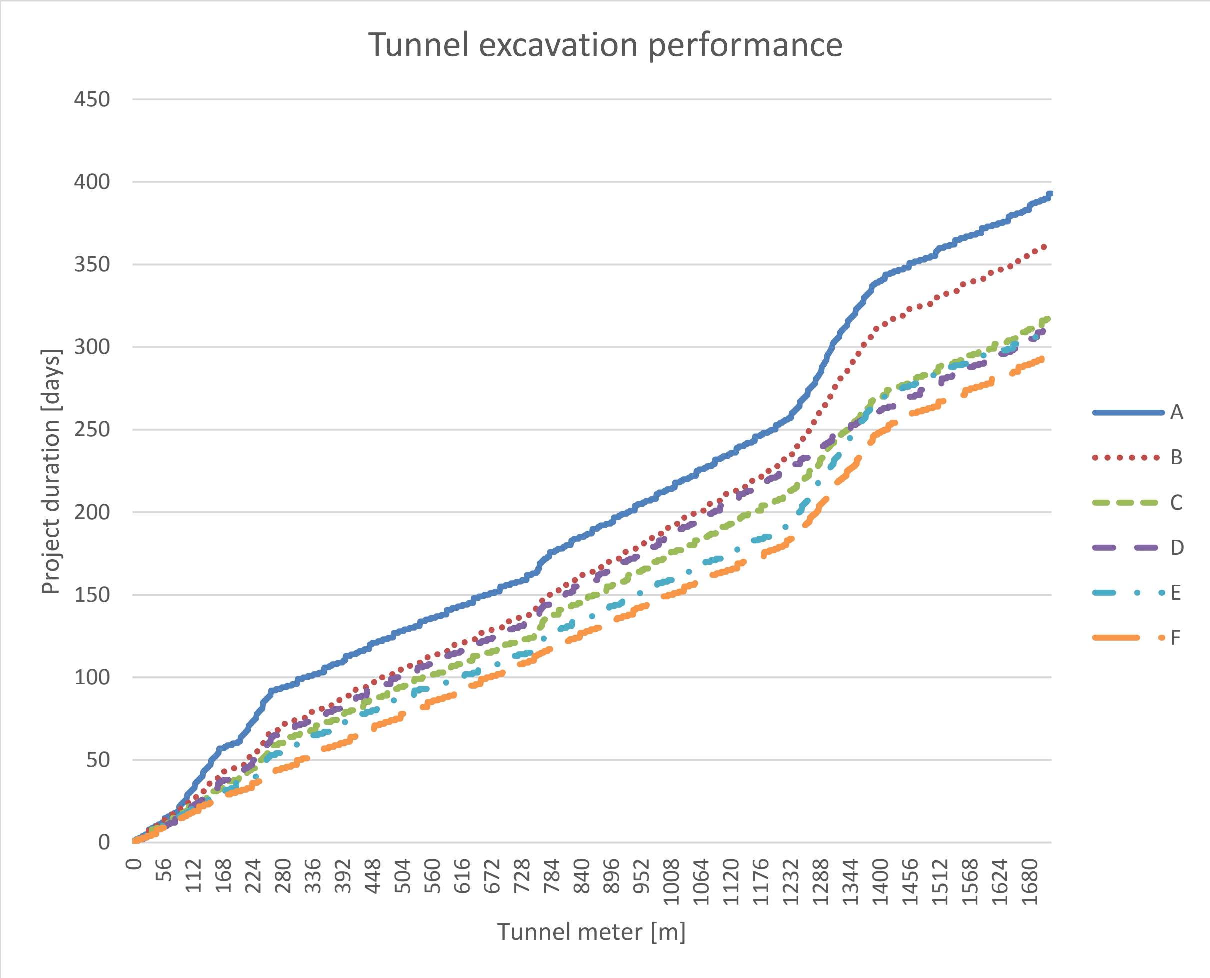

Figure 55 Excavation duration dependent on the tunnel chainage for all of the analyzed options

Table 32 Project duration

Analysed option	A	B	C	D	E	F
Days	393	368	323	316	312	297
percentage difference*	107%	100%	88%	86%	85%	81%

*percentage difference is understood as a % share of the analyzed option in relation to Option B considered as a reference point

14.4 Discussion

14.4.1 Tunnel excavation performance

The total project duration has been estimated to be from 368 to 393 days, dependent on the need of spilling bolts and shotcrete girders support in E rock class sections. Factors like geological condition and vibration restrictions are human independent and cannot be changed, however, the analysis shows that the factors have influence on the excavation time, especially when it comes to the geology. Additional concern is the possibility of the injection. The results does not include systematic injection between Modalen (profile no. 14550-15050) and Svartdalsdammen (profile no. 14050-14300). Due to the estimation eventual systematic injection would increase project duration by 7 weeks what equals 49 days.

When it comes to human dependent factors, the analysis shows that the increment of the working hours by 20% would improve total tunnel excavation performance by 15%. In order to decide whether this solution is efficient or not, a short cost discussion will be undertaken in the next chapter – Excavation Costs.

The analysis shows that 12% time savings could be achieved by removing the time windows. In order to achieve this, one should consider the by-pass roads or closing the traffic for tunnel blast and inspection. Table 33 shows the amount of blasts for the reference option (B) in each of the time windows. According to the Figure 2 and Figure 3 presented in the Chapter 3.2 showing the daily traffic in the Være and Hell tunnels, the highest traffic intensity is reached between 6:30 and 10:00 and between 14:00 and 19:30. This means that during 81 days, the traffic in Stavsjøfjelltunnelen has to be closed for approximately 1 hour between 6:30 and 10:00 as well as during 137 days between 14:00 and 19:30. In case of closing the traffic, the tunnel could be by-passed. Such solution requires transferring the traffic to the city. While using the by-pass road the section would be around 2 km longer (GoogleMaps, 2016). No data about the traffic capacity through the city was obtained. Such solution could be taken into account in the project risk and sustainability assessment and should be further evaluated.

Table 33 The amount of blasts in each of the time windows

from 4:00 to 5:00	from 6:30 to 10:00	from 14:00 to 19:30	from 23:30 to 24:00	from 5:00 to 6:30	from 10:00 to 14:00	from 19:30 to 23:30
13	81	137	8	50	106	112

14.4.2 Excavation costs

As the result of the analysis presented in the Chapter 14.4.1 two options for project performance improvement (Option C and Option E) should be analysed paying special attention to the project costs. Option B is used as a reference point. While analysing the costs one should consider the project cost itself as well as the cost for the society of certain solutions.

Option C:

Option C assumes that the work could be performed 20h per day 5 days in a week without any blasting windows restrictions meaning that the blast could happen at any time between 4:00 and 24:00. When it comes to the project related costs this solution would not result in additional investor's costs like: employment, housing etc. Implementation of this option requires by-passing the tunnel due to the necessary 1 hour brake for the blast and tunnel inspection during and after each blast.

This solution would reduce the total project duration by 25 days and possibly project cost itself. However, would cause traffic related costs, logistics problems and travel duration extension. In this case the society costs could be higher than benefits obtained by reducing the project duration.

In order to evaluate the implementation of the Option C one should make Impact assessment in order to include both Benefit-cost analysis as well as Non-monetised impacts. Such Socio-economic analysis could bring the recommendation weather to implement Option C or not.

Option E:

Option E assumes that the work is performed 24h per day 5 days a week what equals to 120 working hour a week. According to the Technical Manager Bernd Ifland from Marti Norge such solution would probably require shift changes. Dependent on the amount of the new employees the production not related costs could increase by 80 000 NOK per month per every additional employee. (Ifland, 2016) In this case around 10 additional people would be employed for the project period which would be around 10 months. The calculation shows that the project cost would increase by around 8 million kroners. Additionally, in order to work during the nights in the tunnelling industry in Norway both the workers and the company have to belong to Labour Union. Signing up in the Labour Union means additional costs agreed individually with the Union. However, the costs related to membership can be significant.

14.5 Results judgement and final result

The total duration of the project was calculated to be between 368 and 393 days not including the national holidays and systematic injection. If the systematic injection was applied the total duration of the project would be 7 weeks longer. Additional cost analysis should be performed when it comes to the Option C and Option E. Option C could reduce the project duration by 45 days down to 323 days taking Option 2 as a reference point. Option E could reduce the project duration by 56 days down to 312 days taking Option 2 as a reference point.

15. Further work

The estimation of the project duration is based on the NTNU models, existing reports concerning the analysed project, specialists opinions as well as personal experience. The main factor influencing the tunnel design as well as excavation performance is the geological condition. Additional geological analysis could benefit more accurate predictions. Special attention should be paid to the living area between PEG 14700 – 14950. One could improve the input data for vibration limits estimation. Such improvement could be obtained by external geological evaluations and tests. The project duration prediction could be reduced if the analysis shows that the results obtained in the Thesis were conservative.

The Tunnel Excavation Performance Model could be improved by creating easy interface for the program. The program could be additionally upgraded to project performance monitoring tool by enabling the user to update the input data and actual tunnel excavation performance. The tool could become a live prediction model and keep truck on the excavation performance enabling tunnel project managers having better control over the project performance.

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