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Grouting Design Considering Different Geological Conditions

Grout evaluation for the extension of the Blue Metro Line

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Abstract

This thesis forms the basis for a new documented experience of grouting design and work in the extension of the blue metro line in Stockholm. It includes documentation of the grouting design based on theoretical basis, stop criteria and fan geometries for favorable and unfavorable geological conditions. The work is concerned in evaluating the design stop criteria in different geological conditions to assess the efficiency of grouting process, and its compatibility to the maximum permissible leakage according to applications submitted to the land and environmental court in Stockholm.

The work was conducted in cooperation with TYPSA AB and SWECO; the joint venture who designed the grouting work.

This work was initiated by studying the design documents and reports, requirements, geological and hydrogeological prognosis documents. Six access tunnels were analyzed, with different work percentage based on the actual work achieved at the site at the time of conducting the study. Each access tunnel stretch was determined in terms of geological condition (rock quality), hydrogeological domain, and grouting class (IK1, IK2 or IK3). Evaluations and assessments were done for different aspects including evaluating the grout volume uptake per each grouting class, calculating the percentage of boreholes that stopped by time, volume or zero flow per grouting class, and comparing the measured leakage with prognosed leakage to check the efficiency of the design and implementation phases. RTGC (Real Time Grouting Control) method was also applied on some fans to check its validity in grout optimization, knowing that it is a relatively new method and not yet fully validated.

The results showed that geological mappings during the implementation phase were slightly different from the mappings done during the design phase, which is expected due to the high uncertainties in rock mass science. It was also shown that the design stop criteria in this project were promising, through which they have satisfied the requirements according to the application to the land and environmental court. Average grout uptake in typical injection classes were compatible with the results in City Line projects, where the average grout uptake in 2 L/m. However, results also showed that in weakness zones, the average grout uptake was different with high standard deviations. Knowing the fact that unfavorable geological conditions were classified based on different parameters, it is not possible to find one reference value for the grout uptake, but instead results can be used as references in similar geological conditions in main tunnels work and future projects. Some recommendations are made in this thesis on the design stop criteria in weakness zones, surface rock domains, and at fans injected at large water depth. These zones always form the basis for controversial discussions and thus, if documentation of grouting work is carried out and continued in this project, then more knowledge can be gained and transferred to other projects. As part of this thesis, RTGC was applied in favorable conditions where it showed very promising results, the matter that makes it possible to optimize the stop criteria and actual work by conducting trial grouting. However, in unfavorable geological conditions, the RTGC could not be applied because the dimensionality of the flow is 3D, while RTGC was developed for 1D and 1D flow. Therefore, and since it was proven to be as a promising tool, further studies are recommended to develop the method for 3D flow.

Sammanfattning

Denna avhandling utgör grunden för en ny dokumenterad upplevelse av injekteringsdesign och arbete i förlängningen av den blå tunnelbanelinjen i Stockholm. Den innehåller dokumentation av injekteringsdesignen baserad på teoretisk grund, stoppkriterier och fläktgeometrier för gynnsamma och ogynnsamma geologiska förhållanden. Arbetet handlar om att utvärdera designstoppskriterierna under olika geologiska förhållanden för att bedöma injekteringsprocessens effektivitet och dess kompatibilitet med maximalt tillåtna läckage enligt ansökningar som lämnats in till Mark- och miljödomstolen i Stockholm.

Arbetet genomfördes i samarbete med TYPSA AB och SWECO; det gemensamma företaget som designade injekteringsarbetet.

Detta arbete inleddes genom att studera designdokument och rapporter, krav, geologiska och hydrogeologiska prognosdokument. Sex arbetstunnlar analyserades, med olika arbetsprocent baserat på det faktiska arbete som uppnåddes på platsen när studien genomfördes. Varje arbetstunnelsträcka bestämdes i termer av geologiskt tillstånd (bergkvalitet), hydrogeologisk domän och injekteringsklass (IK1, IK2 eller IK3). Utvärderingar och bedömningar gjordes för olika aspekter inklusive utvärdering av injektionsmassans upptag per injekteringsklass, beräkning av andelen borrhål som stoppades av tid, volym eller nollflöde per injekteringsklass och jämförelse av det uppmätta läckaget med prognostiserat läckage för att kontrollera effektiviteten av design- och implementeringsfaserna. RTGC-metoden (Real Time Grouting Control) tillämpades också på vissa fans för att kontrollera dess giltighet vid injekteringsoptimering, med vetskap om att det är en relativt ny metod och ännu inte helt validerad.

Resultaten visade att geologiska kartläggningar under implementeringsfasen skilde sig något från kartläggningarna som gjordes under designfasen, vilket förväntas på grund av den höga osäkerheten inom bergmassevetenskap. Det visades också att designstoppskriterierna i detta projekt var lovande, genom vilka de har uppfyllt kraven enligt ansökan till mark- och miljödomstolen. Genomsnittligt injektering av injekteringsbruk i typiska injektionsklasser var förenligt med resultaten i City Line-projekt, där det genomsnittliga injekteringen av injekteringsbruk i 2 liter / m. Resultaten visade emellertid också att i svaghetszoner var det genomsnittliga injekteringen av injekteringsbruk annorlunda med höga standardavvikelser. Att veta det faktum att ogynnsamma geologiska förhållanden klassificerades baserat på olika parametrar är det inte möjligt att hitta ett referensvärde för injekteringen av injekteringsbruk, utan resultaten kan användas som referenser i liknande geologiska förhållanden i huvudtunnelarbeten och framtida projekt. Några rekommendationer görs i denna avhandling om designstoppskriterier i svaghetszoner, ytbergsdomäner och vid fläktar injicerade på stort vattendjup. Dessa zoner utgör alltid grunden för kontroversiella diskussioner, och om dokumentation av injekteringsarbete utförs och fortsätter i detta projekt, kan mer kunskap fås och överföras till andra projekt. Som en del av denna avhandling tillämpades RTGC under gynnsamma förhållanden där det visade mycket lovande resultat, det som gör det möjligt att optimera stoppkriterierna och det faktiska arbetet genom att utföra försöksfogning. Men under ogynnsamma geologiska förhållanden kunde RTGC inte tillämpas eftersom flödets dimension är 3D, medan RTGC utvecklades för 1D- och 1D-flöde. Därför, och eftersom det visade sig vara ett lovande verktyg, rekommenderas ytterligare studier för att utveckla metoden för 3D-flöde.

Preface

This master's thesis was written at KTH Royal Institute of Technology for TYPSA AB. The work was performed in coordination with SWECO, FUT, and Bever Control. The data was mainly collected from Mashuqur Rahman who was the direct co-supervisor of this thesis, and from FUT database system. The work began in Stockholm in February 2021 and ended in July 2021. At KTH, the supervisor and examiner was Professor Stefan Larsson, PhD, professor in Geotechnology and Head of the division Soil and Rock Mechanics at the Department of Civil and Architectural Engineering.

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List of Symbols

Symbol	Unit	Description
Q	m^3/s	Flow
B_{min}	mm	Minimum aperture
B_{crit}	mm	Critical aperture
ξ	-	Skin factor
H	m	Depth underground water level
L	m	Length of tunnel stretch
K_i	m/s	Hydraulic conductivity after grout
K_o	m/s	Hydraulic conductivity before grout
K_{3D}	m/s	Effective Matheron's hydraulic conductivity
K_g	m/s	Geometric mean of hydraulic conductivity
r	m	Radius of tunnel opening
t	m	Thickness of grouted zone
I	m	Grout penetration length
I_D	-	Relative grout penetration length
I_{max}	m	Maximum grout penetration length
t_o	min	Characteristic grouting time
t	min	Grout time
t_D	-	Relative grout time
b	mm	Aperture
μ_g	pa.s	Viscosity of grout
Δp	Bar/Mpa	Grout excess pressure
P_g	Bar/Mpa	Grouting pressure

τ_o	Mpa	Yield stress of grout
$I_{D,Pb}$	m	Relative penetration length at pressure A
$I_{D,Pa}$	m	Relative penetration length at pressure B
t_{corr}	min	Corrected time
θ	-	Parameter for penetration analysis
vct	-	Water cement ratio
p_a	Bar/Mpa	Is the pressure at point a (constant)
p_b	Bar/Mpa	Is the pressure at point b (changes)
σ	-	Standard deviation

Abbreviations

AT	Access tunnel
FUT	Förvaltning för Utbyggd Tunnelbana
RTGC	Real Time Grouting Control
RMR	Rock Mass Rating
RQD	Rock Quality Designation
IK	Injektering Klass (grouting class)
NORMBG	Normal rock domain
YTBG	Surface rock domain
WEAKZ	Weakness zones

Illustrative definitions

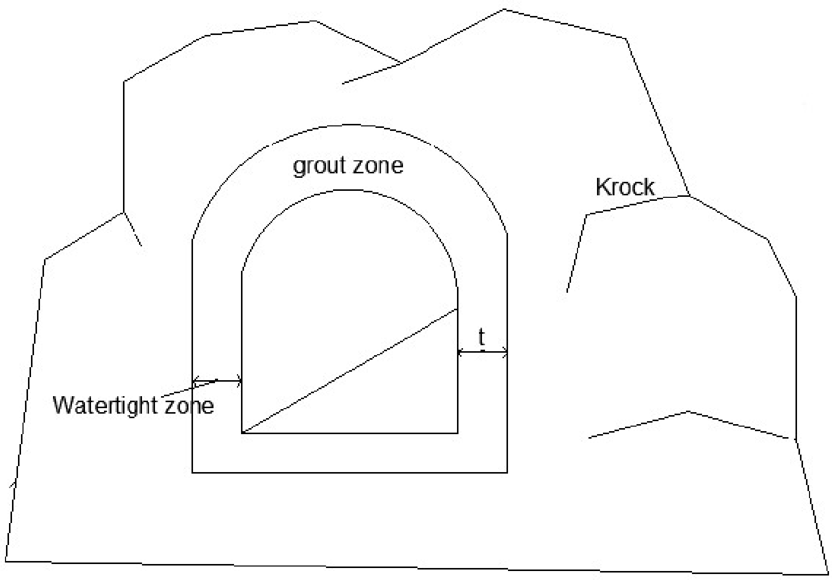


Table of contents

Abstract……………………………………………………………………………………...i

Sammanfattning………………………………………………………………………….ii

Preface…………………………………………………………………………………iii

Acknowledgements……………………………………………………………...………iii

List of Symbols…………………………………………………………………………v

Abbreviations……………………………………………………………………………..v

1. Introduction 1

1.1 Background 1

1.2 Objectives 2

1.3 Scope of work 2

2. Methodology 3

2.1 Available data 5

2.2 Given conditions and assumptions. 5

3. Literature review 6

3.1 History of grouting 6

3.2 Grouting material and tests 7

3.3 Grouting Theories 9

3.4 Grout penetration 10

3.5 Grouting pressure 11

3.6 Stope criteria 12

3.7 Real Time Grouting Control (RTGC) 12

4. Project description 17

4.1 Project location 17

4.2 Leakage requirements 18

4.2.1 Application for permit and final permit regarding groundwater inflow 18

4.2.2 Requirements regarding maximum leakage used for the evaluation of grouting design...... 19

4.2.3 Requirements regarding maximum leakage used for the execution and steering of grouting work 21

4.3 Geological Prognosis 21

4.4 Summary of Geological prognosis 30
4.5 Hydrogeological domains 31
4.6 Summary of hydrogeological domains 36
5. Grouting Design 37
5.1 Grouting Classes 37
5.2 Grout Characteristics 39
5.3 Stop criteria and fan geometries: Typical injection class. 39
5.4 Stop criteria and fan geometries: Specific technical solutions. 41
5.5 Grouting class for each tunnel stretch. 42
5.6 Summary of grouting class results 44
6. Results and Discussions 45
6.1 Evaluation of rock quality 45
6.2. Evaluation of executed grouting works 46
6.2.1 Average grout uptake 46
6.2.2 Evaluation of Stop Criteria 54
6.2.3 Summary of results 59
6.3 Comparison between grout uptake with City Line project 60
6.4 Evaluation of water ingress after grouting 62
6.4.1 Comparison between measured flow and prognosed flow 62
6.4.2 Summary of results 70
6.5 Evaluation of sealing efficiency 72
6.6. Correlation between hydrogeological domains and average grout uptake 78
6.7 Evaluation of grouting data using Real Time Grouting Control Method 80
6.7.1. Fan 403-Londonviadukten 80
6.7.2. Fan 294 (weakness zone)-Londonviadukten 83
6.7.3. Fan 487-Londonviadukten 89
7. Conclusion 91
7.1 Evaluation of grouting design and executed grouting works 91
7.2 Recommendation for stop criteria 93
7.3 Recommendation for follow up of grouting works 94
8. Recommendation for future works 95
References 95

A. Appendix 100
A.1 Typical grouting design fan geometry.. 100
A.2 IK3 design fan geometries and stop criteria 101
A.3 Weakness zones tables (IK3).. 115

List of Figures

Figure 3.1. Grout penetration with respect to aperture size (Stille, 2015).. 8
Figure 3.2. Preliminary design of grouting process (Stille,2012). 9
Figure 3.3. Grout penetration (Kobayashi et al., 2008).. 11
Figure 3.4. RTGC steps/layout.. 13
Figure 3.5. The relationship between grouting time and grouting pressure is shown in the left graph. The penetration change as the grouting pressure increases is shown in the graph to the right (Kobayashi, et al., 2008). 13
Figure 4.1. Extension of blue metro line.. 17
Figure 4.2. The evaluated six access tunnel's locations (highlighted in red)...................................... 18
Figure 4.3. Summary of control and maximum allowable leakage for each access tunnel.................. 19
Figure 4.4. Londonviadukten access tunnel marked in red, and the core boreholes location is indicated in blue (FUT,2019)... 23
Figure 4.5. Järla Östra access tunnel is shown as a blue dashed line, and the core boreholes location is indicated by the green dotted lines with a yellow ring which mark the core boreholes 16S0855, 16S0857 and 16S0859 (FUT,2019).. 24
Figure 4.6. Sundstabacken access tunnel is shown as a dashed blue line, and the core boreholes location is indicated by the yellow rings (FUT,2019)... 26
Figure 4.7. Hammarby Fabriksväg access tunnel marked in red, and the completed core borehole's location around the tunnel (FUT,2020)... 27
Figure 4.8. Core drillings, line mappings around Värmdovägen access tunnel (FUT,2018)............... 28
Figure 4.9. Core drillings, line mappings around Skönviksvägen access tunnel (FUT,2019). 29
Figure 4.10. Distribution rock classes along the access tunnels... 31
Figure 4.11. Geographical division of hydrogeological domains (FUT,2018). 32
Figure 4.12. Distribution of the hydraulic conductivities for normal rock-west on a lognormal scale, based on water loss measurements performed on holes and water wells (FUT,2018)......................... 33
Figure 4.13. Distribution of the hydraulic conductivities for normal rock-east on a lognormal scale, based on water loss measurements performed on holes and water wells (FUT,2018)......................... 33
Figure 4.14. Distribution of the hydraulic conductivities for surface rock on a lognormal scale, based on water loss measurements performed on holes and water wells (FUT,2018)................................... 34
Figure 4.15. Distribution of the hydraulic conductivities for weakness zone-sea crossing on a lognormal scale, based on water loss measurements performed on holes and water wells (FUT,2018). 34
Figure 4.16. Distribution of hydrogeological domain along the access tunnels.................................. 36

Figure 5.1. Distribution of hydrogeological domain along the access tunnels. 44
Figure 6.1.Rock quality distribution based on design documents for evaluated sections. 45
Figure 6.2.Rock quality distribution based on geological mappings from the site for evaluated sections. 46
Figure 6.3.Average grout take for each grouting class excluding the hole filling volume (AT Londonviadukten). 47
Figure 6.4: Grout uptake variation for each round-fan 294-Londonviadukten. 48
Figure 6.5: Grout uptake variation for each round-fan 304-Londonviadukten. 48
Figure 6.6. Average grout take for each grouting class excluding the hole filling volume (Järla Östra). 49
Figure 6.7. Average grout take for each grouting class excluding the hole filling volume (Sundstabacken). 50
Figure 6.8. Average grout take for each grouting class excluding the hole filling volume (Hammarby Fabriksväg). 51
Figure 6.9. Average grout take for each grouting class excluding the hole filling volume (Värmdovägen). 53
Figure 6.10. Average grout take for each grouting class excluding the hole filling volume (Skönviksvägen). 54
Figure 6.11. Comparison of grouting stop criteria (AT Londonviadukten). 55
Figure 6.12. Comparison of grouting stop criteria (AT Järla Östra). 56
Figure 6.13. Comparison of grouting stop criteria (AT Sundstabacken). 57
Figure 6.14. Comparison of grouting stop criteria (AT Hammarby Fabriksväg). 58
Figure 6.15. Comparison of grouting stop criteria (AT Värmdovägen). 58
Figure 6.16.Comparison of grouting stop criteria (AT Skönviksvägen). 59
Figure 6.17.Evaluation of typical injection uptake in City line project. 61
Figure 6.18. Evaluation of typical injection uptake in blue metro- line extension project. 61
Figure 6.19. Londonviadukten accumulated measured flow Vs. prognosed flow in (L/min). 63
Figure 6.20. Londonviadukten measured flow Vs. maximum allowed flow in (L/min/100m). 64
Figure 6.21.Järla Östra accumulated measured flow Vs. prognosed flow in (L/min). 64
Figure 6.22.Järla Östra measured flow Vs. maximum allowed flow in (L/min/100m). 65
Figure 6.23.Sundstabacken accumulated measured flow Vs. prognosed flow in (L/min). 66
Figure 6.24. Sundstabacken measured flow Vs. maximum allowed flow in (L/min/100m). 66
Figure 6.25. Hammarby Fabriksväg accumulated measured flow Vs. prognosed flow in (L/min). 67
Figure 6.26.Hammarby Fabriksväg measured flow Vs. maximum allowed flow in (L/min/100m). 67
Figure 6.27. Värmdovägen accumulated measured flow Vs. prognosed flow in (L/min). 68
Figure 6.28.Värmdovägen measured flow Vs. maximum allowed flow in (L/min/100m). 69
Figure 6.29. Skönviksvägen accumulated measured flow Vs. prognosed flow in (L/min). 69
Figure 6.30.Skönviksvägen measured flow Vs. maximum allowed flow in (L/min/100m). 70
Figure 6.31.Londonviadukten estimated flow before grouting Vs. The measured flow after grouting for evaluated sections. 73

Figure 6.32. Järka Östra estimated flow before grouting Vs. The measured flow after grouting for evaluated sections. 73
Figure 6.33. Värmdovägen estimated flow before grouting Vs. The measured flow after grouting for evaluated sections. 73
Figure 6.34. Hammarby Fäbriksväg estimated flow before grouting Vs. The measured flow after grouting for evaluated sections. 74
Figure 6.35. Skönviksvägen estimated flow before grouting Vs. The measured flow after grouting for evaluated sections. 74
Figure 6.36. Sundstabacken estimated flow before grouting Vs. The measured flow after grouting for evaluated sections. 74
Figure 6.37. Estimated flow before grouting Vs measured flow after grouting for evaluated sections in AT Londonviadukten. 75
Figure 6.38. Estimated flow before grouting Vs measured flow after grouting for evaluated sections in Hammarby Fabriksväg. 75
Figure 6.39. Estimated flow before grouting Vs measured flow after grouting for evaluated sections in AT Londonviadukten 75
Figure 6.40. Estimated flow before grouting Vs measured flow after grouting for evaluated sections in Skönviksvägen 75
Figure 6.41. Estimated flow before grouting Vs measured flow after grouting for evaluated sections in Järla Östra. 75
Figure 6.42. Estimated flow before grouting Vs measured flow after grouting for evaluated sections in Sundstabacken. 76
Figure 6.43.Average grout uptake for normal rock. 79
Figure 6.44. Average grout uptake for rock in weakness zone. 79
Figure 6.45. Average grout uptake for surface rock. 80
Figure 6.46.Flow and pressure distribution for hole number 16 in fan 403 as a function of time. 81
Figure 6.47. Dimensionality indicates that the flow in hole number 16 is likely 1D. 81
Figure 6.48. The achieved penetration length vs. grouting time. 82
Figure 6.49.Estimation of the fracture's apertures. 83
Figure 6.50.Estimated flow Vs. Real recorded flow. 83
Figure 6.51: Flow and pressure distribution for hole number 24 in fan 294 as a function of time in the first round. 85
Figure 6.52: Flow and pressure distribution for hole number 24 in fan 294 as a function of time in the second round. 85
Figure 6.53: Flow and pressure distribution for hole number 24 in fan 294 as a function of time in the third round. 86
Figure 6.54. Dimensionality indicates that the flow in hole number 24 is likely 3D in the first round. 86
Figure 6.55: Dimensionality indicates that the flow in hole number 24 is likely 1D in the second round. 87
Figure 6.56: Dimensionality indicates that the flow in hole number 24 is likely 1D in the third round. 87

Figure 6.57: Flow and pressure distribution for hole number 24 in fan 294 as a function of time in the fourth round. 88
Figure 6.58: Dimensionality indicates that the flow in hole number 24 is likely 1D in the fourth round. 88
Figure 6.59. Flow and pressure distribution for hole number 10 in fan 487 as a function of time. 89
Figure 6.60. Dimensionality indicates that the flow in hole number 10 is 3D. 90
Figure A.1. Tip distance in a typical grouting fan. 100
Figure A.2. Boreole average length (21-24m), overlap 5m, and stick of 5m in a typical grouting fan. 100
Figure A.3.Specific grouting fan in weakness zones 101
Figure A.4. Borehole's length, overlap, and stick in specific grouting fan in weakness zones. 101
Figure A.5.Specific grouting fan in large depth zone 103
Figure A.6.Borehole's length, overlap, and stick in specific grouting fan in large depth zones. 103
Figure A.7. Specific grouting fan in weakness zones 104
Figure A.8. Borehole's length, overlap, and stick in specific grouting fan in weakness zones. 105
Figure A.9. Specific grouting fan in weakness zones 106
Figure A.10. Borehole's length, overlap, and stick in specific grouting fan in weakness zones. 106
Figure A.11. Specific grouting fan in passage near existing structure zone 108
Figure A.12. Borehole's length, overlap, and stick in specific grouting fan in passage near existing structure zone. 108
Figure A.13. Specific grouting fan. 109
Figure A.14. Borehole's length, overlap, and stick in specific grouting fan. 110
Figure A.15.Plan view for connection to main tunnel 111
Figure A.16. Specific grouting fan for the connection to the main tunnel. 111
Figure A.17.Borehole's length, overlap, and stick in specific grouting fan in case of connection to main tunnel. 112
Figure A.18. Specific grouting fan at the pre-cut. 113
Figure A.19. Borehole's length, overlap, and stick in specific grouting fan at the pre-cut. 113

List of Tables

Table 1.1.Percentage of work done for each access tunnel. 2
Table 4.1. Difficulty degree of grouting (Remiss-Projektering av förinjektering för bergtunnlar, Trafikverket, version 170404). 20
Table 4.2.Classification of rock class based on RMR_{base} system. 22
Table 4.3.Classification of rock classes for Londonviadukten access stunnel based on RMR_{base} system. 23
Table 4.4.Classification of rock classes for Järla Östra access tunnel based on RMR_{base} system. 25
Table 4.5.Classification of rock classes for Sundstabacken access tunnel based on RMR_{base} system. 26
Table 4.6.Classification of rock classes for Hammarby Fabriksväg access tunnel based on RMR_{base} system. 28
Table 4.7. Classification of rock classes for Värmdovägen access tunnel based on RMR_{base} system. 29
Table 4.8.Classification of rock classes for Skönviksvägen access tunnel based on RMR_{base} system. 30
Table 4.9.Characteristic values of hydraulic conductivity within assumed hydrogeological domains. 35
Table 4.10. Identified hydrogeological domains for each tunnel stretch along the 6 access tunnels. 35
Table 5.1.Grouting class definition. 37
Table 5.2. Requirements for the properties of the cement grout are required in the form of a range for each property. 39
Table 5.3. Grout mixture properties. 39
Table 5.4.Grouting pressure required based on design documents (1331). 40
Table 5.5.General stop criteria for typical grouting class. 40
Table 5.6. Additional grouting conditions. 41
Table 5.7. Additional grouting rounds. 41
Table 5.8.Grouting class for each tunnel stretch. 42
Table 6.1:Londonviadukten-grout evaluation. 47
Table 6.2: Järla Östra-grout evaluation. 49
Table 6.3.Sunstabacken grout evaluation. 50
Table 6.4. Hammarby Fabriksväg grout evaluation. 51
Table 6.5.Värmdovägen grout evaluation. 52
Table 6.6. Skönviksvägen grout evaluation. 53
Table 6.7: Achieved hydraulic conductivity after grouting for each section in the 6-access tunnels. Yellow highlighted fans achieved lower sealing efficiency than required. 77
Table A.1. Grouting pressure required based on design documents (1331). 102
Table A.2. Complementary actions. 102
Table A.3.Stop criteria round 1 102
Table A.4. Stop criteria round 2. 102
Table A.5.Stop criteria 104
Table A.6.Complementary actions 104
Table A.7.Stop criteria. 105
Table A.8.Complementary actions. 105

Table A.9. Grouting pressure required based on design documents (1331)...................................... 107
Table A.10.Stop criteria round 1. 107
Table A.11.Stop criteria round 2. 107
Table A.12.Complemenatry actions. 107
Table A.13.Stop criteria. 109
Table A.14.Complemantary actions. 109
Table A.15.Stop criteria. 110
Table A.16.Complemantary actions. 110
Table A.17.Grouting pressure required based on design documents (1331)..................................... 112
Table A.18.Stop criteria. 112
Table A.19.Complemantary actions. 112
Table A.20.Stop criteria. 114
Table A.21.Complemantary actions. 114
Table A.22.Lomdonviadukten-evaluation of geological conditions. 115
Table A.23.Järla Östra-evaluation of geological conditions. 116
Table A.24.Sundstabacken-evaluation of geological conditions.. 116
Table A.25.Hammarby Fabriksväg-evaluation of geological conditions. 117
Table A.26.Värmdovägen-evaluation of geological conditions. 118
Table A.27.Skönviksvägen-evaluation of geological conditions. 118

1. Introduction

1.1 Background

In tunneling projects, ground water is a sensitive critical factor that affects the construction site. Thus, sealing of fractures becomes an essential step to maintain safety of the structure. Sealing is done usually by cement-based grouting. The necessity of grouting and how extensive the grouting process should be, depend on the investigations and conditions of the construction site. Grouting is used to seal rock fractures and thus reduce water inflow into the tunnels, and to strengthen the rock mass ahead of the drive (Houlsby,1990)

Grouting process is governed by many factors at the site. Due to complexity of rock mass conditions, it is not simple have an accurate answer on when the grout must stop. The main aim of the grouting is to reduce the hydraulic conductivity of the rock mass up to a certain limit and achieve the required sealing efficiency according to the requirement for leakage according to the application to the land and environmental court.

Since grouting is known to be costly and time-consuming process, it is necessary to optimize the grouting process (Eliasson, 2012). During the past decades, more knowledge has been gained in the field of tunneling regarding grouting. General grouting concepts can be developed by combining statistical data from field investigations such as core drill holes, with refined methods for estimating grouting times and required penetrations. Monitoring and using tools and parameters in active design are essential for optimizing these concepts.

Many studies have been made on developing grouting design concepts to optimize the grouting process and defining stop criteria (Gustafsson and Stille,2005). Stop criteria describe when the grout should stop to achieve sufficient penetration length into the fractures to avoid excessive and uncontrolled grouting. The basis for grouting strategies and design concepts have been developing over the last years. For example, stop criteria for cement grouting was studied to give a clear understanding on when the grout process should be stopped. Other studies were made using theoretical approaches such as RTGC "Real Time Grouting Control ", which was validated on certain case studies (Rafi,2013). Research was also performed by the Swedish Transport Agency to initiate the implementation of a unified design approach, to increase knowledge and awareness about best grouting practices (Creützt, et al. 2017).

A relatively new method was developed to optimize the grouting process, called RTGC. This method is used to calculate the grout penetration length in real time, which helps in defining optimized stop criteria based on achieving the required penetration length. The validity of this method was checked in good geological conditions by Kobayashi et. al. (2008) and Rafi (2010). Tests on RTGC validity were made using Äspö HRL data, where it was concluded that this method is applicable for grouting design and control (Kobayashi et. al. 2008). Another application was made in sedimentary rock with Gotvand Dam data, where it was also concluded that RTGC gives good information on the mixture penetration in real time, and thus the grouting time and mixture recipe can be optimized (Rafi, 2010). In this work, the RTGC was also applied on some fans to check its validity in unfavorable geological conditions.

In this work, data from the extension of the Blue Metro Line project was used, to evaluate the grouting design and stop criteria considering both favorable and unfavorable geological conditions. This case study was chosen due to the variation and complexity of geological and hydrogeological conditions, where the rock mass classification was performed based on RMR_{base} system. The project is designed by a joint venture TYPSA AB and SWECO. The extension project starts at Kungsträdgården and continues into two branches: east to Nacka and south to Sockenplan via Gullmarsplan.

1.2 Objectives

In this work, the main interest was to come up with recommendations for the optimization of grouting process, which can be used in future projects with similar geological conditions, as those assessed in the extension of the Blue Metro Line. This study is of a great importance for documenting new experiences, as grouting process largely depends on observations from past projects. Six access tunnels have been investigated to check the effect of geological conditions' variation on the grout uptake and stop criteria.

The main objectives of the research are:

- Evaluation of the grout design for different geological and hydrogeological conditions to check if the required sealing efficiency is achieved.
- Evaluation of the design stop criteria in the blue metro line extension for different grouting classes, and make recommendations on the most effective stop criteria, i.e., grout take and grouting time.
- Check the validity of grouting time and spread of grout in favorable and unfavorable geological conditions (i.e., using the Real Time Grouting Control, RTGC method).

1.3 Scope of work

Six access tunnels were analyzed. The number of evaluated fans in each access tunnel is different based on the amount of data available at the time of doing this study. Table 1.1 below shows the evaluated length and percentage of work done for each access tunnel. It should be mentioned that Londonviadukten access tunnel has two branches: AT1 and AT2. In this work, only AT1 is analyzed because AT2 branch is postponed due to the high leakage.

Table 1.1.Percentage of work done for each access tunnel.

Access Tunnel	From	To	Length(m)	Last section evaluated	Length evaluated (m)	% of work done
Londonviadukten	0+035	0+821	786	0+515	480	61%
Järla Östra	0+020	0+495	475	0+361	341	72%
Sundstabacken	0+022	0+461	439	0+444	422	96%
Hammarby Fabriksväg	0+046	0+578	532	0+232	186	35%
Värmdovägen	0+017	0+575	558	0+226	209	37%
Skönviksvägen	0+030	0+409	379	0+223	193	51%

2. Methodology

To achieve the desired objectives mentioned in chapter 1, a methodology was developed to perform a comprehensive evaluation of many aspects of the grouting process in design documents and in actual work. The methodology is summarized as follows:

- Assessing engineering geological prognosis; design requirements and design reports.
- Evaluating the grout volume uptake for each grouting fan, for each grouting round in 6 access tunnels. The grout uptake was then calculated for each grouting class in each tunnel separately.
- Statistical calculations were made for calculating the average and standard deviation for grout uptake per grouting class. Grouting classes are described in detail in chapter 5.
- Calculations of the percentage of boreholes stopped by time, volume, and flow. The percentage of the achieved stop criteria was then evaluated for each grout class.
- Comparison between the measured water flow after grouting with the prognosed flow and application to the environmental court.
- Comparing the measured flow after grouting and estimated flow before grouting and estimating the achieved hydraulic conductivity after grouting.
- Application of RTGC on some fans. This step includes comparing the dimensionality for each grouting class, calculating the theoretical penetration lengths to evaluate the designed stop criteria.
- Summarizing the mapping results from field and comparison with design.

Method of Analysis

The assessment was done for both favorable and unfavorable geological conditions, at which four main parameters were assessed: total number of dominant fractures' direction, fractures' direction vs. the tunnel direction, fractures' endurance, and fractures' fillings. The analysis is shown in Appendix (A) chapter A.3 for each access tunnel, as taken from the design documents. The final overall assessment was classified from best case to worst case as favorable, relatively favorable, relatively unfavorable, and unfavorable.

Due to high uncertainties and simplifications used in calculation models, the work requires comprehensive analysis of the geological conditions, design reports, and project demands. Thus, the work was divided into several parts.

- Assessing geological and hydrogeological reports summarized in chapter 3 for each of the six access tunnels was the first thing to start with. It is important to study and to be aware of all design requirements, especially leakage measurements as stated in the application submitted to the land and environmental court.
- The next step was to come up with a fair interpretation regarding the efficiency of the grouting process performed at the sites. This step involves the evaluation of rock mass quality, hydraulic conductivity for different hydrogeological domains, and design requirements at the areas where the tunnels are to be built.

- The prognosed rock mass quality was then compared with the actual rock mass quality based on the geological maps performed by the contractors.
- The next step was to identify tunnel stretches with different grouting classes which is explained in detail in chapter 5. This step is summarized by defining grouting data for the corresponding grouting class including grout mixture, stop conditions (time, volume, flow), and fan geometries. For the stop criteria and grout uptake analysis, an evaluation of the average grout uptake for each grouting class for all the tunnels was performed. Each fan was studied in terms of number of injection rounds, number of boreholes, boreholes length, total drilling length, grout mixture type, total injected volume, and grout uptake per meter. The percentage for boreholes that stopped by time, volume, or flow was calculated for each grouting class to judge the most dominant stop criteria for each grouting class.
- This was followed by calculation of the average gout uptake for each fan and thus for each grouting class, with a statistical calculation for the average uptake and standard deviation. For leakage analysis and sealing efficiency evaluations, the work started here by choosing the most representative data of the measured water flow, as performed by the FUT (Förvaltning för utbyggd tunnelbana) after grouting. In this step, all unrepresentative values of flow after grouting at different tunnel sections were eliminated to obtain a logical flow chart for each tunnel. These values were then compared to the prognosed flow and the application to the land and environmental court to assess the efficiency of the grout work. Using the representative leakage measurements values in equation (2-1) (Gustafsson, 2009), the hydraulic conductivities after grouting were calculated and compared to the values that are supposed to be obtained based on the design reports to assure the efficiency of the grouting design.

$$Q_{grouted} = \frac{2\,\pi\,k_0\,H\,L}{\ln(\frac{2H}{r})+(\frac{K_0}{K_i}-1)\ln\left(1+\frac{t}{r}\right)+\xi} \qquad (2\text{-}1)$$

Where:

H = Groundwater pressure.
L = Length scale for distance to which the inflow refers to.
K_i= Hydraulic conductivity after grouting.
K_o = Hydraulic conductivity before grout.
r = Equivalent tunnel/rock space radius.
t = The thickness of the grouted zone outside the tunnel.
ξ =Skin factor (3).

Using the estimated hydraulic conductivities before grouting in design reports, the flow before grouting was estimated using equation (2-2), to make judgements on the accuracy of estimating the effective hydraulic conductivities for ungrouted zones in each hydrogeological domain.

$$Q_{ungrouted} = \frac{2\,\pi\,k_0\,H\,L}{\ln(\frac{2H}{r})+\xi} \qquad (2\text{-}2)$$

At last, the RTGC method was applied on certain fans from Londonviadukten, since it is the most completed tunnel and has a large variation in geological conditions. This makes it a good choice to reflect the validity of this method with both favorable and unfavorable geological conditions. In chapter

2, RTGC method's application is explained in detail and the outcomes are shown later in the Results chapter.

2.1 Available data

- Geological, hydrogeological reports.
- Grouting classes distribution along the tunnels.
- Applications to land and environmental court for maximum permissible leakage after grouting are available for each access tunnel.
- Calculated hydraulic conductivities before grouting.
- Measured inflow measurements after grouting.
- Stop criteria and fan geometries for all grouting classes, including the specific technical solutions.
- Injection protocols, including grouting pressure, time, volume, and flow.

2.2 Given conditions and assumptions.

- The representative measured water flow after grouting is assumed to be correct.
- The hydrogeological domains of the tunnel stretches are assumed to be correct, and the uncertainty of the classifications has not been investigated.
- The grout mix change was not taken into consideration for time correction during the application of RTGC. Only change in pressure was considered.
- The grouting data obtained from the site regarding each element of the grouting process regarding workers, grout mix, and grouting equipment's, was assumed to be correct and percentage of errors was zero.
- The hydrogeological domains of the tunnel stretches are assumed to be correct, and the uncertainty of the classifications have not been investigated.

3. Literature review

3.1 History of grouting

Back to the time of ancient Greek and Roman construction, grouting was more like a trial-and-error process with no rules to be based on (Eliasson, 2012).This made the issue of "turning grouting from black magic into science" the desire of the Commission on Rock Grouting, initialized by the International Society for Rock Mechanics (ISRM) in 1996 (ISRM, 1996). The commission also states that "although increasingly successful in academic terms it (grouting) is still in its infancy in comparison with other engineering sciences" (Eliasson, 2012). According to (Bodén et. al., 2001), goal-oriented research in the grouting field has mostly started in the 1980's. Lately in Sweden, there has been a number of significant research projects in for the purpose of understanding the different aspects of grouting in hard rock. In Äspö Hard Rock Repository, several experiments and studies were performed for research purposes between 1996 and 2000. In the Stockholm area, the first major tunnels with high requirements for sealing effect constructed as a part of a larger investment in traffic tunnels were the South Link Road tunnels, which were completed in 2004, and the North Link which were constructed in 2012. Today, it can be said that grouting has reached adolescence and it is no longer an infant (Eliasson, 2012). And if these core elements become common knowledge to those involved in production, then grouting has no longer to be a matter of trial and error. Research have been thoroughly conducted to study the behavior of grout within the rock mass and the properties of grouting material.

One fact about grouting is that workers cannot see what is happening inside the fractures which makes it complicated (Houlsby, 1990). People who work with grouting must deduce what is happening regarding the flow pattern and spread of grouting inside the rock mass. Many methods were developed to assess the grouting process, but none of them was direct. Examples on these methods are radioactive tracers in the grout to follow the process from sensors in the holes nearby, grout coloring, and scanning rocks with rays. These methods did not add critical improvements for an experienced person. It is better that people who work with grout can have a deep understanding on each step in the process to make it possible to do judgements and assessments. Grouting process includes many steps and can be summarized as follows: site investigation, decision making on the necessity and extensivity of the grouting process, grouting design, contracts' arrangements, site preparations, equipment's' preparation, drilling, water testing, commencing the process, making observations, mixture reviewing, completion and assessments.

Rock mass grouting is widely used nowadays for sealing rock fractures to reduce permeability of rock mass. Mostly, it is a cement-based grout, and this process consumes both cost and time. Therefore, it is very essential to make assessment and follow up during grouting to improve the work and avoid time and financial losses. But since grouting process can be complicated, permeation and compaction grouting can both be seen in a dominated fracture grouting process. The final selected grouting modes can be affected by many factors like rock type, in-situ stresses, grout material, and grouting pressure (Zhang et al., 2014). The goal of successful grouting is to maintain sealing of fractures and fissures without allowing any ground movement caused by the applied pressure (Rafi, 2013).Thus, to fulfil requirements for this process, different empirical methods have been developed to select the grout mix properties, pumping pressure, and stop criteria to verify the sealing efficiency in both favorable and weak rock conditions.

In tunneling projects in Sweden, there are always restrictions on the maximum permissible leakage after grouting to satisfy the requirement for leakage according to the application to the land and environmental court. Therefore, continuous pre-grouting is usually performed. However, the necessity of grouting varies along the tunnel. A methodology was developed to identify the potential areas where grouting is needed. It is based on a pre-investigation program which describes the hydrogeological conditions of the area, in parallel with analyzing the "groutability'' of fractures, and requirement for leakage in the design phase. The aim of this methodology is to assess the necessity and intensity of grouting work when needed. This is called demand-assessed grouting. From previous case studies, it was found that in four studied Scandinavian tunnels (Ulvin, Hagan, Namtall and Nygård), only 15-30% of tunnel stretches needed continuous grouting. This indicates that by using demand-assessed grouting, time and costs can be saved. Most of the cases, especially in urban areas where there are high restrictions on water ingress, continuous pre-grouting is still used (Wilén et. al., 2017).

Generally, all contracts in tunneling projects in Sweden are performance contracts. Although the knowledge in grouting work has been increased, but there is still high variation in grouting design and in performance among different projects. Therefore, the Swedish Transport Agency has initiated a design approach, of what is called a unified design model approach to ensure efficient sealing in hard crystalline rock (Creütz, et al. 2017). The approach aims to have a structured methodology for grouting design, that would lead to an effective and transparent design independent of designers. The principle of this approach is to set the preliminary design categories in the design phase, assess grouting difficulty, evaluate of conditions that need specific solutions, and confirm the final grouting design to be reflected in the tender documents. This design approach is based on theories and compiled experiences. However, the approach is not yet implemented. The approach is expected to improve tendering documents and make the procurement process easier and thus reduce costs.

3.2 Grouting material and tests

Grouting can consist of different materials and mixtures based on needs and goals. Cement based grout is a mixture of cement and water with some additives and is considered be cheap and have less impact on the environment. Grout rheological and penetrability properties are determined by Rheometer and Penetrability meter tests. There are also other important tests to get better understanding of the workability and separation tendency of the grout to be done, such as Marsh cone test. Properties of grout are influenced by many factors including but not limited to; cement type, water cement ratio (w/c), chemical additives existence, mixing method and temperature (Eriksson et.al. 2003).

- **Penetrability meter test**

Penetrability tests results are dependent on the cement grain size and the apertures sizes. They are performed to determine B_{min}; which is the smallest fracture that the can be penetrated by the grout and B_{crit}; which is the smallest fracture aperture that can be penetrated without filtration as shown in Figure 3.1. Penetrability meter instrument consists of a container, attached pipe with a valve and a cap holder, mesh filters with pre-defined sizes which will be installed in the cap (Eriksson et.al. 2003) .The grout is pumped until the grout pressure is one bar, then the valve is opened. The grout that has passed the filter is collected in separated glassware's. Accordingly, and based on (Eriksson& Stille, 2003), the largest aperture of the filter is determined where no grout passes and defined as B_{min}. Different filters are used then and every time, the passing grout is separated and recorded until the measured volume is larger than the maximum, then the mesh size is defined as B_{crit}.

- **Rheometer test**

This test is performed to measure the flow trend of the suspension cement-based grout material in when forces are applied. The instrument working mechanism states that the grout is put in one cylinder inside another, where one of them rotates at a constant speed. The matter that makes it possible to determine the shear rate inside the cylinder where the grout is. However, the grout will drag the other cylinder around, which generate a force (torque) on the cylinder and thus can be measured and converted to shear stress. Data are registered by the rheometer which generates diagrams of shear stress versus shear rate. The way that can lead the experimenter to determine the rheological properties of the grout. (Rafi, 2010). The yield stress is represented by the intersection line of the scattered data with the shear stress axis, whereas the slope of the lines represents the viscosity.

According to Håkansson (1993), the time dependency of rheological properties of grout, is usually not encountered in grouting design, but still considered to be an important factor. Different measurements techniques can lead to different results regarding the yield stress and viscosity. Rheological properties of cement-based grouting were compared between in-line measurements using the Ultrasound Velocity Profiling (UPV) with Different Pressure measurements (PD) "UPV+PD'' and off-line measurements using conventional rotational rheometer test. It was found out that "UPV+PD'' is a promising tool to better determine the rheological properties of cement-based grouting and its changes with time and concentration (Håkansson and Rahman, 2013).

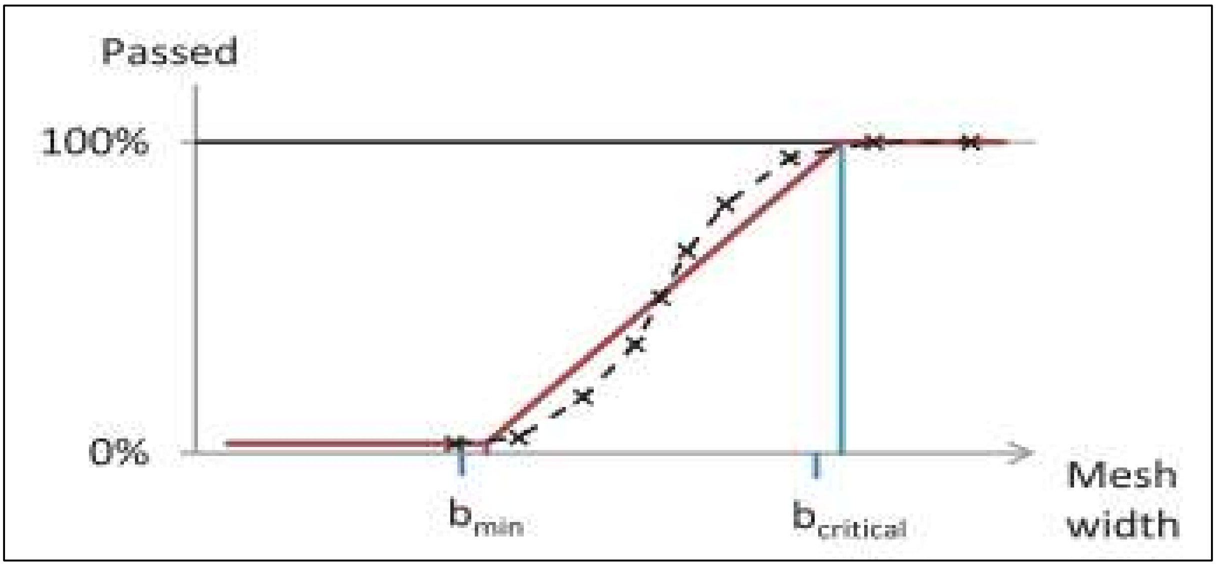


Figure 3.1. Grout penetration with respect to aperture size (Stille, 2015).

Other types of grouting are the chemical grouting such as Marithan product with high adhesive strength and outstanding mechanical properties (Zhang et al., 2014).It can be used for preparing a grout mixture that is used to inject the sections with weak conditions. It creates a bond with the rock and allows it to become intact throughout the whole project. By injecting the ground with this chemically modified grout, the low-viscosity mixture stays in liquid state for a few seconds and penetrates the fissures and fractures very easily, and start to expand, to set and to seal the weak zone. Chemically modified grout can be used to reinforce the ground, decrease the rock's permeability, and thus any water inflow problem that can occur at the site.

3.3 Grouting Theories

- Preliminary design

A conceptual model developed by Stille (2012) shown in Figure 3.2 illustrates the grouting design process. The grouting design model constitutes of three main parts; Part one: concepts related to the fractures and joints in the rock mass, which requires estimations of the hydraulic conductivity and water flow calculations. Part two: concepts related to grout spread and penetrability, which requires determination of grout rheological properties, i.e., viscosity, strength, and separation. Part three: concepts related to grout performance in sealing the fractures, i.e., stop criteria, grouting pressure, and fan design. It is essential to investigate the properties of the fractured rock mass and the permissible water flow to be able to apply this conceptual model.

Knowing the complexity of grouting design process, and that large uncertainties exist in this empirically based model, grouting design cannot only be verified mathematically. Thus, it is important to adapt observational approach during the grouting process, with a follow up to check the efficiency of the actual performance. This helps in the optimization of the grouting process.

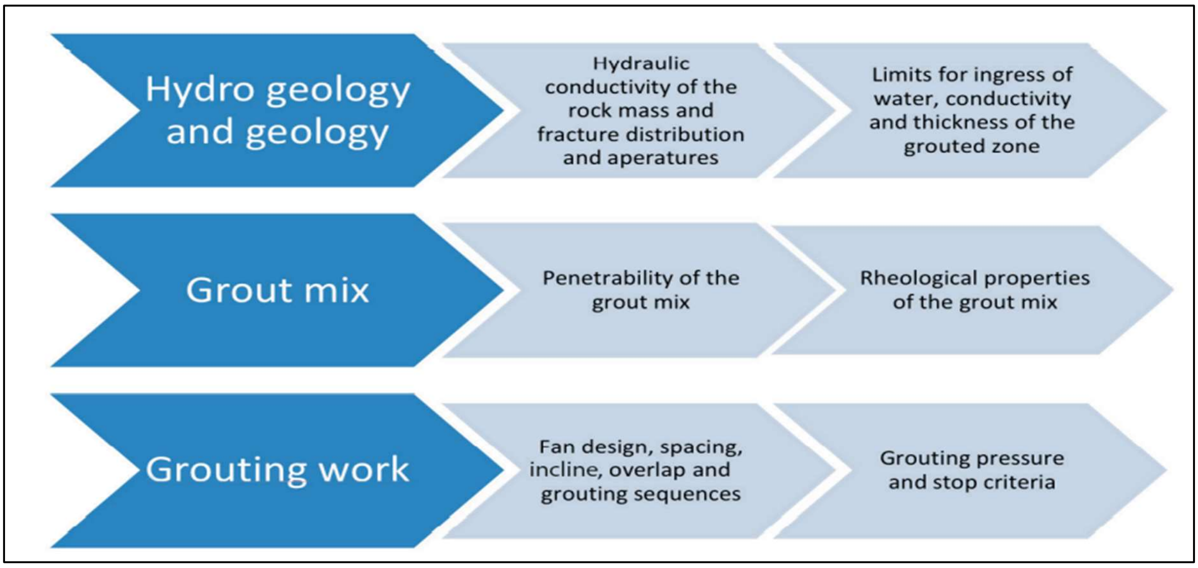


Figure 3.2. Preliminary design of grouting process (Stille,2012).

3.4 Grout penetration

The most common type of grouting is cement-based grout which is used for analysis in this report. Cement-based grout is Bingham fluid that is defined by having a viscosity μ_g and yield stress τ_0 (Kobayashi et al., 2008). Unlike Newton fluids (such as water), Bingham fluids, i.e., the grouting material, needs an initial pressure to penetrate the borehole, which has to overcome the water pressure and the yield stress of the material that will act in the opposite direction to the grout. Figure 3.3 illustrates the grout penetration at constant pressure.

The maximum penetration length can be calculated using the equation below (Hässler 1991).

$$I_{max} = \left(\frac{\Delta p}{2\tau_0}\right).b \qquad (3\text{-}1)$$

Where:

b is the aperture.
Δp is the grout excess pressure.
τ_o is the yield stress of grout.
I_{max} is the maximum grout penetration length.

According to Gustafson and Stille (2005), the grout penetration can be calculated analytically by calculating the characteristic grouting time t_0, relative grouting time t_D, and relative penetration length I_D using the following equations:

$$t_0 = \frac{6\Delta p.\mu_g}{\tau_0^2} \qquad (3\text{-}2)$$

$$t_D = \frac{t}{t_0} \qquad (3\text{-}3)$$

$$I_D = \frac{I}{I_{max}} \qquad (3\text{-}4)$$

Where:
τ_o is the yield stress of grout.
Δp is the grout excess pressure.
I_{max} is the maximum grout penetration length.
I is the grout penetration length.
I_D is the relative grout penetration length.
t_o is the characteristic grouting time.
t is the grout time.
t_D is the relative grout time.
μ_g is the viscosity of grout.

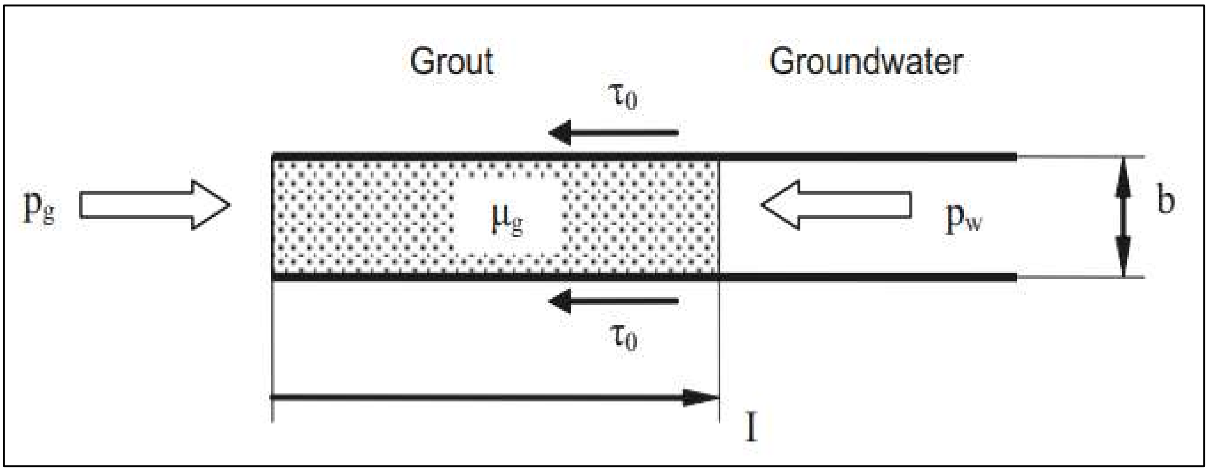


Figure 3.3. Grout penetration (Kobayashi et al., 2008).

Equations from (3-1) to (3-4) are used to calculate the relative penetration length I_D for both one dimensional and two-dimensional flow (Gustafson and Stille, 2005). The relative borehole radius (γ=I_{max}/r_b) is the ratio between maximum penetration length and borehole radius.

3.5 Grouting pressure

The grouting pressure is proportional to the grout spread in the fractures; higher the effective pressure (the grouting pressure minus the ground water pressure), higher the rate of spread (Stille, 2015). This parameter is of a great importance since it can also affect the sealing process adversely, especially when the pressure is too high and exceeds the in-situ stresses of the rock mass, where jacking can start. The grouting pressure can also cause time prolongation to achieve the required sealing efficiency if it was too low. Thus, grouting pressure needs to be established carefully first since it has impacts on other parameters, i.e., stop criteria, penetration length and sealing effect.

Three different deformations' patterns can be produced when the effective pressure is changed (Gothäll and Stille, 2008). The first pattern is when the grouting pressure is less than the initial normal stress in fractures, then small deformations can occur due to the unloading of contact points. Those deformations are small and can be negligible. The second pattern is when the grouting pressure is larger than the initial normal stress but less than the bearing capacity of the rock mass, then deformations occur and because what is called elastic jacking. However, the elastic jacking is irreversible, because the packers are retained until the grout is hardened, so it can be considered elastic just because of the elastic deformation and not because it is reversible. The last pattern, plastic jacking, which causes permanent deformation, occurs when the grouting pressure exceeds the ultimate bearing capacity of the rock mass. This leads to large permanent deformations formation and uncontrolled grouting process. Ultimate jacking can cause fractures dilations with is called hydro-jacking or might cause new fractures openings in week rock conditions which is called hydro-fracturing.

3.6 Stope criteria

Grout is injected through the borehole, and it spreads through the fractures. For each fracture, the grout penetrates a certain distance in a certain time t after the borehole filling (Gustafson and Stille, 2005). To isolate the tunnel from the rock outside and to ensure a successful grouting process for the water-bearing fractures between the boreholes, the grout penetration must bridge the distance L between the boreholes. During grouting, it is not possible to directly measure the required outcomes such as sealing of fractures and grout penetration. So, one cannot know accurately when injection of grout needs to be stopped. To ensure that the fractures are well sealed, and a proper penetration of grout has taken place, it is common in Sweden, that the grout shall continue until the stop pressure is achieved. However, it should be noted that there is no pressure in which grout cannot continue spreading at. Thus, grouting is supposed to be stopped when the flow is less than a certain value.

According to Kobayashi et al. (2008), the stop criteria is based on achieving the required grout penetration, where "Grouting is completed when the grout penetration of the smallest fracture that has to be sealed is above a certain minimum value (target value) or before the grout penetration for the largest fracture aperture reaches a certain maximum value (limiting value)''. The target value is set to be up to the maximum holes' spacing, to ensure that the space between two grout holes is tight and has no gaps for the water to flow. While the limiting value is related to the thickness of the grouted zone, where it must exceed the grouted zone with a certain distance.

3.7 Real Time Grouting Control (RTGC)

Rock science usually depends on practical experience applications combined partly with scientific methods. The reason behind this is that the rock science generally and grouting procedure specifically are full of uncertainties that are associated with many factors. Such factors are hydraulic conductivity estimation and scaling, fractures apertures, fractures distribution, and dimensionality of flow. During the grouting execution, the sealing efficiency needs to be verified with reference to certain requirements, which means that the stop grouting criteria, maximum volume, grouting time, grouting classes, and injected materials properties can be changed to meet the demands. To optimize the grouting process economically and environmentally, the grouting performance must be evaluated as it takes place, which can be done by RTGC. This means that this method relies on the input data from the site. To use this method, the grout flow and pressure must be logged in real time (Stille, 2015). More information about RTGC and its applications are described in Kobayashi et al. (2008) and Stille et al. (2009). The RTGC steps is illustrated in Figure 3.4 below. This method enables the user to check if the grout penetration length for the smallest fractures apertures is above the target value and if the penetration for the largest fracture apertures is below the limiting value (Stille, 2015).

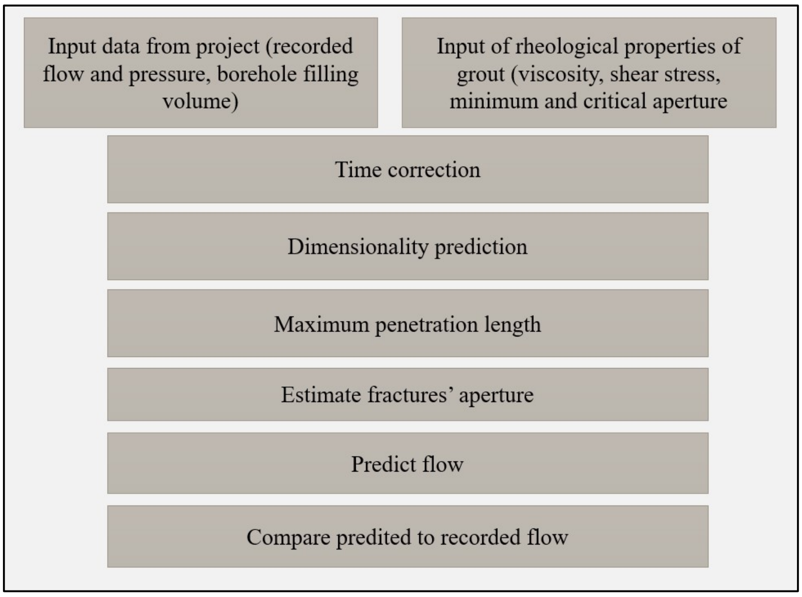


Figure 3.4. RTGC steps/layout.

- Time correction and Dimensionality prediction

Correction of time is an important step to better predict the dimensionality. Time correction considers the pressure changes and grouting mixture properties changes. For pressure changes, the corrected time is the time needed to get same spread with a higher pressure that was achieved with a lower pressure (Stille, 2015). The grouting pressure is not constant and starts from a low value and increases to the maximum. In this research time correction for pressure changes is a basic step before predicting the dimensionality. Figure 3.5 describes the simplest case of pressure changes to illustrate time correction process. Equation (3-5) is used to describe the relationship between $I_{D,Pa}$ (relative penetration under the pressure a) and $I_{D,Pb}$ (relative penetration under the pressure b).

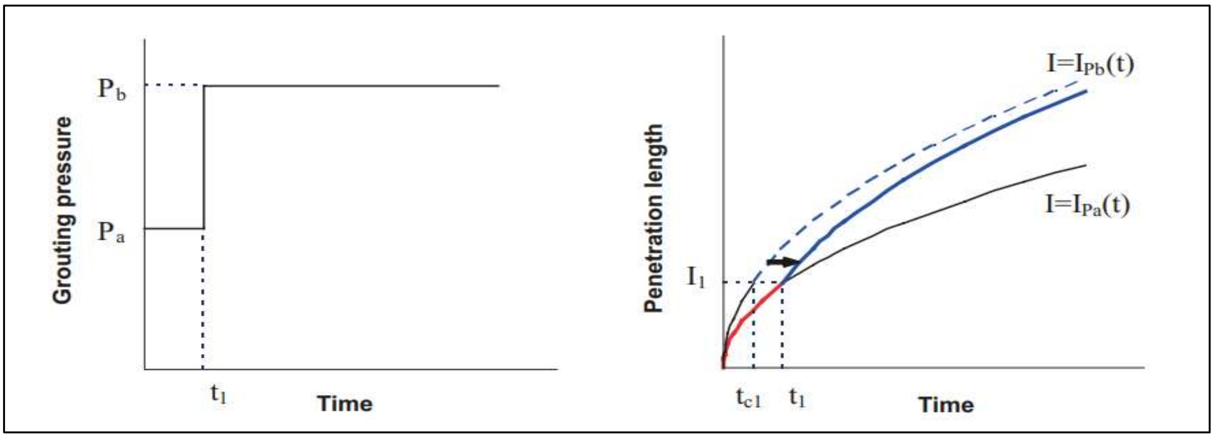


Figure 3.5. The relationship between grouting time and grouting pressure is shown in the left graph. The penetration change as the grouting pressure increases is shown in the graph to the right (Kobayashi, et al., 2008).

$I_{D,Pb} = I_{D,pa} \cdot \frac{p_a}{p_b}$ (3-5)

Where:

$I_{D,Pb}$ is the relative penetration length at pressure A.
$I_{D,Pa}$ is the relative penetration length at pressure B.
p_a is the pressure at point a (constant).
p_b is the pressure at point b (changes).

The corrected time is then calculated using equations (3-6) to (3-10) below for both 1D and 2D cases (Kobayashi et al.,2008).

$$\theta = \frac{I_D^2}{4-2I_D} \quad (3\text{-}6)$$

$$t_{1D} = \frac{1.2\ \theta_D}{1-2\theta_D} \quad (3\text{-}7)$$

$$t_{2D} = \frac{6\ \theta_D}{1-2\theta_D} \quad (3\text{-}8)$$

$$t_D = \begin{pmatrix} 10^{\ln\left(\frac{I_D}{0.7032}\right)/0.9072} & I_D < 0.4016 \\ 10^{\ln\frac{I_D-0.6266}{0.3643}} & 0.4016 < I_D < 0.7869 \\ 10^{\ln\left(\frac{1-I_D}{0.4522}\right)/(-1.7098)} & I_D > 0.7869 \end{pmatrix} \quad (3\text{-}9)$$

$$t_{corr} = t_{pb} = t_{D,pb} \cdot \frac{6p_b \cdot \mu_g}{\tau_0^2} \quad (3\text{-}10)$$

Where:

t_{corr} is the corrected time.
θ is the parameter for penetration analysis.
t_{1D} is the grouting time for 1D case.
t_{2D} is the grouting time for 2D case.
t_{pb} is the grouting time at pressure B.
$t_{D,pb}$ is the grouting time at pressure A.

Dimensionality describes the most dominant grout flow pattern. One of the ways to predict dimensionality is by plotting the index Qt_{corr}/V versus the time (Gustafson and Stille, 2005). The plot usually shows some fluctuations and sometimes needs good judgment with experience to decide on the overall dimensionality for a certain hole. Other indexes are used in the dimensionality predictions but in this research, we will only focus on Qt_{corr}/V versus the time. The dimensionality plotted is then compared with the theoretical 1D (channel flow) and 2D (disk flow) cases to better estimate the correct flow dimensionality (Stille, 2015). By plotting the theoretical d functionimensionality, it can be noticed that it ranges between 0.3-0.5 for 1D and 0.8-1.0 for 2D cases (Kobayashi et al., 2008).

- Fracture's aperture estimation and theoretical flow prediction

After dimensionality prediction and penetration length calculations, the fractures apertures are estimated and the theoretical flow is calculated using equations below (Kobayashi et al., 2008).

$$Q = \frac{dI_D}{dt_D} \cdot \frac{1}{t_0} \cdot \left(\frac{\Delta p}{2\tau_0}\right) . \sum wb^2 \qquad \text{for 1D case} \qquad (3\text{-}11)$$

$$Q = 2\pi . I_D . \frac{dI_D}{dt_D} \cdot \frac{1}{t_0} \cdot \left(\frac{\Delta p}{2\tau_0}\right)^2 . \sum b^3 \qquad \text{for 2D case} \qquad (3\text{-}12)$$

Where:

Q is the flow.

w is the width of the channel.

The two terms ($\sum wb^2$) for 1D and ($\sum b^3$) for 2D are calculated by plotting $dI_D/dt_D \cdot (1/t_0) \cdot (\Delta p/2\tau_0)$ vs. real flow and $2\pi \cdot I_D \cdot dI_D/dt_D \cdot (1/t_0) \cdot (\Delta p/2\tau_0)^2$ vs. real flow respectively. A regression line is drawn and must pass the origin, where the slope of the regression line represents the mentioned terms of ($\sum wb^2$) and ($\sum b^3$). Afterwards, the theoretical flow is calculated and compared to the real flow.

- Jacking

RTGC helps in identifying the normal and unnormal grout flow as well as jacking occurrence (Stille, 2015). In case jacking happens, the grout spread with time is reduced due to the increased fractures volume. The jacking can be noticed when dimensionality graph is plotted; especially when the graph suddenly exceeds the theoretical Qt/v or the theoretical dimensionality. By revieing the pressure/flow diagram, the jacking can be identified. Fractures' apertures are increased, accordingly sudden increase in the flow is observed with no huge changes in the pressure. Also, if the dimensionality plot suddenly deviates from the theoretical dimensionality line, then jacking might have occurred in such areas.

- Case studies and previous applications

RTGC method has been applied to dam and tunnel projects. Details on the application of the theory can be found in Stille et al. (2009) which show the application on three projects to demonstrate the validity of this proposed new method. The three projects have been taken from different parts of Sweden with different geological properties, where it was also recorded all the required data to apply the RTGC to analyze the dimensionality, fractures apertures and the theoretical flow. The results showed that the grout flow had so far followed the theories of RTGC. (Stille, 2015)

Another case was performed by Kobayashi et al. (2008) to verify the method by using Äspö HRL data (Hard Rock Laboratory) at the 450 m level. The research had showed that the calculated measurables such as the flow dimensionality, fractures' apertures and grout flows were quite close to the measured ones, which indicates that the ''RTGC'' might be applicable to similar situations in real grouting design. (Kobayashi et al., 2008).

This method was also used by Jalaleddin Rafi to demonstrate its validity in a case study in sedimentary rock with Gotvand Dam data (Rafi, 2010). The results from this case study's research showed that RTGC provides quite good results and can be applicable in similar projects as well. The results showed that this method can support the design process and give information on the grout mix spread pattern or trend in real time, which helps in identifying practical timing for the process and provides more understanding on the grout recipes and its corresponding behavior.

However, there are still limitations and difficulties to apply this method on three-dimensional flow since it was developed based on 1D and 2D flow patterns. Thus, in rock massed with high porosity, the application of this method will be difficult and not yet fully verified or developed.

4. Project description

4.1 Project location

In this work, the study area is the expansion of the Blue Metro Line in Stockholm, which has started in the early 2020 and is expected to be finished in 2030. The new blue line will connect Kungsträdgården to Nacka and Söderort via Södermalm as shown in Figure 4.1. The dashed blue line in the figure shows the extension from Kunsträdgården east to Nacka and south until Sockenplan. Total length of the extension line is 11.5 Km to be built totally under the ground. The project includes building 5 new stations at Sofia, Hammarby Kanal, Sickla, Järla and Nacka, and a new platform in Gullmarsplan will be built under the existing green line. The new stations will form important hubs for Stockholm transportation and will create more travel opportunities with shorter time between different areas.

In this research, 6 access tunnels were analyzed from different aspects. The access tunnels in this study are highlighted in red and shown in Figure 4.2.

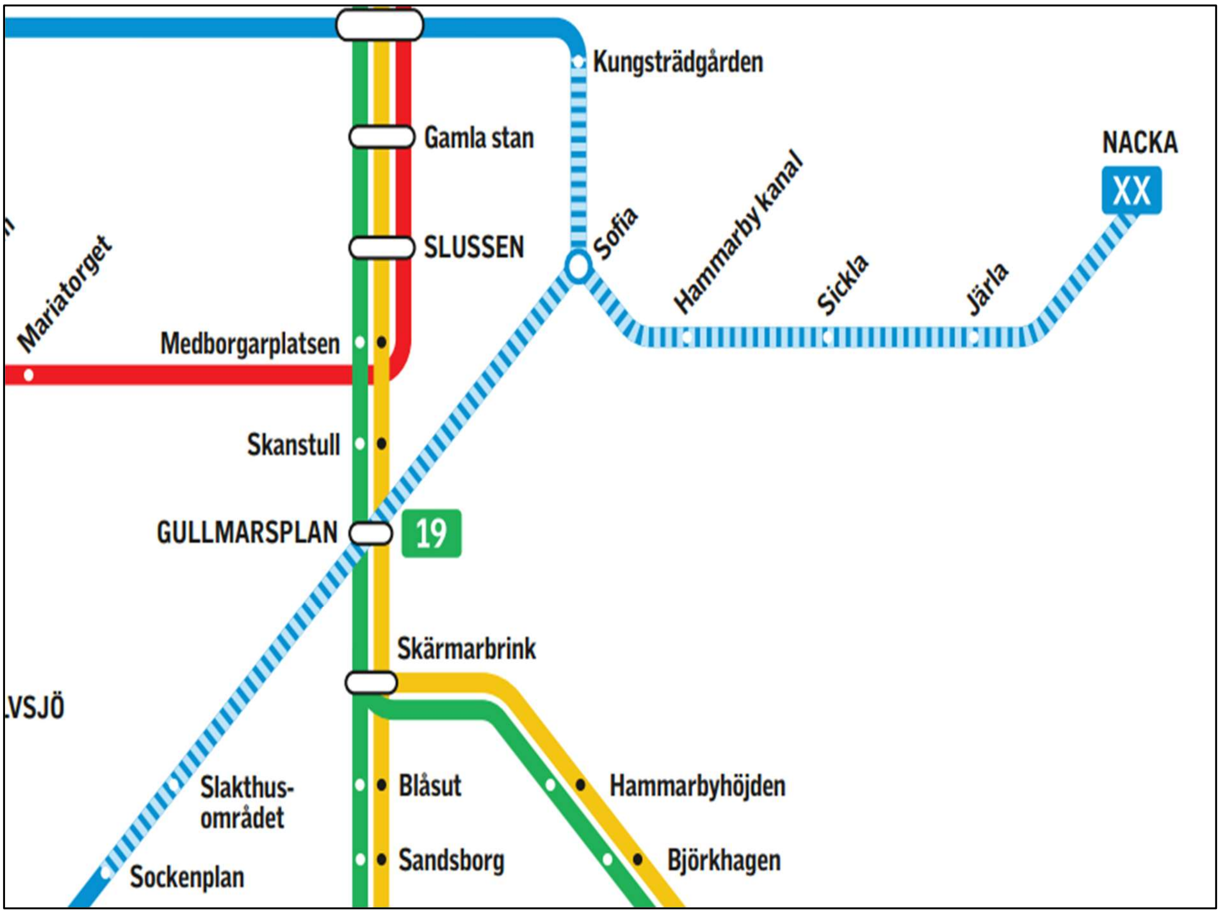


Figure 4.1. Extension of blue metro line.

Figure 4.2. The evaluated six access tunnel's locations (highlighted in red).

The region Stockholm /SLL is an organization that consists of many parts. One part consists of political bodies in different areas of activity and the other one is a management organization (förvaltningar) in different working areas to assists the politicians in their work and ensure that taken decisions are implemented. One of the administration parties is, FUT (Förvaltning för utbyggd tunnelbana) that is responsible on the expansion of the metro line.

The FUT (Förvaltning för utbyggd tunnelbana) is the administration department that has the overall responsibility of the extension project, including planning, designing, and construction of the new metro lines and stations. A joint venture of SWECO/TYPSA was tasked to produce plans and technical designs as part of this 25.7 billion SEK project to extend the Stockholm Metro. The tasks related to the joint venture include building seven new stations. The project is managed using a BIM process. Since the project contains challenges, such as the land layout and the depth of ground depression, it was preceded with much analysis. The project includes international team from different SWECO offices in Sweden, Finland, and Poland with TYPSA company from Spain. Six main blocks constitute a significant part of the project which are geotechnics and mechanics of rock, track systems, networks and appliances, architecture and constructions and the environmental protection.

4.2 Leakage requirements

4.2.1 Application for permit and final permit regarding groundwater inflow

Since the variation in ground water level can affect the properties of third parties, it is important to perform the grouting process to reduce hydraulic conductivity of the rock mass and yet preserve the ground water level. Ground water level is changed due to leakage, which means that such projects need permissions from the land and environmental court according to the environmental act in Sweden. In the application for permit, Region Stockholm suggested a maximum allowed groundwater inflow divided along the planned tunnel line. Maximum levels of groundwater inflow to the tunnels along these stretches was then set in the verdict from the court. The actual permission regarding maximum inflow of groundwater is although lower in the final permit than in the application.

4.2.2 Requirements regarding maximum leakage used for the evaluation of grouting design

In this project, the detail design of grouting was performed to satisfy the requirement of leakage based on the application to the land and environmental court. The requirements for leakage according to the application to the court has been translated into maximum permissible groundwater inflow measurements into the tunnel. The requirements for leakage are managed by control values that represent 90% of the maximum allowable groundwater inflow into the tunnel, which provides better safety margin to prevent negative effects on the surrounding environment.

In this thesis a comparison has been made between the measured leakage and the requirements for maximum leakage in the detail design (same as in the application to the court). No comparison has been made to the allowed maximum leakage in the permit or the detailed prognosis values for the tunnels, which originates from the permit. The purpose is to investigate how the previous requirements for the detail design has been fulfilled with the grouting done in the tunnels.

Figure 4.3 below shows the control values for each access tunnel based on the design documents.

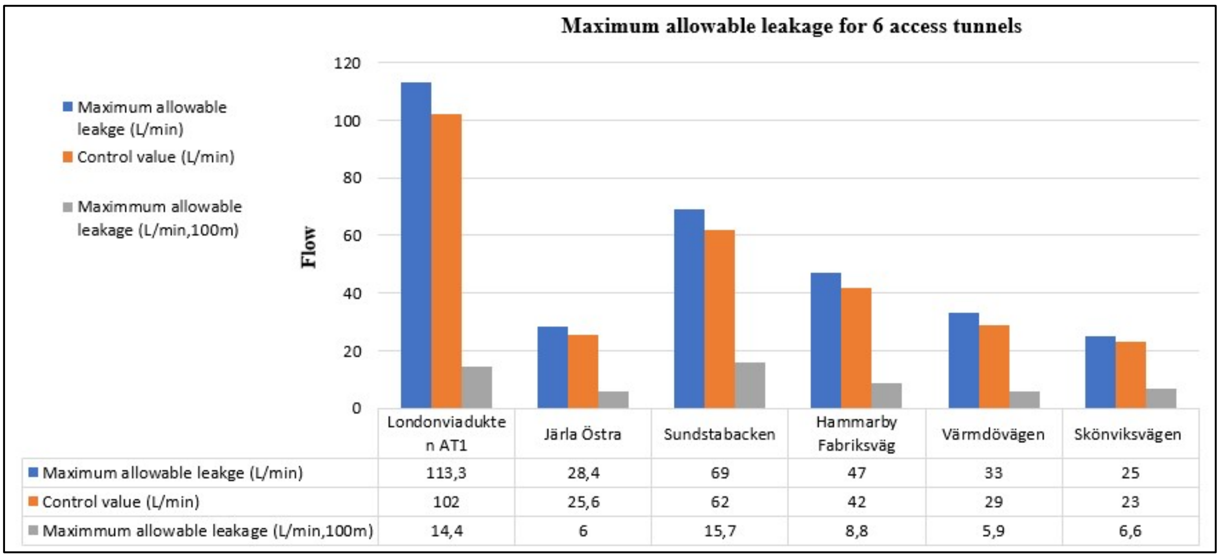


Figure 4.3. Summary of control and maximum allowable leakage for each access tunnel.

Using Table 4.1 below, the difficulty of grouting can be identified for each access tunnel (Remiss-Projektering av förinjektering för bergtunnlar, Trafikverket, version 170404).

Classification is made according to:

- Low requirements,> 0.25 l / min, 100 m tunnel and depth, are marked in green.
- Moderate requirements, 0.15-0.25 l / min, 100 m tunnel and depth. Marked in yellow.
- High requirements, 0.10-0.15 l / min, 100 m tunnel and depth. Marked in orange.
- Very high requirements, <0.10 l / min, 100 m tunnel and depth. Marked in red.

The table describes the difficulty degree of grouting in relation with the leakage requirements per 100m and tunnel depth. In Londonviadukten and Sundstabacken, the maximum allowable leakage according to the application to the environment court are 14.4 and 15.7 L/min in100m respectively as shown in Figure 4.3 above. Therefore, the difficulty of grout in these two tunnels is ranging from low to high, depending on tunnel depths. Whereas, in the other access tunnels, i.e., Järla Östra, Hammarby Fabriksväg, Värmdövägen and Skönviksvägen, the leakage requirements according to the application to the environment court were lower and around 6-9 L/min in100m. This shows that there is high possibility to end up with many tunnel fans that have very high difficulty degree of grouting, knowing that higher the depth and lower the allowable leakage means higher the difficulty degree of grout.

Table 4.1. Difficulty degree of grouting (Remiss-Projektering av förinjektering för bergtunnlar, Trafikverket, version 170404).

	Inläckagekrav [l/min, 100 m tunnel]						
Djup [m]	15	12	10	8	6	4	2
10	1,50	1,20	1,00	0,80	0,60	0,40	0,20
20	0,75	0,60	0,50	0,40	0,30	0,20	0,10
30	0,50	0,40	0,33	0,27	0,20	0,13	0,07
40	0,38	0,30	0,25	0,20	0,15	0,10	0,05
50	0,30	0,24	0,20	0,16	0,12	0,08	0,04
60	0,25	0,20	0,17	0,13	0,10	0,07	0,03
70	0,21	0,17	0,14	0,11	0,09	0,06	0,03
80	0,19	0,15	0,13	0,10	0,08	0,05	0,03
90	0,17	0,13	0,11	0,09	0,07	0,04	0,02
100	0,15	0,12	0,10	0,08	0,06	0,04	0,02
110	0,14	0,11	0,09	0,07	0,05	0,04	0,02
120	0,13	0,10	0,08	0,07	0,05	0,03	0,02
130	0,12	0,09	0,08	0,06	0,05	0,03	0,02
140	0,11	0,09	0,07	0,06	0,04	0,03	0,01
150	0,10	0,08	0,07	0,05	0,04	0,03	0,01

4.2.3 Requirements regarding maximum leakage used for the execution and steering of grouting work

As described earlier, the actual permission regarding maximum inflow of groundwater is lower in the final permit than in the application to the court. Region Stockholm will work according to the observation method in Eurocode 7 to fulfil the requirements in the permit. Therefore, the grouting in the tunnels is done and evaluated compared to certain prognosis values of leakage. These prognosis values are based on the maximum allowed groundwater inflow along each tunnel stretch in the permit, with a breakdown in detailed parts of each tunnel. A total inflow budget for each stretch with maximum allowed inflow in the verdict is thus distributed into prognosis values for the construction time and the operating time for the new metro. For example, a prognosis of the maximum inflow of groundwater along each access tunnel has been set. The prognosis values work also as alarm values for the contractors and designers to evaluate the leakage into the tunnel and to make the necessary decisions regarding the need for further grouting.

4.3 Geological Prognosis

In this project, the RMR_{base} system was used to classify the rock mass from the good to poor rock quality (A-D). According to (Bieniawski ,1989), RMR classification system, is based on investigating 6 parameters and rating each one as a number. These numbers are then summed up for each zone to give the RMR value, which aims to classify the type of rock mass into classes going from excellent rock to very poor rock. The evaluation gives a number up to 100, which represents a rock mass with excellent quality. In the characterization process, which is only related to rock mass features, RMR_{base} value was obtained by summing all the rating numbers but setting ground water conditions to completely dry. RMR value is the RMR_{base} value subtracting the rating number for discontinuity orientation based on given data. The 6 parameters are: uniaxial compressive strength of intact rock, spacing of discontinuities, condition of discontinuities (including, roughness, separation, weathering, and infillings), groundwater conditions, orientation of discontinuities, and Rock Quality Designation (RQD).

According to (Deere, 1963), RQD is used as a tool for a simple classification of the rock mass stability. Five rock classes which represents the rock mass quality ranging between very poor, poor, fair, good, and excellent. RQD is defined as the sum of pieces of a core drilling that are more than 10 cm over the total core length. Thus, RQD is a percentage ranging between 0 and 100.

$$RQD = \frac{\sum length\ of\ specimens > 10\ cm}{Total\ length\ of\ core\ drill\ (1\ m)} \qquad (4\text{-}1)$$

Each of the access tunnels was classified according to rock mass quality into four classes: Class A, B, C and D. The characterization was made using RMR_{base} system. Table 4.2 below shows the RMR_{base} vs. each class.

Table 4.2.Classification of rock class based on RMR_{base} system.

RMR_{base}	Rock Class
$75 \leq RMR_{base} < 100$	A
$60 \leq RMR_{base} < 75$	B
$45 \leq RMR_{base} < 60$	C
$RMR_{base} < 45$	D

The quality of the rock is one of the factors that must be considered before selecting the geotechnical category. Design requirements such as carrying capacity, stability, durability, and water tightness are highly dependent on the appropriate application of conventional reinforcement and sealing methods depending on the nature and quality of rock mass classified in early stages.

A summary of the geological conditions for each access tunnel is described below.

- **Londonviadukten**

Londonviadukten tunnel starts in an area near the intersection of Stadsgårdsleden and Folkungagatan. The access tunnel is separated into two branches after 270m from the access tunnel start. AT1 is the first branch and continues south under Yourstagatan The second branch is AT2 which breaks off to the west under Folkungagatan.

For AT1 branch, cores (15S0543, 15S0544, 17S0547) and surface mapping performed north of Folkungatan were performed as shown in Figure 4.3 below. the dominant rocks mapped mainly are mainly grey fine to coarse-grained granite, gneiss granite and sediment gneiss. The dominant rock mass in the area at Folkungatan and north of Folkungatan (AT1 0+036 km –about 0+320 km) were mapped with RMR_{base} values mainly ranging between 60-80.

Almost 83% of the core 15S0543, shows an RMR_{base} values ≥60., And approximately 87% and approximately 75% of the cores 15S0544 and 17S0547 respectively show an RMR_{base} values ≥60. For cores 15S0543 and 15S0544, the average crack frequency (excluding weakness zones) is about 4 cracks per meter. While core 17S0547 is interpreted with an average crack rate of about 7 cracks per meter. Based on an obvious increase in crack frequency, weakness zones occur in cores 15S0543 and 15S0544, between drill length 53-60 m in 15S0543, between 64-68 m in 15S0544, and between 85-88 m in 17S0574. The weakness zone has RMR_{base} values between 33-63 and an average RMR_{base} of about 50. Throughout the cores, there are crack zones in all three cores, with increased crack frequency, where the average RMR_{base} is about 55 for the crack zones.

Figure 4.4. Londonviadukten access tunnel marked in red, and the core boreholes location is indicated in blue (FUT,2019).

The average RMR_{base} varies between 75-78 in mapped cores 15S0545 and 15S0546.While almost 61% of the core 15S0545 shows $RMR_{base} \geq 75$, and approximately 80% of the drill core 15S0456 gives an $RMR_{base} \geq 75$. Table 4.3 shows the rock quality classification per tunnel stretch. The average crack frequency of both cores is 3-4 cracks per meter. The rock mass is estimated to have a RMR_{base} value of 79 and an average crack rate of 3 cracks per meter, based on results from mapping of cuts and rock areas. For AT2 branch, the crack frequency is increased with an average of 15 cracks per meter for core 15S0252 and about 7 cracks per meter for core zones 15S0253.

Table 4.3.Classification of rock classes for Londonviadukten access stunnel based on RMR_{base} system.

Tunnel section (Km)	RMR_{base}	Rock Class
0+35 to 0+285	$60 \leq RMR_{base} < 75$	B
0+285 to 0+320	$45 \leq RMR_{base} < 60$	C
0+320 to 0+360	$60 \leq RMR_{base} < 75$	B
0+360 to 0+730	$75 \leq RMR_{base} < 100$	A
0+730 to 0+821	$60 \leq RMR_{base} < 75$	B

- **Järla Östra**

Heterogeneous, banded, and foiled sediment gneiss (fine-grained to medium-grained) is dominating in the rock mass of the entire working tunnel at the Järla Östra area. As well as some granites (medium grain to coarse-grained) both subparallel with and cutting on the strapping/foiling. The survey includes three mapped core boreholes (16S0855, 16S0857 and 16S0859) see Figure 4.4, seventeen-line mappings and a surface mapping that was carried out at the work tunnel's planned addition north of Järla Östra skolväg. There are places in all core boreholes where the rock mass show oxidation (red staining), exhibits an increase in crack frequency, and the presence of clay as crack filling material. This led to the conclusion that there is a possibility that such rocks may contain a fault zone.

A minor slump exists in the rock surface that runs NNW-SSE along Birkavägen, which crosses the tunnel between 0+230 and 0+270. It concurs with batches of slightly cracked and oxidized (red-colored) rock. In the upper part of two core drills (KBH16S0857 and KBH16S0859) a slightly lower rock quality than the other rocks in the surrounding area. There is a demarcation of the rock area about 90 m to the west that lies subparallel to Birkavägen. It crosses the working tunnel between 0+346 – 0+376. This batch made up of weak rock coincides as well with batches of slightly cracked and oxidized (red-colored) rock. A slightly lower rock quality than other rocks in the surrounding area are found at the lower part of two core boreholes (KBH16S0855 and KBH16S0857).

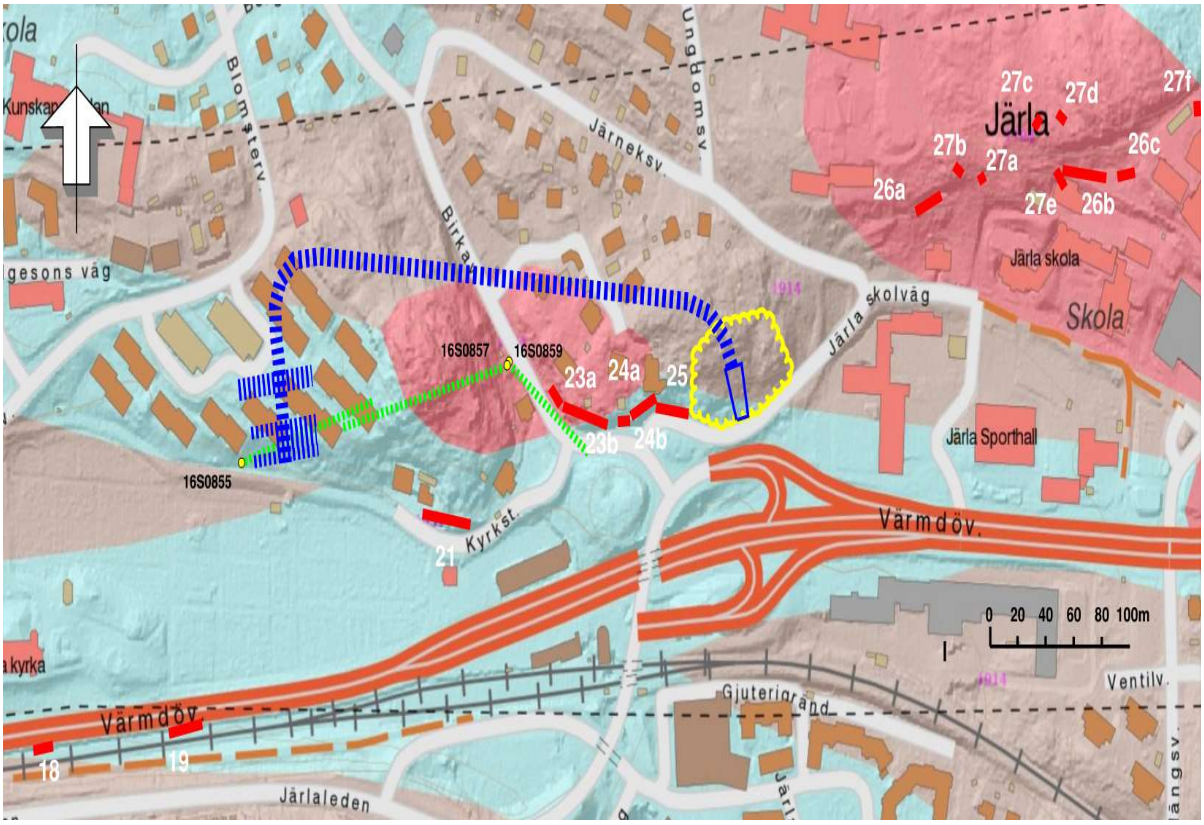


Figure 4.5. Järla Östra access tunnel is shown as a blue dashed line, and the core boreholes location is indicated by the green dotted lines with a yellow ring which mark the core boreholes 16S0855, 16S0857 and 16S0859 (FUT,2019).

It is estimated that the rock quality along the tunnel section varies between 60 and 80, which lead to the conclusion that the rock mass is mainly classified as good rock with class A (75≤ RMR_{base} <100) and rock class B (60≤ RMR_{base}<75).At the oxidized fault zones (between 0+232 – 0+272 and 0+346 – 0+376) the rock quality drops slightly to RMR_{base} about 45-70 ,which gives a rock classified between rock class C (45≤ RMR_{base} <60) and rock class B. Table 4.4 below shows the rock class quality along each tunnel stretch. Average crack frequency of approximately 4–5 cracks per meter is estimated for the rock mass based on results from mapping of rock cuttings and rock areas. For all three drill cores the average crack frequency is from 4 (KBH16S0855 and KBH16S0857) to 5 (Kbh16S0859) cracks per meter. The uniaxial compressive strength of the rock mass is estimated to vary from about 70 to 280 MPa for mapped rocks in the area.

Table 4.4.Classification of rock classes for Järla Östra access tunnel based on RMR_{base} system.

Tunnel section (Km)	RMR_{base}	Rock Class
0+20 to 0+52	60≤ RMR_{base}<75	B
0+52 to 0+241	60≤ RMR_{base}<75	B
0+241 to 0+267	45≤ RMR_{base}<60	C
0+267 to 0+287	60≤ RMR_{base}<75	B
0+287 to 0+365	75≤ RMR_{base}<100	A
0+365 to 0+383	60≤ RMR_{base}<75	B
0+383 to 0+394	45≤ RMR_{base}<60	C
0+394 to 0+408	60≤ RMR_{base}<75	B
0+408 to 0+488	75≤ RMR_{base}<100	A
0+488 to 0+495	60≤ RMR_{base}<75	B

- **Sundstabacken**

The mapped values for RMR_{base} showed that the dominating rock mass in the area around Sundstabacken was rock class A and B except in weakness zones where it was rock class C and D. In the eastern part of the area 5 boreholes were located, where three of them (15S1710, 15S1711 and 15S1712) are assessed to hit a weakness zone as shown in Figure 4.6 below. After observing the area around the work tunnel, it was found that the mean value of RMR_{base} in the mapped drill cores 15S1710, 15S1711 and 15S1712 was 70, 61 and 66, respectively. Whereas in drill cores 15S1713 and 15S1714 the mean value of RMR_{base} was 69 and 75, respectively. In Table 4.5 below the rock classes for each section of this tunnel are shown. For boreholes 15S1710, 15S1711 and 15S1712 the mean crack frequency varies slightly between 7-12, while 15S1713 has a crack frequency of 9 and 15S1714 has 5 cracks per meter.

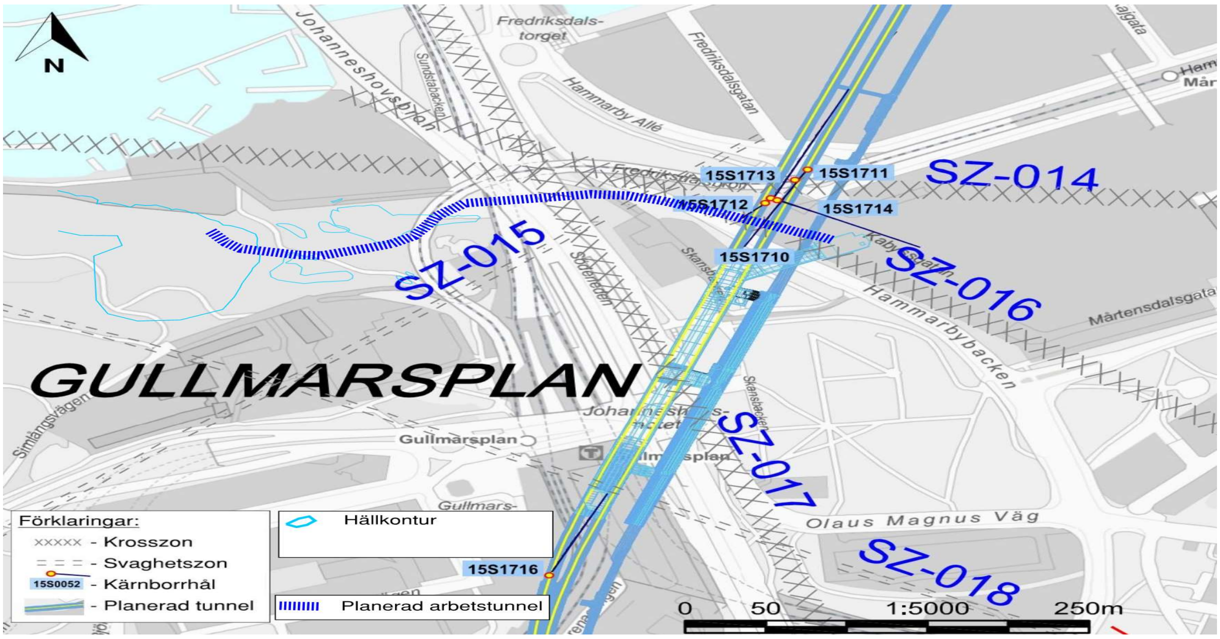


Figure 4.6. Sundstabacken access tunnel is shown as a dashed blue line, and the core boreholes location is indicated by the yellow rings (FUT,2019).

It was estimated that the rock mass had a typical value for RMR_{base} values of approximately 70 (± 10), and an average crack frequency of approximately 2–4 cracks per meter, based on results from mapping of the rock area west of Skanstullsbron.

The uniaxial compressive strength of the intact rock greatly varied between 60 to 155 MPa, based on estimated values in mapped drill cores.

Table 4.5.Classification of rock classes for Sundstabacken access tunnel based on RMR_{base} system.

Tunnel section (Km)	RMR_{base}	Rock Class
0+22 to 0+48	$60 \leq RMR_{base} < 75$	B
0+48 to 0+120	$60 \leq RMR_{base} < 75$	B
0+120 to 0+240	$75 \leq RMR_{base} < 100$	A
0+240 to 0+284	$60 \leq RMR_{base} < 75$	B
0+284 to 0+294	$45 \leq RMR_{base} < 60$	C
0+294 to 0+336	$45 < RMR_{base}$	D
0+336 to 0+408	$45 \leq RMR_{base} < 60$	C
0+408 to 0+461	$60 \leq RMR_{base} < 75$	B

- **Hammarby Fabriksväg**

According to the design and geological reports, sedimentary and both younger and older granite are the dominant types of rocks. In the weakness zones the rock mass is characterized by high crack frequency, weathering to graphite and clay, as well as brecciated parts. In the rock mass sulfide mineralization occurs as crack filling. In 2016, core drilling was performed. Mapping of cores was carried out in August 2016. Geographical location of completed core boreholes is reported in Figure 4.7.

Approximately 97% of the core 15S0181 shows RMR_{base}values≥60, and 61% shows RMR_{base}values≥75. Five cracks per meter is the average frequency of core 15S0181. There is a fault zone in core 15S0181, interpreted by the observation of an increase in the crack frequency, between core length about 44.5 m and about 45m. The fault zone has been assessed by showing an RMR_{base} <60. According to results from mapping, the rock mass is estimated to have a RMR_{base} of 79 and an average crack frequency of about 3 cracks per meter. The crack endurance varies between 1.5 m and 20 m for all crack groups.
For the mapped rocks in the area, the uniaxial compressive strength of the intact rock ranges from 110 to 215 MPa. Whereas the uniaxial compressive strength in weakness zones in the area is considered to vary but it is less than ≤70 MPa.

In the rock surface model, there is a slump area interpreted as fault zone, which has an orientation in an ESE-WNW direction. This zone is the rock mass between 0+220 and 0+380. At this tunnel stretch the rock mass quality varies but it is generally considered to be good. In Table 4.6 below the rock classes for each section of this tunnel are shown.

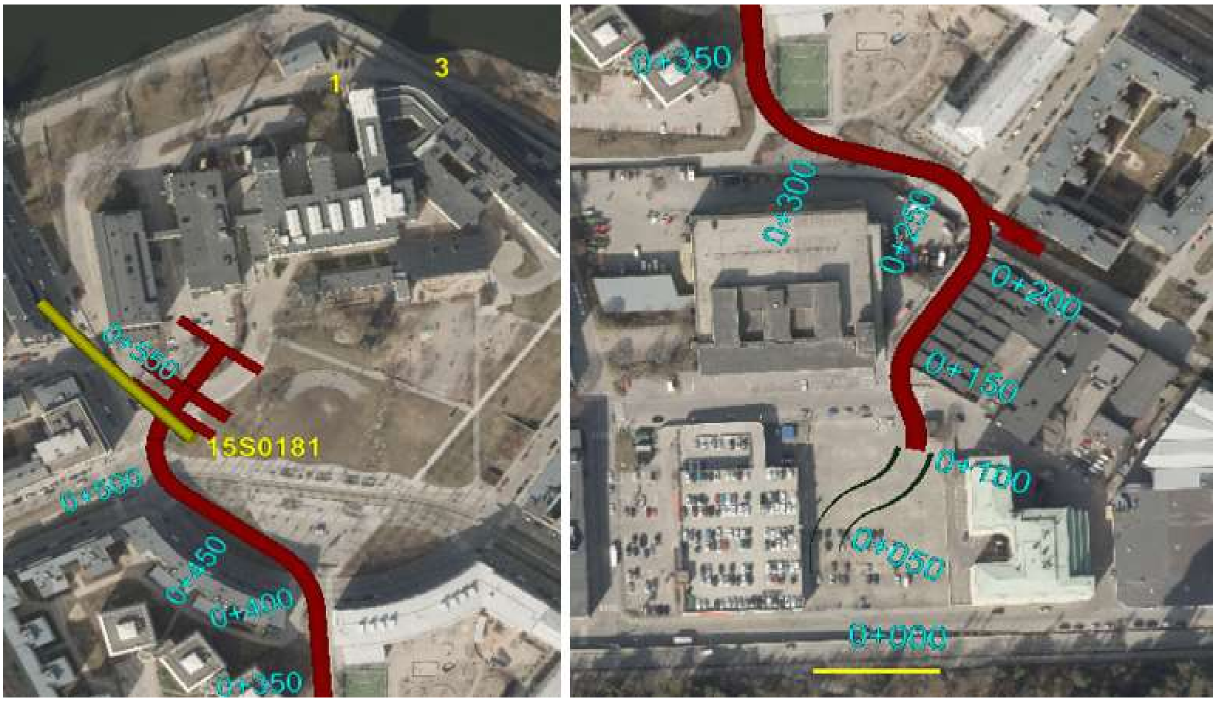


Figure 4.7. Hammarby Fabriksväg access tunnel marked in red, and the completed core borehole’s location around the tunnel (FUT,2020).

Table 4.6.Classification of rock classes for Hammarby Fabriksväg access tunnel based on RMR_{base} system.

Tunnel section (Km)	RMR_{base}	Rock Class
0+103 to 0+220	$60 \le RMR_{base} < 75$	B
0+220 to 0+380	$45 \le RMR_{base} < 60$	C
0+380 to 0+450	$60 \le RMR_{base} < 75$	B
0+450 to 0+578	$75 \le RMR_{base} < 100$	A

- **Värmdovägen**

The dominating rock mass over the entire working tunnel is heterogeneous, foiled, and banded sediment gneiss (fine-grained to medium-grained), with hints of granites (medium grain to coarse grain). As a basis for the description of the structural geology in the area, eleven-line mappings (lines 9a, 9b, 10, 11, 13a, 13b, 13d, 13e, 13f, 13g and 14), mapping from tunnel walls and ceilings in the entrance tunnel to the mountain room north of Värmdövägen with connection to the planned impact, and mappings from two core boreholes (16S0774 and 16S07745) are used. The other two boreholes 15S0951 and 15S0957 cannot be used for directions since they are not oriented, however it can be used for crack properties data as shown in **Error! Reference source not found.**

Based on the mapped values of RMR_{base} in the area around Värmdövägen,it is observed that it is dominated by a rock mass of class B, as shown in Table 4.7.

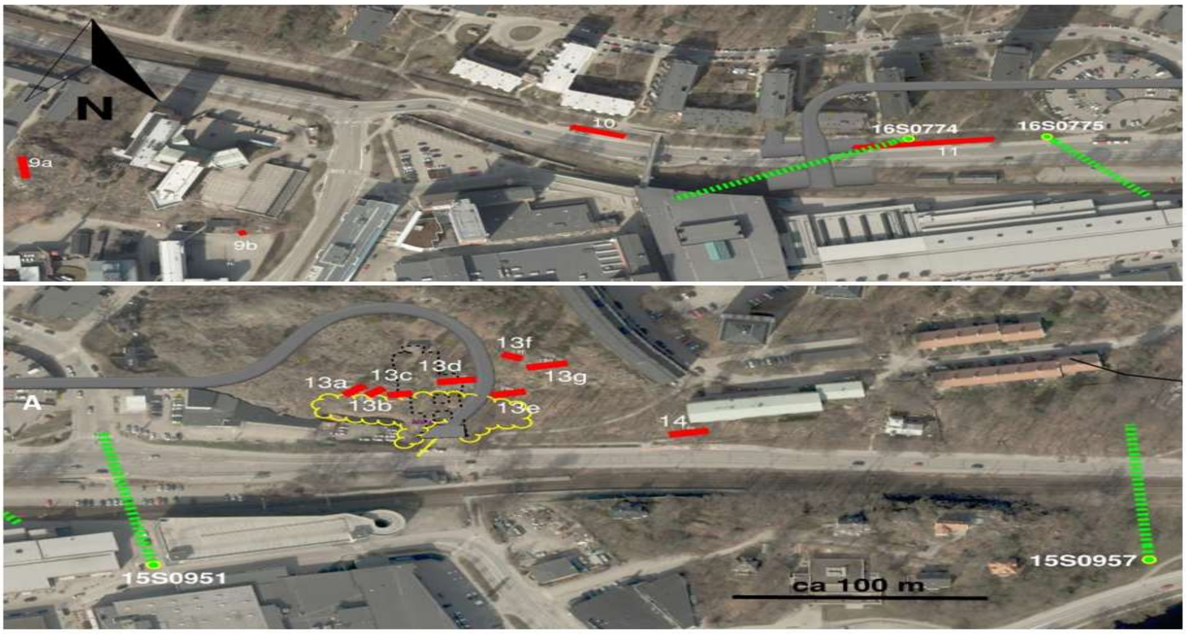


Figure 4.8. Core drillings, line mappings around Värmdovägen access tunnel (FUT,2018).

Table 4.7. Classification of rock classes for Värmdovägen access tunnel based on RMR_{base} system.

Tunnel section (Km)	RMR_{base}	Rock Class
0+17 to 0+27.5	$60 \leq RMR_{base} < 75$	B
0+27.5 to 0+100	$60 \leq RMR_{base} < 75$	B
0+100 to 0+227	$75 \leq RMR_{base} < 100$	A
0+227 to 0+237	$60 \leq RMR_{base} < 75$	B
0+237 to 0+253	$45 \leq RMR_{base} < 60$	C
0+253 to 0+575	$60 \leq RMR_{base} < 75$	B

For all four cores, the average crack frequency is from 5 (KBH16S0775) to 8 (KBH15S0951) cracks per meter. Neither in core mappings nor in line mappings the length of the cracks is included in but depending on results from mapping of intersections and rock areas where cracks are longer than 3 m, it is estimated that the rock mass have a value for RMR_{base} of about 70-80 and an average crack rate of about 2 to 4 cracks per meter. For mapped rocks in the area, the uniaxial compressive strength of the rock mass is estimated to vary from about 70 to 280 Mpa.

- **Skönviksvägen**

The dominating rock mass over the entire working tunnel is heterogeneous, foiled, and banded sediment gneiss (fine-grained to medium-grained), with variable amounts of granites (medium grain to coarse grain). As a basis for the description of the structural geology in the area, line mappings (lines 31, 32, 24a, 34b and 35) and mappings from two core boreholes (15S1142 and 15S1143) are used as shown in Figure 4.9.

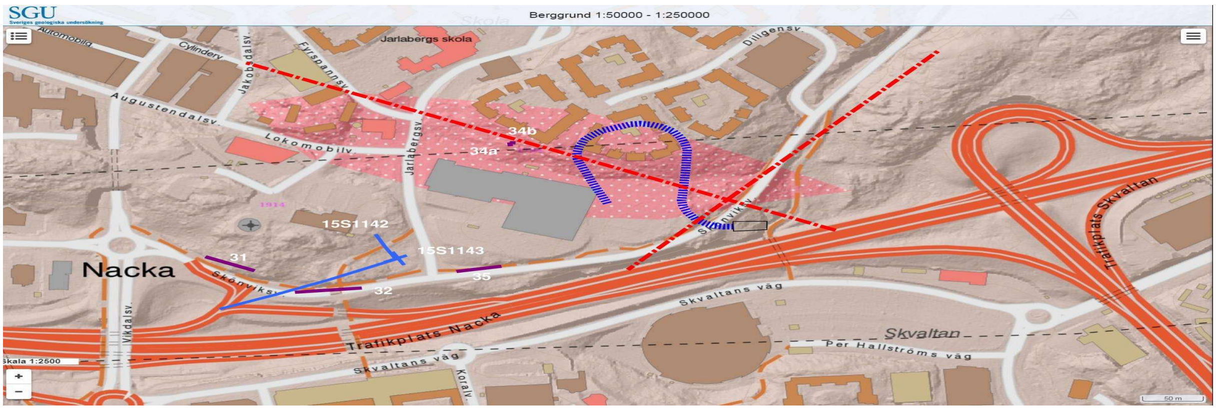


Figure 4.9. Core drillings, line mappings around Skönviksvägen access tunnel (FUT,2019).

It is estimated that the rock quality along the tunnel section is mainly classified as good rock with class A (75≤ RMR_{base} <100). But this quality is assumed to drop slightly to rock class B (60≤ RMR_{base} <75) at the point where the cross of the liniment is expected (between 0+060-0+100, about 0+100 – 0+160 and about 0+310-0+370) as shown in Table 4.8 below. For both cores the average crack is 5 cracks per meter. The rock mass is estimated to have a value of about 80 for RMR_{base} based on results from mapping cuts and rock areas. As well as an and an average crack frequency of about 2 cracks per meter.

It is also estimated that for mapped rocks in the area, the uniaxial compressive strength of the rock mass ranges from about 70 to 280 MPa.

Table 4.8.Classification of rock classes for Skönviksvägen access tunnel based on RMR_{base} system.

Tunnel section (Km)	RMR_{base}	Rock Class
0+30 to 0+50	60≤ RMR_{base}<75	B
0+50 to 0+80	60≤ RMR_{base}<75	B
0+80 to 0+85	45≤ RMR_{base}<60	C
0+85 to 0+95	60≤ RMR_{base}<75	B
0+95 to 0+121	75≤ RMR_{base}<100	A
0+121 to 0+131	60≤ RMR_{base}<75	B
0+131 to 0+140	45≤ RMR_{base}<60	C
0+140 to 0+150	60≤ RMR_{base}<75	B
0+150 to 0+321	75≤ RMR_{base}<100	A
0+321 to 0+331	60≤ RMR_{base}<75	B
0+331 to 0+342	45≤ RMR_{base}<60	C
0+342 to 0+384	60≤ RMR_{base}<75	B
0+384 to 0+409	75≤ RMR_{base}<100	A

4.4 Summary of Geological prognosis

Based on design documents, the rock mass quality along the 6 access tunnels were identified for each tunnel stretch. The percentage of each rock class was calculated by the authors and presented in Figure 4.10 below.

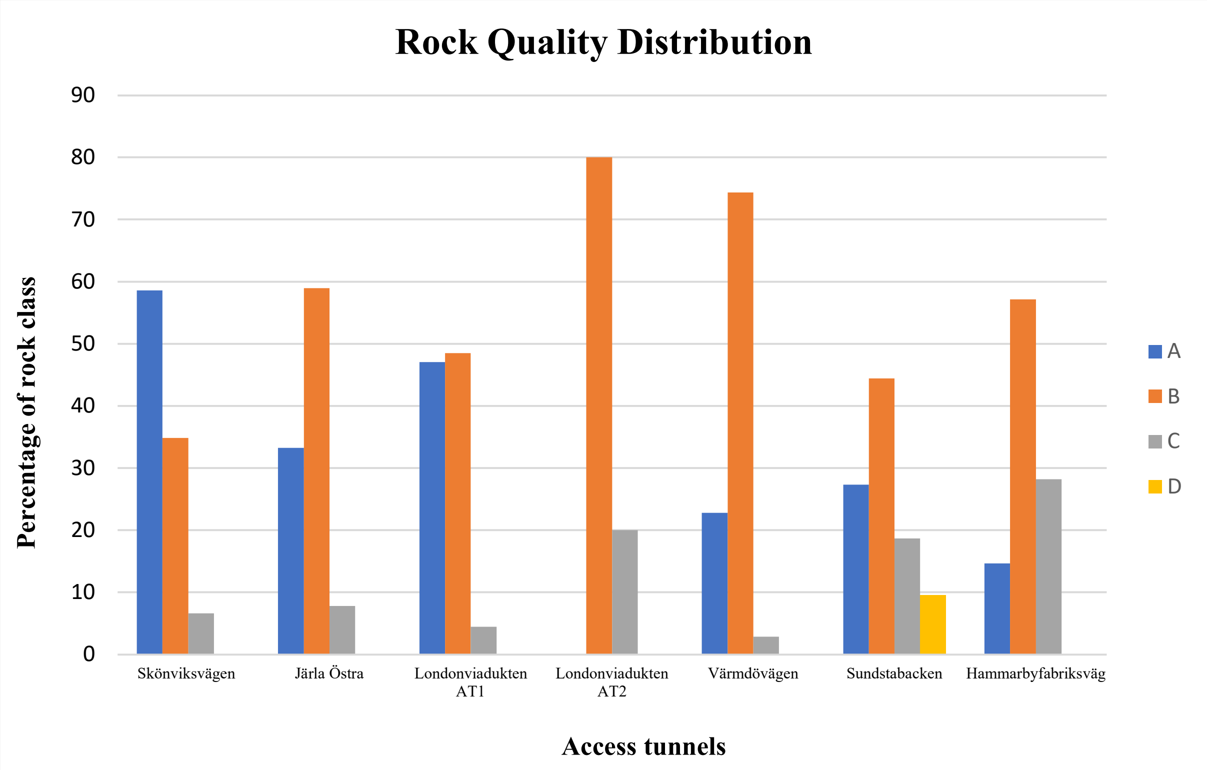


Figure 4.10. Distribution rock classes along the access tunnels.

4.5 Hydrogeological domains

The hydraulic conductivity is estimated through water loss measurements from 37 boreholes or from the water capacity of the existing wells. Using the arithmetic mean and standard deviation of hydraulic conductivity, the hydrological domains are then identified. Based on design documents, the hydrogeological domains were classified into three main groups with similar properties: surface rock, normal rock, and weakness zones. The data investigation showed that the normal rock must be divided into two parts: Normal east (Nacka) and Normal west (Kungsträdgården, Sofia, Sickla, Järla, Gullmarsplan) as shown in Figure 4.11. The boundary between surface and normal rocks is 20 m below the ground level. Surface rocks are generally more cracked with high hydraulic mean. Weakness zones do not always mean higher permeability, because these zones can be as dense as the surrounding rocks. However, high cracks frequency increases the likelihood of high permeability, thus increases the likelihood that more gout shall be performed to achieve the required sealing efficiency.

Water loss measurements were performed with 6 m section length (scale) for each borehole. Hydraulic conductivity calculations in the sections with water loss measurements were considered statistically distributed as a lognormal distribution as shown in Figures 4.12 to 4.15. From the figures, it can be noticed that the hydraulic conductivity for each domain falls within a range of values. Therefore, the designers took an assumed value for K_0 for each hydrogeological domain, at which they depended on effective hydraulic conductivity calculations. Effective hydraulic conductivity values fall between the geometric and arithmetic mean of the whole values. Thus, the actual hydraulic conductivity might be lower or higher. Equation (4-1) illustrates effective hydraulic conductivity calculations based on Matheron's assumption.

$$K_{3D} = K_g \times \exp\left(\frac{\sigma^2}{6}\right) \quad (4\text{-}1)$$

Where:
K_{3D} is the effective Matheron's hydraulic conductivity.
K_g is the geometric mean of hydraulic conductivity.
σ is the standard deviation.

Since water loss measurements have high uncertainties. The log normal distributions can be calculated using the readings in the frequency range 50%-90%. However, water loss measurements were originally carried out for a total of 43 boreholes, but then the number of considered boreholes in the analysis was reduced due to some measurement limitations. Examples on measurement limitations are the boreholes were the flow measured exceeds the maximum pump flow rate, collapsed boreholes where measurements had to be carried out for one full, and areas where no measurements have been recorded.

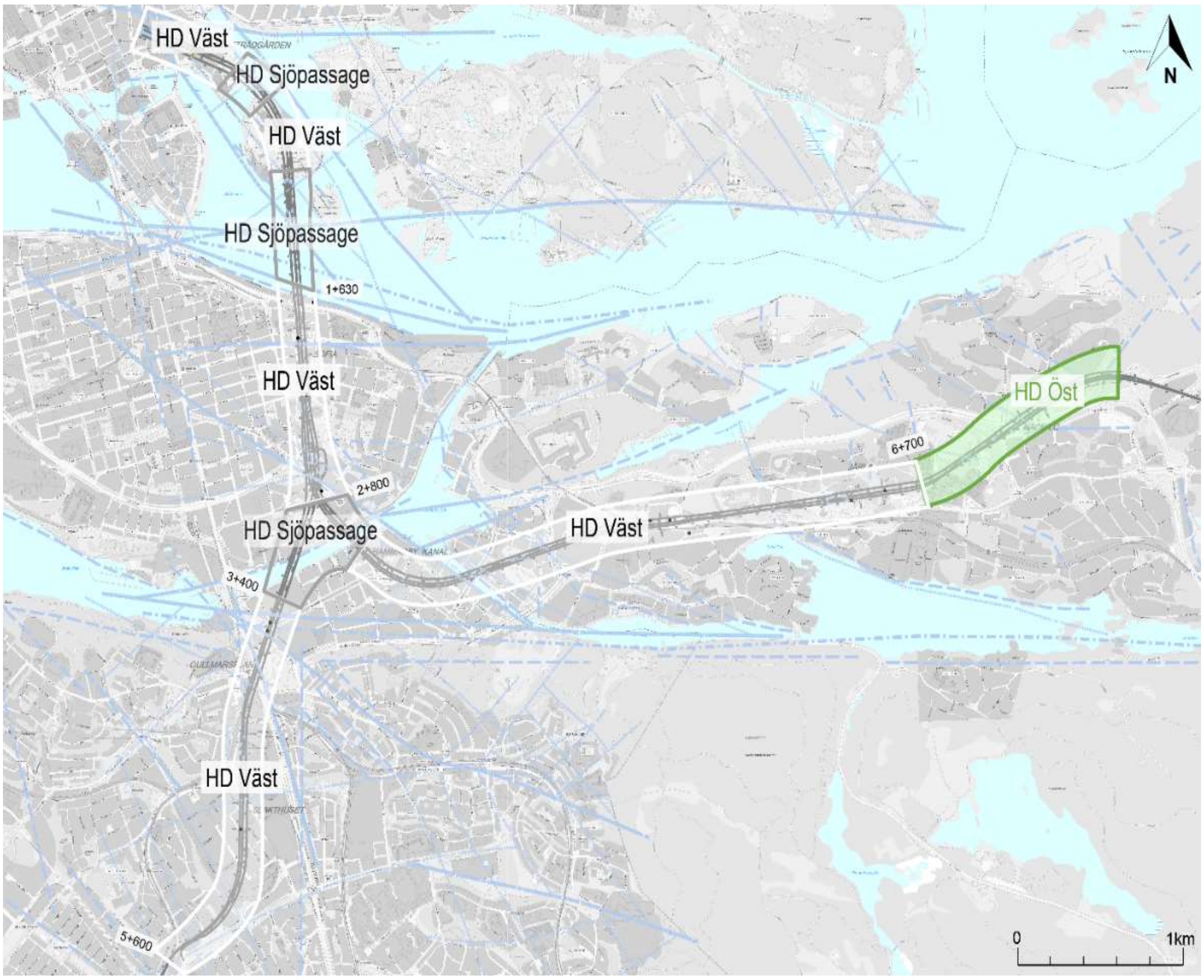


Figure 4.11. Geographical division of hydrogeological domains (FUT,2018).

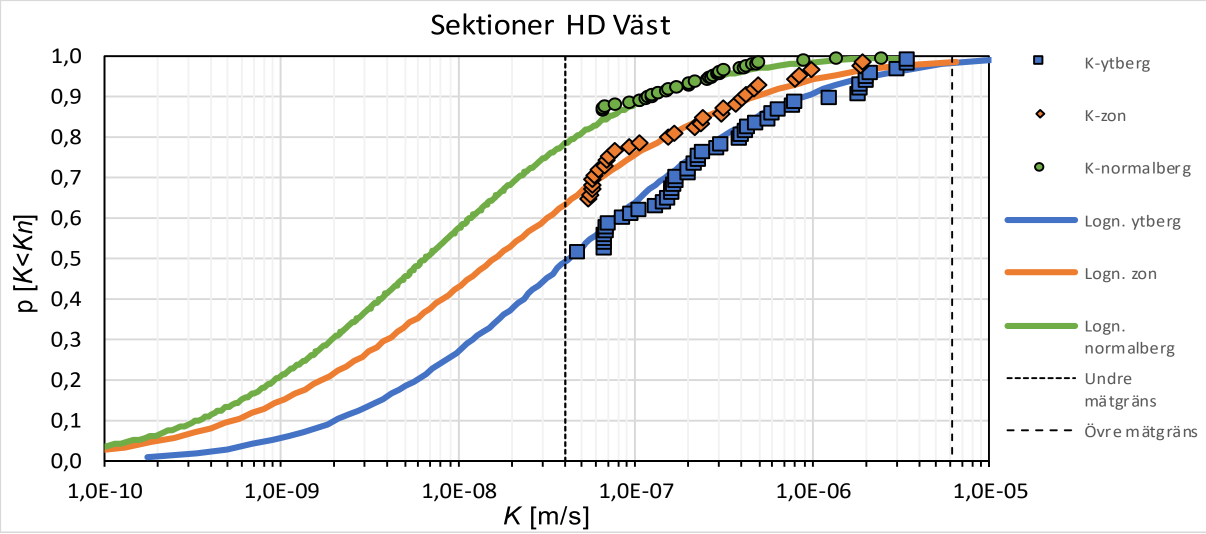


Figure 4.12. Distribution of the hydraulic conductivities for normal rock-west on a lognormal scale, based on water loss measurements performed on holes and water wells (FUT,2018).

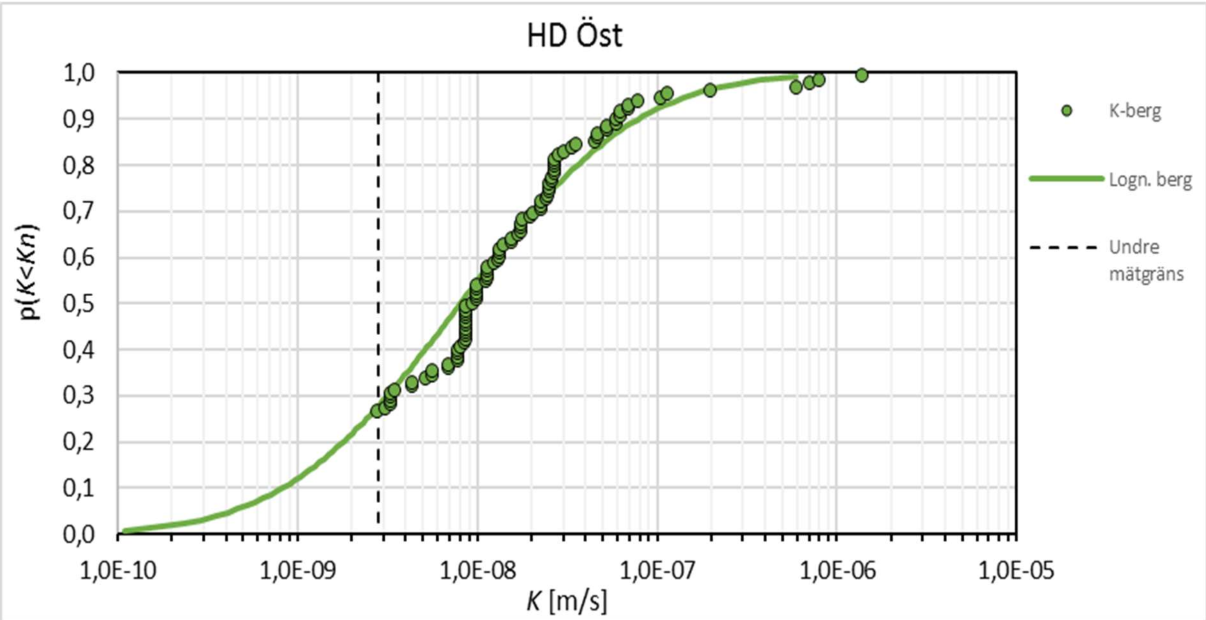


Figure 4.13. Distribution of the hydraulic conductivities for normal rock-east on a lognormal scale, based on water loss measurements performed on holes and water wells (FUT,2018).

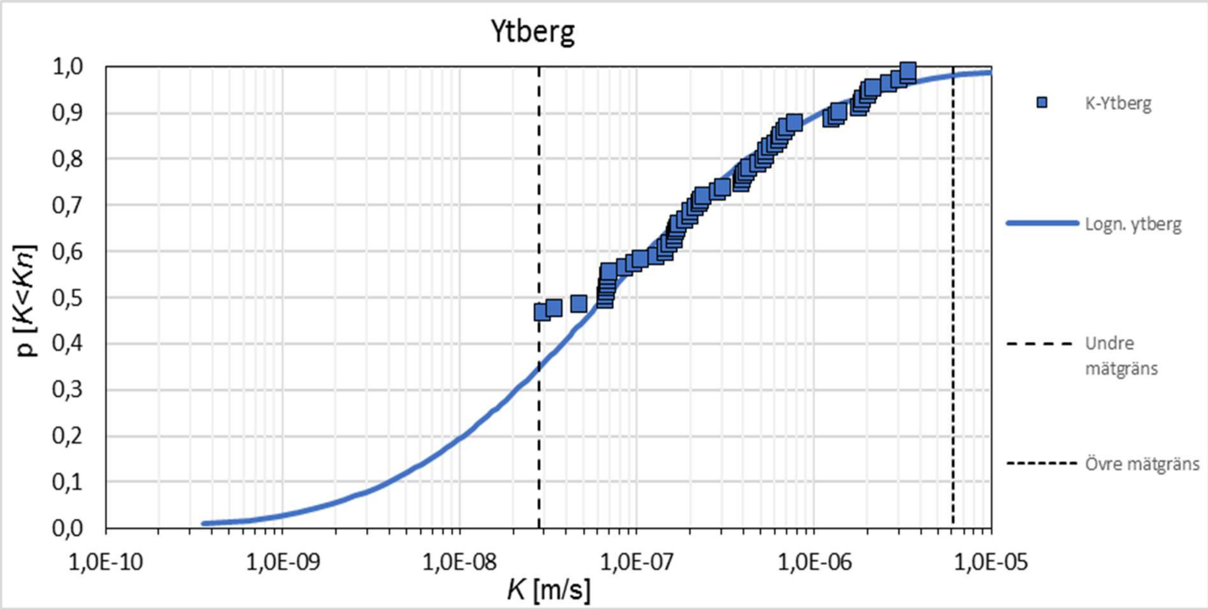


Figure 4.14. Distribution of the hydraulic conductivities for surface rock on a lognormal scale, based on water loss measurements performed on holes and water wells (FUT,2018).

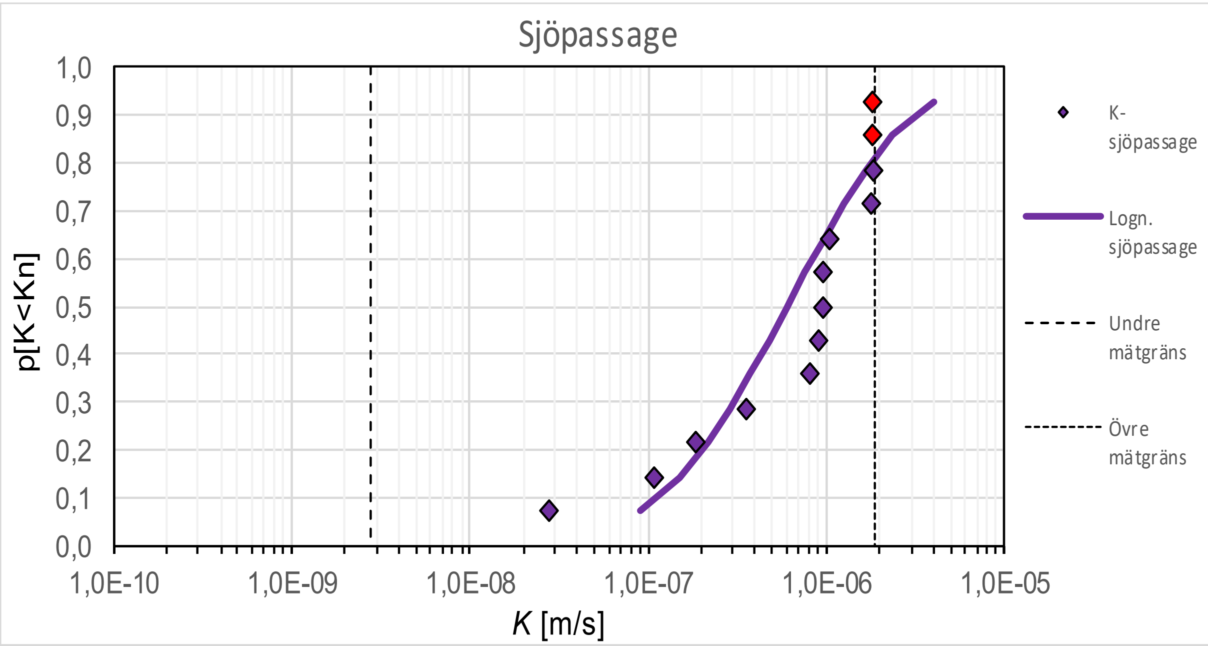


Figure 4.15. Distribution of the hydraulic conductivities for weakness zone-sea crossing on a lognormal scale, based on water loss measurements performed on holes and water wells (FUT,2018).

As described in the design documents, the estimated hydraulic conductivity values (using Matheron's equation) corresponding to each hydrogeological domain is presented in Table 4.9.

Table 4.9.Characteristic values of hydraulic conductivity within assumed hydrogeological domains.

Domain	**Assumed mean *K* [m/s]**
Normal rock West	$2x10^{-8}$
Normal rock East	$1x10^{-8}$
Surface rock	$1x10^{-7}$
Weakness zones	$5x10^{-8}$ - $5x10^{-7}$
Weakness zone sea crossing	$1x10^{-6}$

Based on design documents, the hydrogeological domains along the 6 access tunnels were identified for each tunnel stretch as shown in tunnel in Table 4.10.

Table 4.10. Identified hydrogeological domains for each tunnel stretch along the 6 access tunnels.

Tunnels	**From**	**To**	**Hydrogeological Domains**
Londonviadukten	0+035	0+112	YTNBG
	0+112	0+285	NORBG
	0+285	0+320	SVAGZ
	0+320	0+360	NORBG
	0+360	0+730	NORBG
	0+730	0+821	NORBG
Järla Östra	0+052	0+240	YTNBG
	0+240	0+267	SVAGZ
	0+267	0+373	NORBG
	0+373	0+405	SVAGZ
	0+405	0+495	NORBG
Sundstabacken	0+050	0+140	YTNBG
	0+140	0+214	NORBG
	0+214	0+280	YTNBG
	0+280	0+294	SVAGZ
	0+294	0+340	SVAGZ
	0+340	0+418	SVAGZ
	0+418	0+461	NORBG
Hammarby Fabriksväg	0+046	0+103	YTNBG
	0+103	0+230	YTNBG
	0+230	0+380	SVAGZ
	0+380	0+385	YTNBG
	0+385	0+578	NORBG

Tunnels	From	To	Hydrogeological Domains
Värmdövägen	0+027.5	0+060	YTNBG
	0+060	0+270	NORBG
	0+270	0+325	WEAKZ
	0+325	0+575	NORBG
Skönviksvägen	0+030	0+050	YTNBG
	0+050	0+090	YTNBG
	0+090	0+121	NORBG
	0+121	0+150	SVAGZ
	0+150	0+321	NORBG
	0+321	0+352	SVAGZ
	0+352	0+409	NORBG

4.6 Summary of hydrogeological domains

The percentage of each HD (hydrogeological domain) was calculated by the authors and presented in Figure 4.16 below.

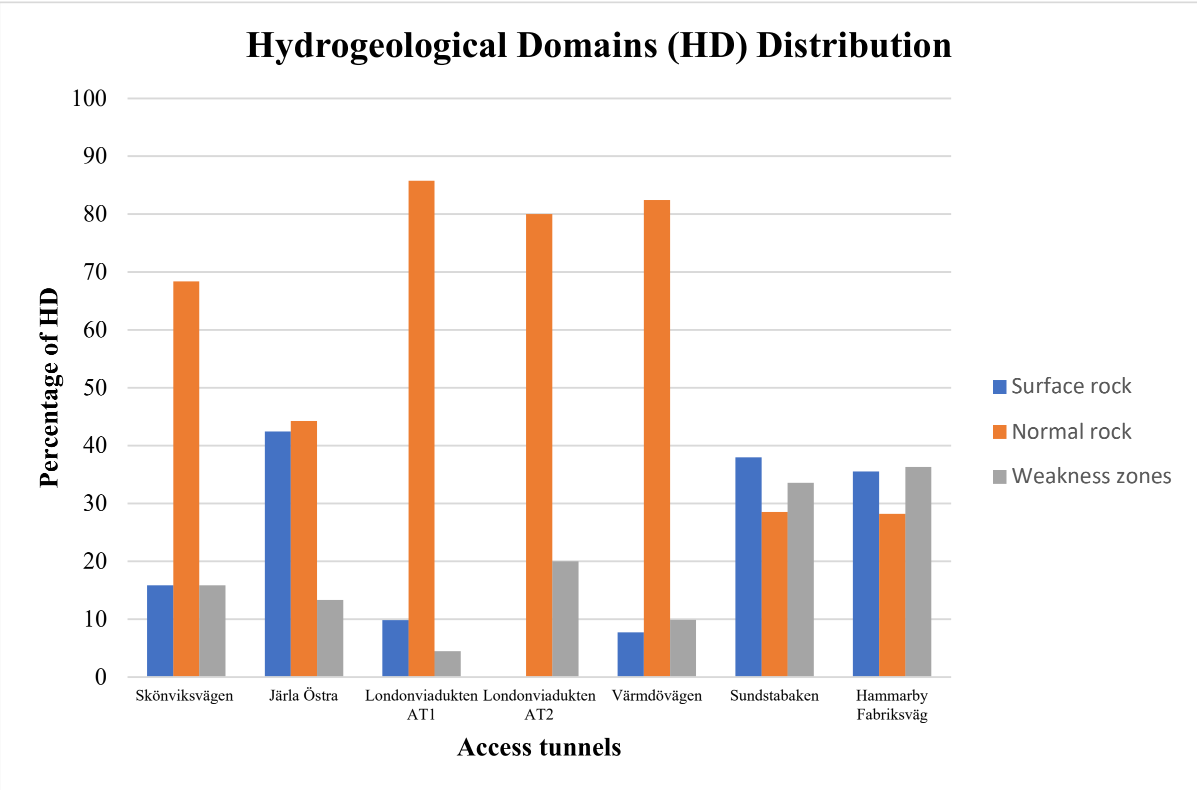


Figure 4.16. Distribution of hydrogeological domain along the access tunnels.

5. Grouting Design

5.1 Grouting Classes

Uncertainties in the grouting process are a challenge that cannot be one hundred percent solved, especially that many parameters are estimated, and the final decisions are usually related to previous experiences from old similar projects. The methodology and characteristics used to describe the water carrying capacity before and after grouting, and the calculation models come with huge uncertainties which will affect the grouting design process. The uncertainties regarding the sealing operations process and grouting performance shall be reduced to an acceptable level. The matter that requires tests, measurements and quality assurance that are given by drawings, drilling and grouting plans, stop criteria and technical specifications.

In this project, three main grouting classes were used at different sections of each access tunnel. Table 5.1 below briefly describes each grouting class based on the design documents. The grouting classes constitute of typical injection and specific injection classes to account for the variation in geological and hydrogeological conditions in different sections. The expected grouting effects and achieved sealing efficiency were estimated to calculate the ratio between the hydraulic conductivity before and after grouting. The matter that will help in the distribution of typical grouting classes. Equation 5-1 below is used to calculate the sealing efficiency (Stille, 2015).

$$\eta = 1 - \frac{K_i}{K_0} \qquad (5\text{-}1)$$

Table 5.1.Grouting class definition.

Grouting Class	Definitions
IK1	Typical injection''Typinjektering'' class. IK1 is a one grouting round class, with typical stop criteria and fan geometry, supported by additional grouting based on the first-round outcomes. IK1 is the most dominant grouting class. IK1 was designed to reach sealing efficiency (η<90%), thus using equation (5-1) above, it is obtained that $\frac{K_0}{K_i} \leq 10$.
IK2	Typical injection''Typinjektering'' class. IK2 is a two grouting rounds class with typical stop criteria and fan geometry, supported by additional grouting based on the second-round outcomes. IK2 is used in areas where IK1 is not applicable provided that the conditions for typical injection class are valid, for example where mud slumps occur in the terrain or when we have a high-water ground pressure and there is a high demand for sealing zone around the tunnel. IK2 was designed to reach sealing efficiency (90%≤η≤95%), thus using equation (5-1) above, it is obtained that $10 < \frac{K_0}{K_i} \leq 20$.

Grouting Class	Definitions
IK3	Specific technical solutions, with specific stop criteria combined with specific contract drawings and fan geometries specifications. IK3 is used when neither IK1 nor IK2 are applicable, or when the sealing demands are not met by typical injection class. Examples of cases where IK3 must be used; low rock coverage, high water pressure, weakness zones with poor rock quality, connections between tunnels, passage near existing rock spaces and at the pre-cut (förskärning) and portal (påslag).

The validity of typical injection class is governed by some requirements as shown below as per the design documents. If those conditions are not met, then specific technical design solutions, i.e., grouting class IK3 must be developed.

The distance to existing rock spaces or near structures shall be greater than 15 m. Otherwise, IK3 shall be used.

The groundwater pressure shall be between 10-70m of water column at the bottom of the rock space to be sealed. If it is less than 10m or greater than 70m, IK3 shall be used.

The tunnel or rock compartment shall be located below the groundwater level at a depth greater than approximately 1.3 times its diameter.

- Distribution of typical injection grouting classes shall be based on the conditions set for maximum sealing effect that can be achieved in IK1 and IK2, $\frac{K_0}{K_i} \leq 10$ and $10 < \frac{K_0}{K_i} \leq 20$, respectively.
- The allowable groundwater inflow requirement shall be capable of being achieved with the limitation that the rock conductivity in the injected zone meets, $k_i \geq 2\text{x}10^{-9}$ m/s.
- If groundwater inflow based on the requirements for leakage according to the application to the land and environmental court cannot be achieved, IK3 shall be used.
- The stop criteria were designed for a maximum crack transmissivity in a single crack of a maximum $6\text{x}10^{-6}$ m^2/s. In rock mass with high water supply, water loss measurements may need to be carried out to verify the validity of the stop criteria.
- The characteristics of the grout shall be verified by pre-testing and continuous testing.
- Designed grouting pressures and stop conditions should be used together with conditions for additional grout.
- The observation method must be used. Uncertainties that cannot be minimized shall be handled in the construction document.

5.2 Grout Characteristics

Grout characteristics plays an important role in affecting the grout spread in cracks, thus grout is required to have certain properties for typical injection classes to ensure durability and to achieve the desired functions. Water cement ratio must be between 0.8 and 1.0 to ensure that the grout has achieved the required resistance and it has also to show a value of b_{crit} not more than 90 µm and a value of b_{min} not more than 63 µm. During 2015 and 2016, tests were carried out in Sweden for four different grout grades, and the required properties had been formulated as shown in Table 5.2 below, as per the design document in this project. The requirements of the grout shall be verified by pre-tests and continuously monitored by contractors. The grout recipes are shown in Table 5.3 below.

Table 5.2. Requirements for the properties of the cement grout are required in the form of a range for each property.

Property	Requirements
Water cement ratio	$0.8 \leq vct \leq 1.0$
Minimum crack opening as any part of the the cement can penetrate, b_{min}	$b_{min} < 63$ µm
Minimum crack opening that 100% of the cement can penetrate, b_{crit}	$b_{crit} < 90$ µm
Viscosity, µ	$10 \text{ mPas} \leq \mu \leq 35 \text{ mPas}$
Yield strength	$1 \leq \tau \leq 3$ Pa

Table 5.3. Grout mixture properties.

Mixture, water cement ratio	Viscosity (mPas)	Yield strength (Pa)
vct 0.8	24	2.5
vct 0.6	57	6.2

5.3 Stop criteria and fan geometries: Typical injection class.

It is important that the grout is effectively spread in both large, opened cracks and fine cracks, in order to effectively reduce the groundwater flow in the injected zone. The injected zone must be homogenously sealed; thus, the grouting time must be sufficient to reach the desired penetration length. The grout spread shall be compatible with the minimum and maximum requirements of the penetration length, at which time needed to reach the required grout spread highly depends on the grout rheological properties and grouting pressure. According to the design documents, typical injection requires that the grout spread must be at least 3.0m in physical crack openings of 90 µm, which means that the distance between grouting holes should be determined before setting the stop conditions. Accordingly, a tip distance of 3m between two adjacent boreholes considered to be sufficient to obtain a homogenously sealed zone. With increasing crack openings, the grout will spread faster. However, a spread over 10m around the tunnel does not significantly increase the sealing efficiency and therefore considered uneconomic. Hence, the stop conditions shall reduce the risk of large spread because it might also pose adverse environmental impacts. As mentioned previously in the validity conditions of typical injection, the existing structures shall be at a distance of at least 15m from the injection zone. This leads to say that the maximum grout spread shall be 15 m in the largest fractures' apertures, b_{max} which is 400 µm. The stop conditions developed in this project assumes that the grout spread might vary between 3 and 15m in the same borehole.

The stop criteria are significant to guide the contractors when and where it is needed to perform additional grouting holes or/and additional grouting operations. For typical injection class, Tables 5.4 to 5.7 show the designed grouting pressure, stop criteria, additional stop conditions and additional conditions for additional grouting, respectively.

The grouting pressure was assigned in relation to the current ground water pressure or rock cover as seen in Table 5.4. They were designed to reduce the risk of jacking or lifting in the rock mass according to Rafi and Stille (2014).

Table 5.4.Grouting pressure required based on design documents (1331).

Grouting Pressure* [MPa]	**Groundwater pressure / Rock cover [m]**
1.0	10 m ≤ Groundwater pressure < 15 m
1.5	15 m ≤ Groundwater pressure< 20 m
2.0	20 m ≤ Groundwater pressure≤ 70 m

Since the grouting pressure is different, different grouting times are required to achieve the required penetration length and thus the sealing efficiency. Therefore, the stop criteria were developed for each grouting pressure as shown in Table 5.5 below. Time stop criteria are supported by maximum volume criteria to ensure that the injected volume is compatible with the requirements of maximum allowed grout spread, i.e., spread of 15m in the largest apertures b_{max}.

Table 5.5.General stop criteria for typical grouting class.

Grouting Pressure [MPa]	**Instructions for stop conditions**	**Time* [min]**	**Injected volume** [Liter]**
1.0	Grouting into the current borehole should be interrupted when any of the next-up conditions are met	12	300
1.5		10	300
2.0		10	300

*Time spent is the amount of time grouting has been going on after hole filling with the required grouting pressure. Planned and unplanned outages are not included.
**Injection volume is consumption of grout excluding hole filling.

The additional stop criteria are described below in Table 5.6 to handle the unwanted events, while Table 5.7 shows the conditions that require additional grouting rounds.

Table 5.6. Additional grouting conditions.

Unwanted event	Action
Predefined grout pressure is not achieved	Hole filling should be carried out with mixture vct=0.5 immediately after end of grout in the current hole

Table 5.7. Additional grouting rounds.

Examples of additional grout conditions	Action*
The stop conditions for injected volume are reached before the stop conditions for injected time for more than <*number*> grouting holes	New holes are drilled and incubated on each side of the current hole where the conditions for additional grout are met
Volume stop conditions are reached before the time stop condition for more than <*number*> of all grout holes in the fan	New grout operation is performed

*Drilling may be started no earlier than after the grout has hardened to 0.5 kPa according to the results of the pre-test.

Fan geometries for typical injection sections drawing are shown in Appendix (A), where the borehole length, inclination, overlap, stick, and tip distance are illustrated. In typical injection class, the borehole lengths range between 21-24 meters, usually 24m in this project. The tip distance between boreholes is 3m, the stick is 5m and the overlap is also 5m.

5.4 Stop criteria and fan geometries: Specific technical solutions.

As mentioned previously, specific technical solutions were developed when typical injection class cannot be applied or does not meet the required sealing efficiency. Cases where technical solutions IK3 were developed, are specific for each fan that requires IK3. Figures A.3 to A.19 and Tables A.1 to A.21 in Appendix (A) show the borehole length, inclination, overlap, stick, tip distance, grouting pressures and stop conditions for all IK3 tunnel stretches. However, to summarize this section, Table 5.8 below was formulated to describe most of the important IK3 solutions developed for each access tunnel. For IK3, it was divided into 2 groups: IK3 due to geometry and IK3 due to weakness zones. The first group covers situations where connection between tunnels exist, low rock coverage, and precut at the beginning of each tunnel, and other situations such as passing near existing structures (distance <15m), and injection at large depth (water pillar >70 m). Whereas the second group applies for situations such as passage in areas with poor rock quality and unfavorable geological conditions regarding the number of dominant crack directions, crack direction vs. tunnel direction, cracks' endurance, and cracks' fillings.

5.5 Grouting class for each tunnel stretch.

The grouting classes were estimated for each tunnel stretch based on the expected sealing effect, i.e., K_0/Ki. Since the expected sealing efficiency depends on rock mass characteristics, a methodology for grout class distribution was developed based on the difficulty of the injections with respect to its corresponding geological conditions. The methodology includes the assessment of 4 parameters: number of dominant crack directions, crack direction vs. tunnel direction, cracks' endurance, and cracks' fillings. Tables A.22 to A.27 in Appendix show the assessment made for all access tunnels. The results were classified from the least severe to the most sever conditions as follows: favourable, relatively favourable, relatively unfavourable, and unfavourable. This assessment helps in determining the sealing power K_0/Ki. The maximum sealing power for IK1 is 10 and for IK2 is 20. For IK3, the assessment is based on specific conditions. Table 5.8 shows the grouting class for each tunnel stretch as per the design documents.

Table 5.8.Grouting class for each tunnel stretch.

Tunnels	From	To	K_0/Ki	Grouting Classes	Required Sealing Efficiency (%)
Londonviadukten	0+035	0+046	8	IK3 based on IK1	87.5%
	0+046	0+085	8	IK1	87.5%
	0+085	0+115	8	IK3 based on IK1	87.5%
	0+115	0+285	5	IK1	80%
	0+285	0+320	5	IK3 based on IK2	80%
	0+320	0+440	5	IK1	80%
	0+440	0+821	4	IK3	75%
Järla Östra	0+020	0+052	-	IK3	
	0+052	0+064	6	IK3 based on IK1	83.8%
	0+064	0+090	6	IK1	83.8%
	0+090	0+230	8	IK1	87.5%
	0+230	0+270	6	IK3 based on IK2	83.8%
	0+270	0+373	5	IK1	80%
	0+373	0+405	10	IK2	90%
	0+405	0+468	5	IK1	80%
	0+468	0+495	-	IK3	-

Tunnels	**From**	**To**	***K_0/Ki***	**Grouting Classes**	**Required Sealing Efficiency (%)**
Sundstabacken	0+022	0+048	-	IK3	
	0+048	0+060	4	IK3 based on IK1	75%
	0+060	0+140	10	IK3 based on IK2	90%
	0+140	0+160	4	IK3 based on IK2	75%
	0+160	0+280	4	IK1	75%
	0+280	0+340	8	IK3 based on IK2	87.5%
	0+340	0+418	12	IK2	91.7%
	0+418	0+461	4	IK1	75%
Hammarby Fabriksväg	0+046	0+103	-	IK3	
	0+103	0+114	8	IK3 based on IK1	87.5%
	0+114	0+135	8	IK1	87.5%
	0+135	0+210	8	IK3 based on IK1	87.5%
	0+210	0+235	12	IK2	91.7%
	0+235	0+280	10	IK3 based on IK2	90%
	0+280	0+380	12	IK2	91.7%
	0+380	0+400	12	IK2	91.7%
	0+400	0+410	4	IK1	75%
	0+410	0+578	4	IK1	75%
Värmdövägen	0+017	0+027,5	-	IK3	
	0+027,5	0+060	5	IK3	80%
	0+060	0+95	4	IK3	75%
	0+95	0+270	4	IK1	75%
	0+270	0+325	12	IK2	91.7%
	0+325	0+529	4	IK1	75%
	0+529	0+575	4	IK3	75%
Skönviksvägen	0+030	0+050	-	IK3	
	0+050	0+062	8	IK3 based on IK1	87.5%
	0+062	0+90	8	IK1	87.5%
	0+90	0+121	2.5	IK1	60%
	0+121	0+150	12	IK2	91.7%
	0+150	0+321	2.5	IK1	60%
	0+321	0+352	12	IK2	91.7%
	0+352	0+365	2.5	IK1	60%
	0+365	0+409	-	IK3	

5.6 Summary of grouting class results

The percentage of each grouting class was calculated by the authors and presented in Figure 5.1 below. The most dominant grouting class according to design is IK1.

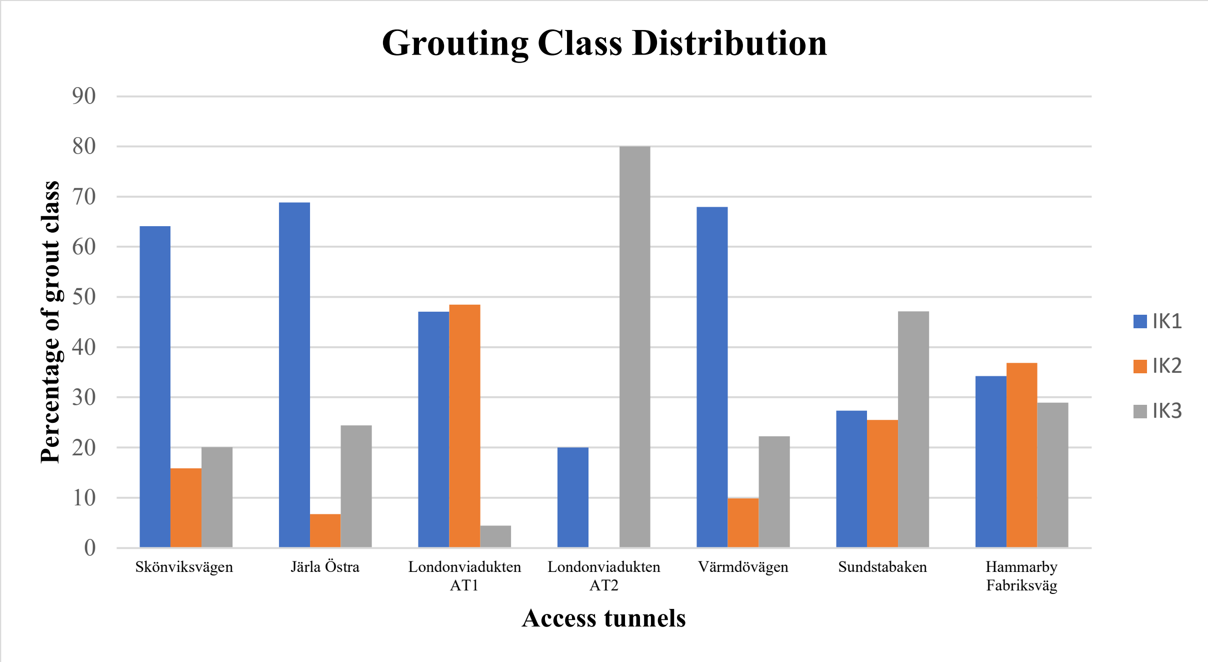


Figure 5.1. Distribution of hydrogeological domain along the access tunnels.

6. Results and Discussions

6.1 Evaluation of rock quality

The percentage of rock mass classes was calculated for each access tunnel based on design documents classification as shown in Figure 6.1. The percentage of actual rock mass quality distribution was also calculated using the geological mappings from the site as shown in Figure 6.2, at which only certain fans with available data were investigated. Thus, to make a fair comparison between the actual and determined rock quality in the design documents, only the sections with available data in both site and design were taken into consideration. Total length evaluated for each access tunnel is written on the x-axis in the figures below.

Due to uncertainties in rock mass classification from the test boreholes during design phase, some fans had different rock mass quality according to the geological mapping results. Figure 6.2 shows that the rock class percentage distributions from geological mapping results are different from the ones in the design documents. Deviation in rock mass quality classifications is important to study the effect of rock quality on sealing efficiency and grouting process, and to link some changes in the quality to certain events. In Londonviadukten, it is shown that rock class D was not predicted in the design phase, but it can be seen later that 4% of rock class D appears in the geological mapping, mainly in the weakness zones. Whereas in Sundstabacken, field investigations prior to the design showed that there is rock class D. However, the geological mapping shows that the weakest rock in Sundstabacken was rock class C not D. Therefore, the effect of such deviations is discussed in the next sections.

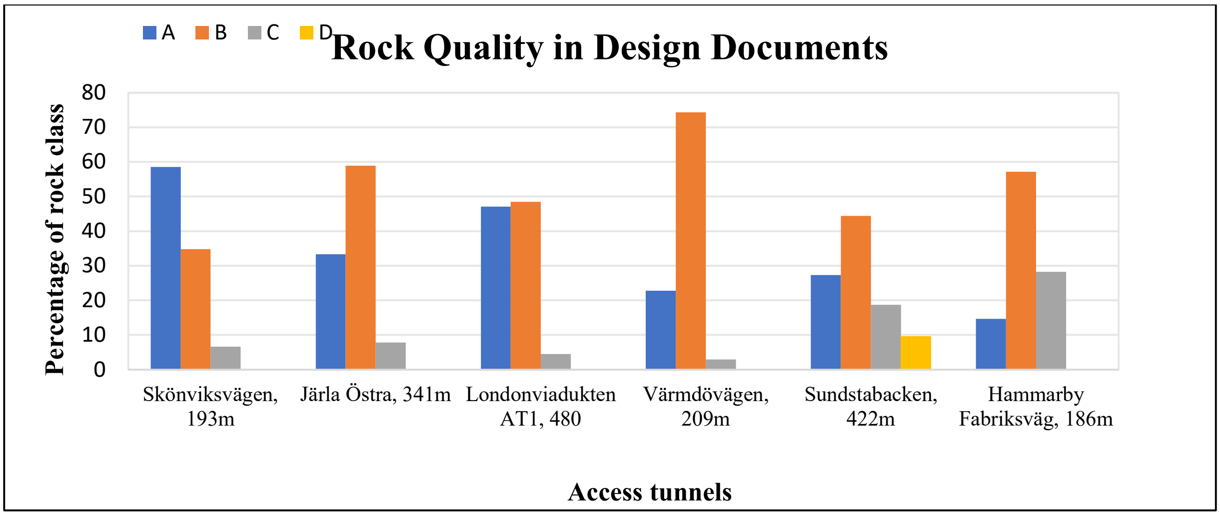


Figure 6.1.Rock quality distribution based on design documents for evaluated sections.

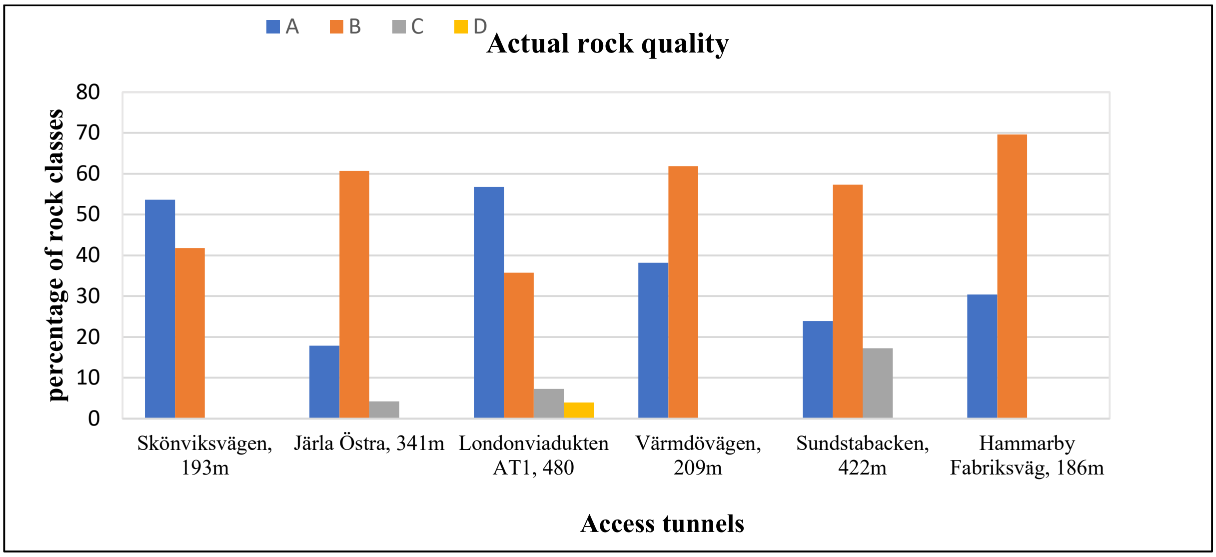


Figure 6.2.Rock quality distribution based on geological mappings from the site for evaluated sections.

6.2. Evaluation of executed grouting works

6.2.1 Average grout uptake

To make an assessment for the actual grouting process performed at site, each fan has been studied in terms of number of boreholes, number of injection rounds, grout mixture properties, total injected volume, total drilling length, and grout uptake. injected volume per meter. Then the stop criteria for each borehole were evaluated individually. The stop criteria were defined as the following:

- If the volume of grout take reaches the maximum volume according to the design for the evaluated grouting fan, grouting was stopped by **volume criteria**.
- If the volume of grout does not reach the evaluated borehole filling volume according to the design for the evaluated grouting fan, stop criteria is **zero flow** (already sealed or the boreholes does not pass-through fractured zones).
- If grouting is stopped after the design grouting time, grouting was stopped by **time criteria**.

6.2.1.1 Londonviadukten

In Londonviadukten, access tunnel a total of 31 fans from chainage 0+035 to chainage 0+515, have been evaluated. Some fans had only one round of grouting and some had 2 or even 5 rounds. Out of these 31 fans, only 3 fans were in weakness zone. In Table 6.1, the number of fans, injection times, and total injected volume for each grouting class is shown. Almost 60 tons of grout were injected for this tunnel. But only 45% of this volume were taken by grouting classes IK1 and IK3 geometry. Whereas IK3 weakness zone consumed 55% of the injected volume although the length for this weakness zone was only 35 meters, i,e, 5% of the total evaluated length.

Table 6.1:Londonviadukten-grout evaluation.

Grouting Class	**Number of fans**	**Total length (m)**	**Number of injections (including boreholes that injected one time and multi-times)**	**Total grout excluding hole filing volume (L)**	**Percentage of grout uptake**
IK1	20	304	559	14,058	23%
IK3 Weakness zone	3	22	230	33,132	22%
IK3 Geometry	8	83	226	13,243	55%
IK2	0	0	0	0	0
SUM	**31**	**409**	**1015**	**60,433**	**100%**

The average of grout uptake was calculated for each fan per grouting class and compared to other tunnels as well. The aim of this process is to try to find correlations or reference values for the average grout uptake for different geological conditions per grouting class. Figure 6.3 below shows the average uptake with the standard deviation for each grouting class. For weakness zones, the standard deviation was the highest because some of the fans took around 59 L/m and some took only 5 L/m. Because of this high standard deviation, it was hard to predict a reference value for weakness zones. Therefore, the average uptake can be taken as estimation for similar geological conditions in future work in the main tunnel or for any other projects with similar conditions. Figure 6.3 also shows that this access tunnel did not have IK2 zones. For IK1 and IK3 geometry, the values are used to make a comparison with all the other access tunnels to check if some reference values can be obtained.

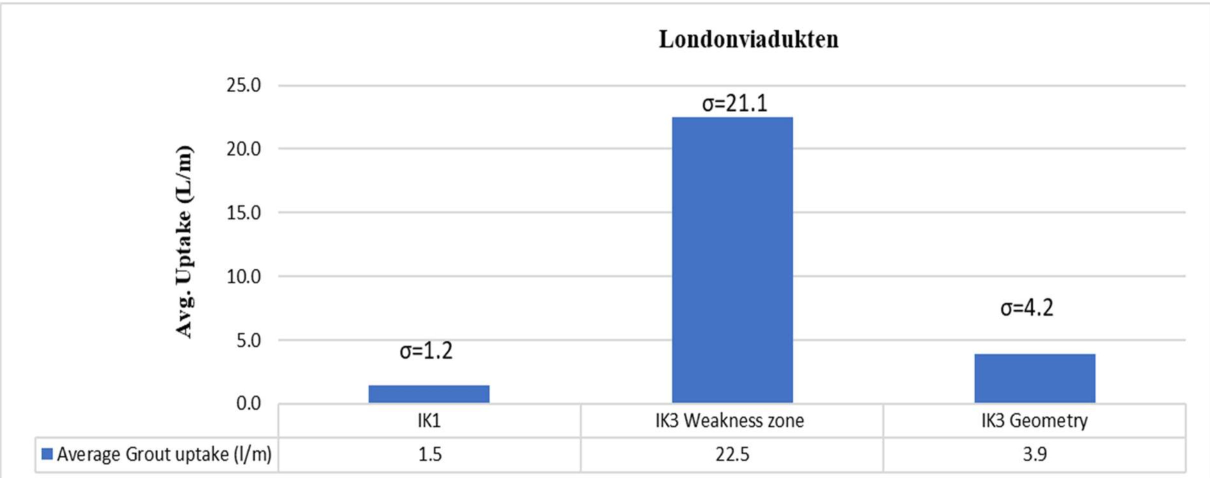


Figure 6.3.Average grout take for each grouting class excluding the hole filling volume (AT Londonviadukten).

Due to the variation of hydraulic conductivity in weakness zones as shown in Figure 4.15 in chapter 4, the average grout uptake varies after each grouting round as shown in Figures 6.4 and 6.5 below. In fan 294, the average grout uptake decreased from 20 L/m (excluding hole and hose filling volume) to almost

0 L/m after 5 rounds of injection (from fan 294 round 1 until fan 298 round 5). But it should be mentioned that the results at fan 297 round 2 are irrelevant because at this fan, the grouting was done for horizontal boreholes outside the already grouted zone. While in fan 304, the grout uptake was reduced from almost 4 to 1 L/m (excluding hole and hose filling volume) after 2 grouting rounds. These chainages are a good example to show how the hydraulic conductivity does not only vary from one hydrogeological domain to another but also within the same domain. Even though these two fans fall within the same weakness zone as stated in the design documents, the hydraulic conductivities can vary depending on the fractures' apertures and distribution, which is why the grout uptake changed differently after each grouting round.

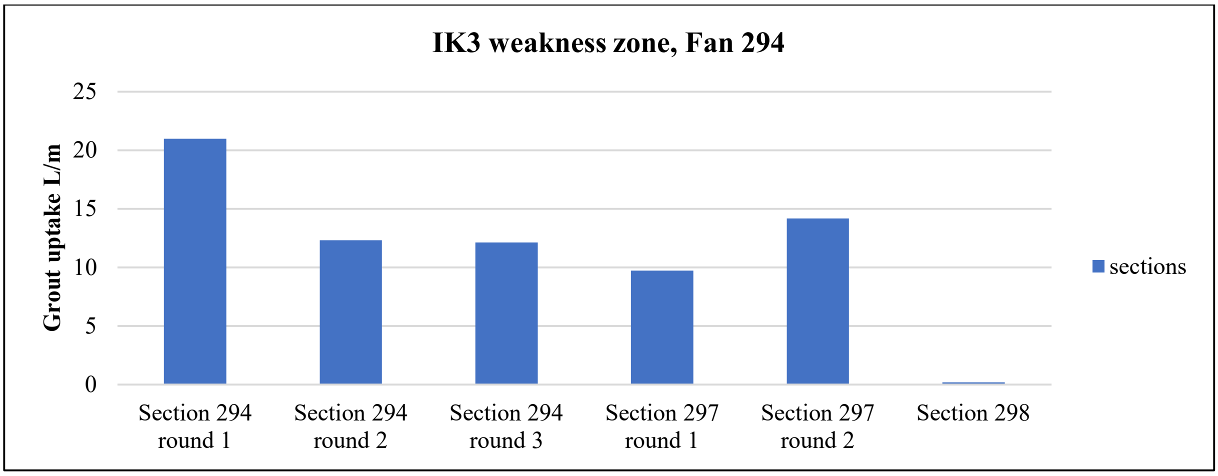


Figure 6.4: Grout uptake variation for each round-fan 294-Londonviadukten.

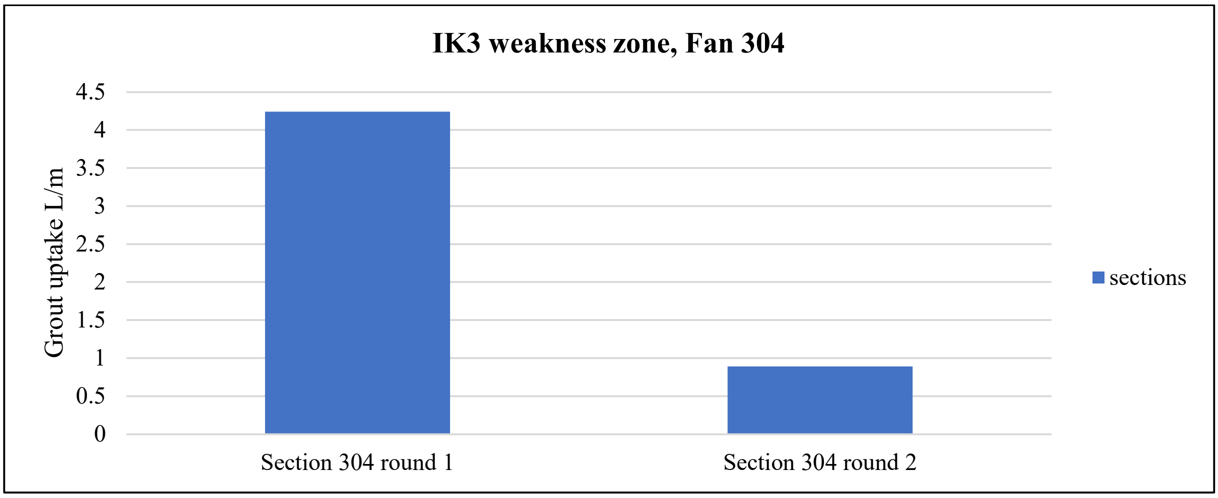


Figure 6.5: Grout uptake variation for each round-fan 304-Londonviadukten.

6.2.1.2 Järla östra

In Järla Östra access tunnel, a total of 19 fans, from chainage 0+052 to chainage 0+361, have been evaluated. As shown in Table 6.2 below, more than 50% of the total grout uptake is typical injection class IK1, which is normal because most of the evaluated fans were in IK1 zones.

Table 6.2: Järla Östra-grout evaluation.

Grouting Class	Number of fans	Total Length (m)	Number of injections (including boreholes that injected one time and multi-times)	Total grout excluding hole filing volume (L)	Percentage of grout uptake
IK1	14	212	285	22,912	56%
IK3 Geometry	1	52	34	5,985	15%
IK3 Weakness zone	4	45	102	12,015	29%
IK2	0	0	0	0	0%
Sum	**19**	**309**	**421**	**40,912**	**100%**

The average of the grout uptake and standard deviation were calculated for each fan per grouting class as shown in Figure 6.6 below. Only one fan was in IK3 geometry zone with a large uptake in a surface rock zone, that is expected to be with high crack intensity and high permeability.

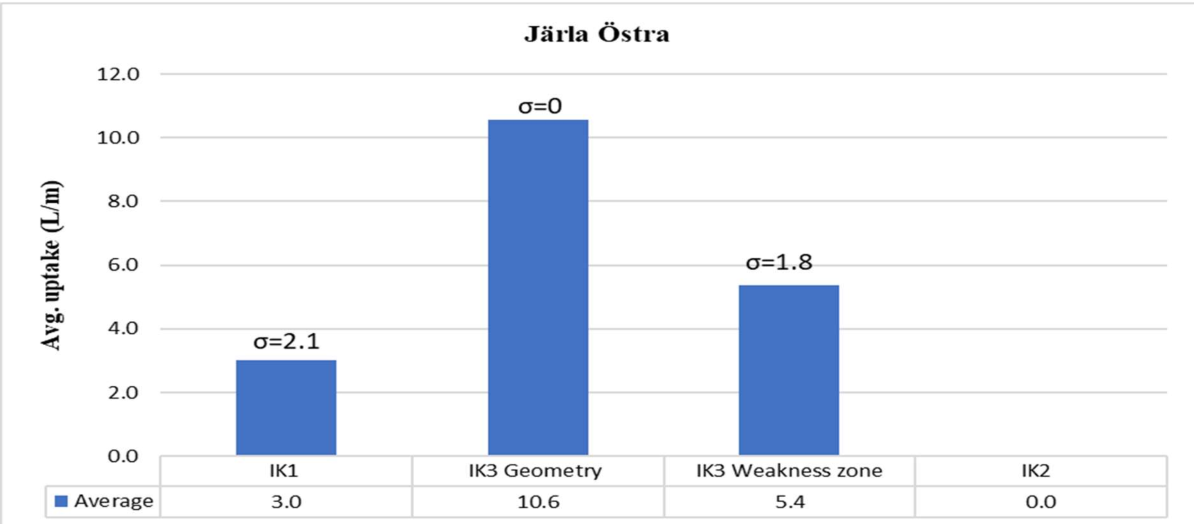


Figure 6.6. Average grout take for each grouting class excluding the hole filling volume (Järla Östra).

6.2.1.3 Sundstabacken

In Sundstabacken access tunnel, a total of 25 fans, from chainage 0+022 to chainage 0+467, have been evaluated. As shown in Table 6.3 below, 62% of the grout uptake was typical injection classes IK1 and IK2. It should be mentioned that in this access tunnel, the weakness zones were injected with IK2 not with IK3. It is also shown in Figure 6.7 that this access tunnel took less grout compared to the other tunnels, which can be interpreted by the fact that the rock at this tunnel is generally a good rock with no filling materials that can block the fractures and prevent the grout from penetrating through.

Table 6.3.Sunstabacken grout evaluation.

Grouting Class	Number of fans	Total Length (m)	Number of injections (including boreholes that injected one time and multi-times)	Total grout excluding hole filing volume (L)	Percentage of grout uptake
IK1	14	269	407	1,812	36%
IK3 Geometry	6	97	181	1,928	38%
IK3 Weakness zone	0	0	0	0	0
IK2	5	106	105	1,322	26%
Sum	**25**	**472**	**693**	**5,062**	**100%**

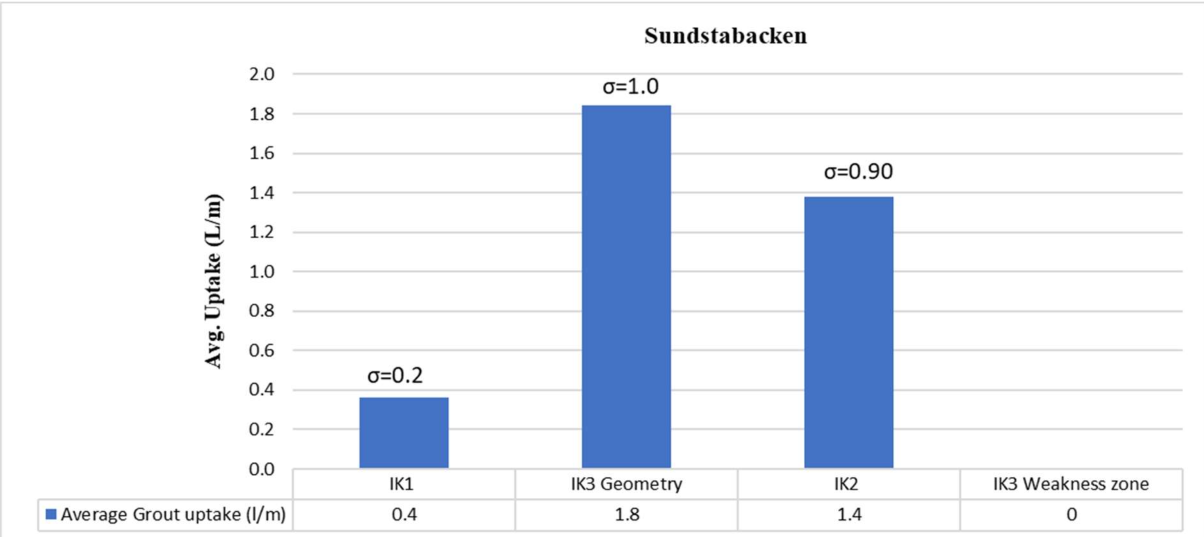


Figure 6.7. Average grout take for each grouting class excluding the hole filling volume (Sundstabacken).

6.2.1.4 Hammarby Fabriksväg

In Hammarby Fabriksväg access tunnel, a total of 11 fans, from chainage 0+050 to chainage 0+255, have been evaluated. As shown in Table 6.4 below, more than 50% of the total grout uptake is taken by IK3 weakness zones because most of the fans analyzed were in weakness zone.

Table 6.4. Hammarby Fabriksväg grout evaluation.

Grouting Class	**Number of fans**	**Total Length (m)**	**Number of injections (including boreholes that injected one time and multi-times)**	**Total grout excluding hole filing volume (L)**	**Percentage of grout uptake**
IK1	1	21	21	487	9%
IK3 Geometry	2	23	50	857	15%
IK3 Weakness zone	6	73	129	3,772	67%
IK2	2	39	44	551	10%
Sum	**11**	**156**	**244**	**5,667**	**100%**

The average of the grout uptake and standard deviation were calculated for each fan per grouting class as shown in Figure 6.8 below. It is shown that the average uptake in weakness zone in Hammarby Fabriksväg took lower grout uptake average compared to Londonviadukten and Järla Östra, which indicates so far that it is difficult to have one reference value for grout uptake in weakness zones. Instead, these can be used for similar geological conditions. The average crack frequency is about 3 cracks per meter which is lower than that in Londonviadukten and Järla Östra which were 4 and 5 cracks per meter respectively. As for the crack endurance in this tunnel it varies between 1.5 m and 20 m for all crack groups.

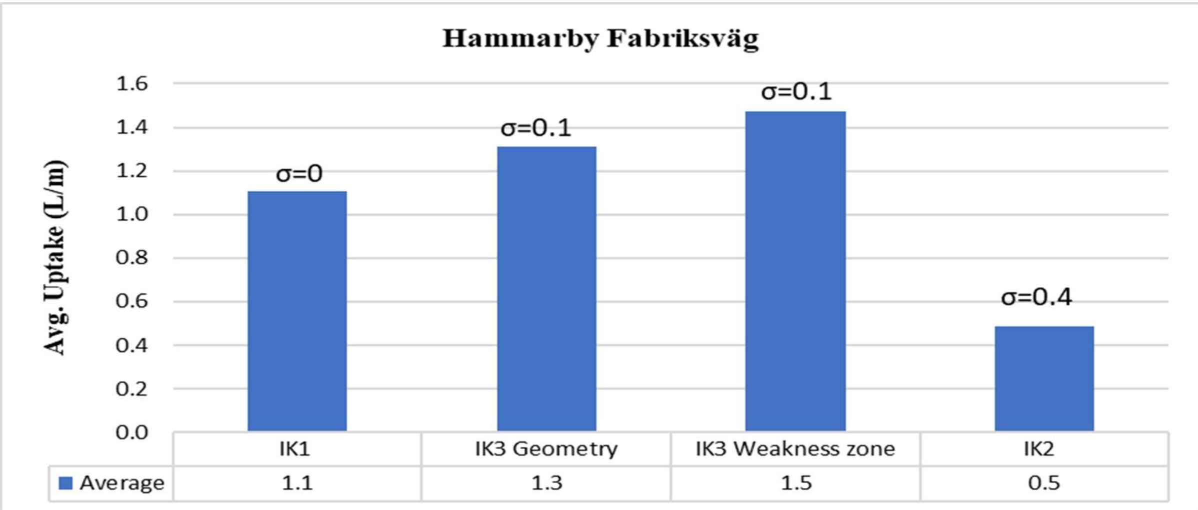


Figure 6.8. Average grout take for each grouting class excluding the hole filling volume (Hammarby Fabriksväg).

6.2.1.5 Värmdovägen

In Värmdövägen access tunnel, a total of 17 fans, from chainage 0+031 to chainage 0+226, have been evaluated. Working percentage in this access tunnel was 37%. Thus, few data are presented. Table 6.5 below shows that IK3 geometry zones took more than 50% of the total grout uptake even though the number of fans and boreholes were almost half the fans and holes in IK1 zones. IK3 geometry took place in surface rock and normal rock hydrogeological domain zones. It is mainly taken by pre-cut (Påslag) fans.

Table 6.5.Värmdovägen grout evaluation.

Grouting Class	Number of fans	Total Length (m)	Number of injections (including boreholes that injected one time and multi-times)	Total grout excluding hole filing volume (L)	Percentage of grout uptake
IK1	11	124	202	6461	46%
IK3 Geometry	6	58	102	7549	54%
IK3 Weakness zone	0	0	0	0	0
IK2	0	0	0	0	0
Sum	**17**	**182**	**244**	**5,667**	**100%**

The average of the grout uptake and standard deviation were calculated for each fan per grouting class as shown in Figure 6.9 below.

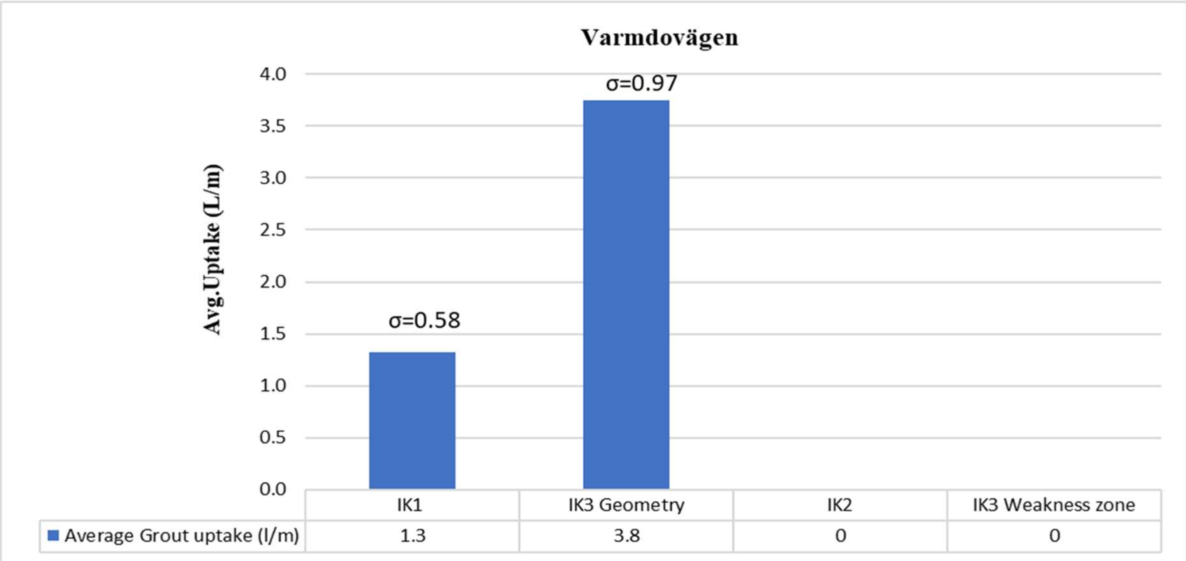


Figure 6.9. Average grout take for each grouting class excluding the hole filling volume (Värmdovägen).

6.2.1.6 Skönviksvägen:

In Skönviksvägen access tunnel, a total of 13 fans, from chainage 0+048 to chainage 0+223, have been evaluated. Table 6.6 below shows that the typical injection zones IK1 and IK2 took 98% of the total grout uptake because there was only one fan for IK3 geometry class.

Table 6.6. Skönviksvägen grout evaluation.

Grouting Class	Number of fans	Total Length (m)	Number of injections (including boreholes that injected one time and multi-times)	Total grout excluding hole filing volume (L)	Percentage of grout uptake
IK1	10	106	204	11034	86%
IK3 Geometry	1	48	34	220	2%
IK3 Weakness zone	0	0	0	0	0
IK2	2	20	40	1484	12%
Sum	**13**	**174**	**244**	**5,667**	**100%**

The average of the grout uptake and standard deviation were calculated for each fan per grouting class as shown in Figure 6.10 below.

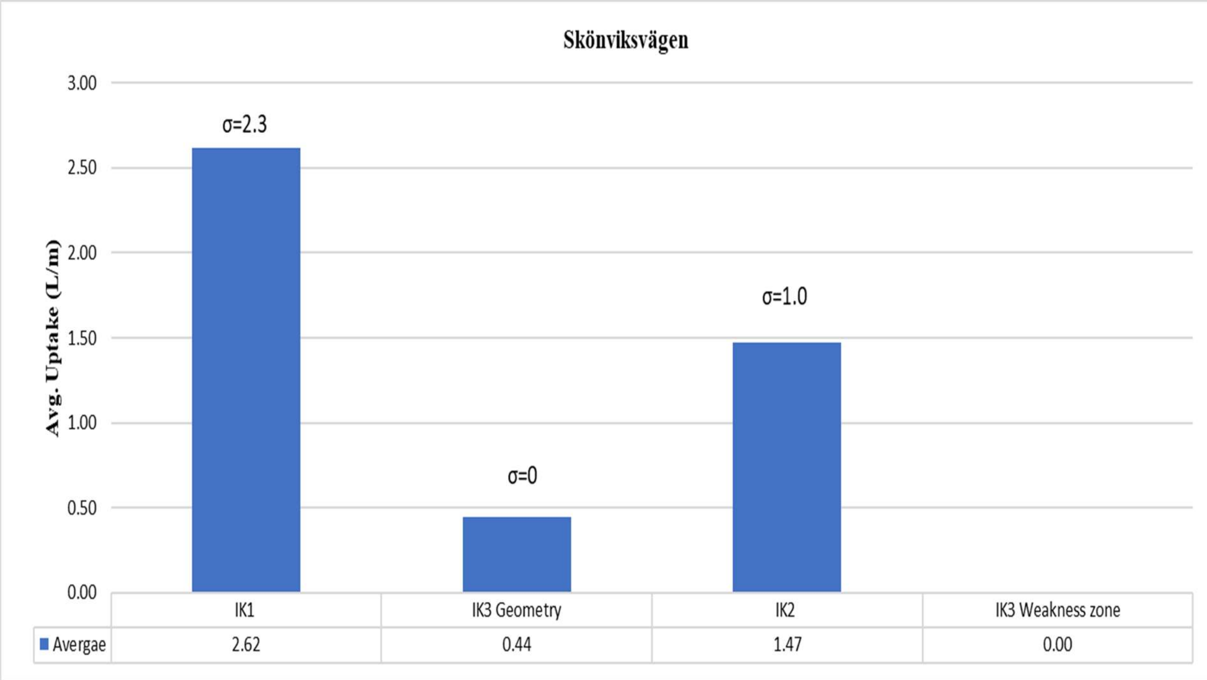


Figure 6.10. Average grout take for each grouting class excluding the hole filling volume (Skönviksvägen).

6.2.2 Evaluation of Stop Criteria

6.2.1 Londonviadukten

The stop criteria were then evaluated for each borehole. Figure 6.11 shows the percentage of boreholes that were stopped by volume, time or zero flow for each grouting class. In IK3 class (IK3 weakness and geometry), 12-13% of the boreholes were stopped by volume, and re-injected which is an indicator that 500L as a maximum volume according to design stop criteria was not enough, thus additional grouting rounds were required. While in IK1 class, 50% of the boreholes took zero flow, which is an indicator that those holes are either already sealed or filled with clay. It could be also that these holes do not pass cracked zones. Other expected reasons for such case are that the boreholes pass through zones with low hydraulic conductivity, or the fractures' apertures are very fine at which the grout cannot penetrate.

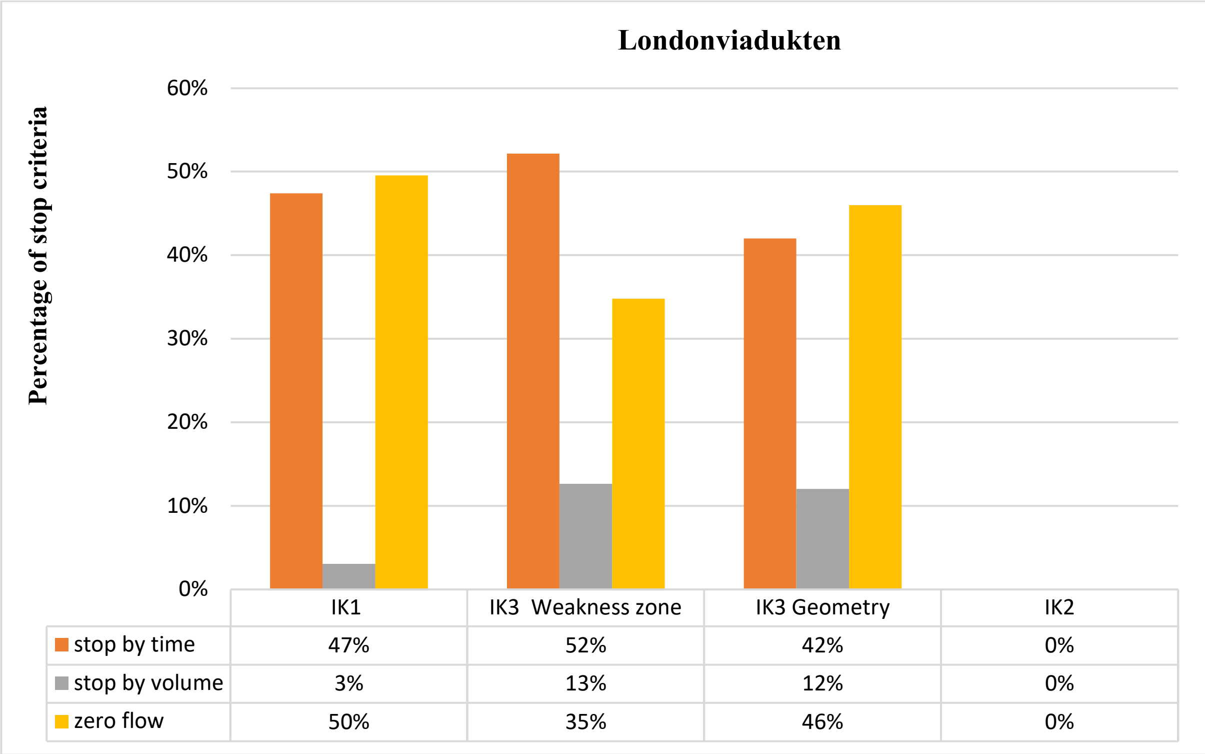
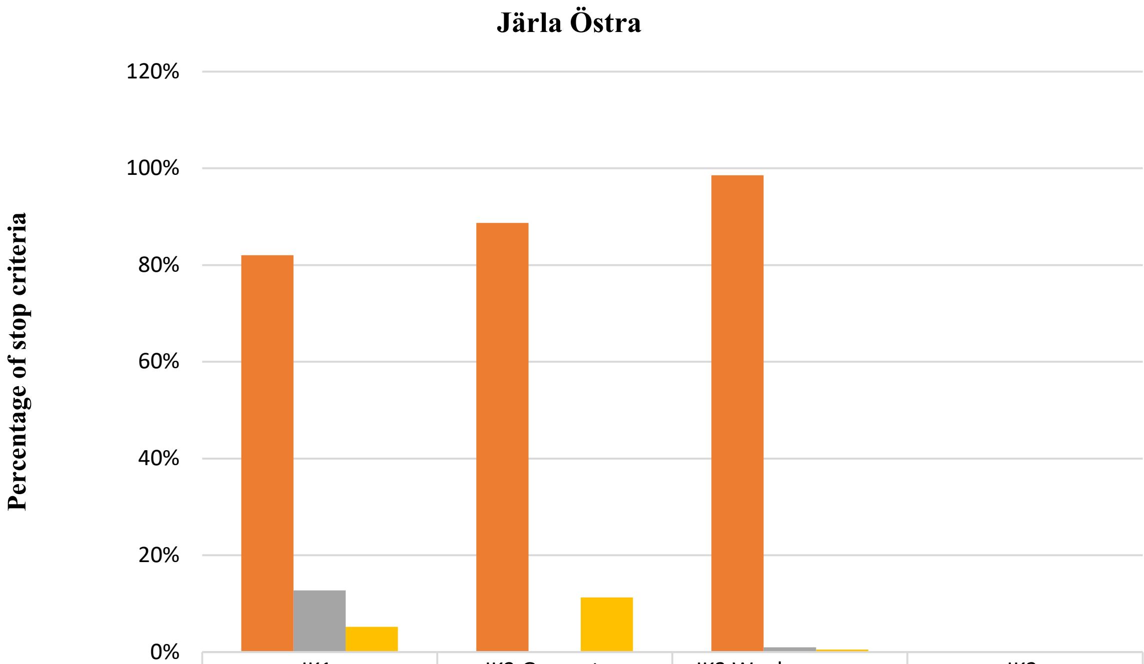


Figure 6.11. Comparison of grouting stop criteria (AT Londonviadukten).

6.2.2 Järla Östra

The stop criteria were then evaluated for each borehole as shown in Figure 6.12. In IK1 class, 13% of the boreholes were stopped by volume. This is an indicator here that these holes, which are in surface rock domain (highly conductive zone) as shown in Figure 4.14 in chapter 4, need to be injected more. This also depends on whether those holes have achieved the required sealing efficiency or not. In later chapters, the measured flow is compared with the prognosed flow that indicates if the stop criteria in IK1 zone here must be changed or not, and that is mainly linked to the existence of wide span of conductive surface rocks. It is also noted that percentage for boreholes stopped by time was high for all grouting classes. This is an indicator that the grouting design in this tunnel was good. As for the boreholes that were stopped by flow, it was only 5% for fans which took IK1 and 11% for fans that took IK3 geometry. This can be interpreted by the presence of too fine fractures or already sealed.



	IK1	IK3 Geometry	IK3 Weakness zone	IK2
stop by time	82%	89%	99%	0%
stop by volume	13%	0%	1%	0%
zero flow	5%	11%	0%	0%

Figure 6.12. Comparison of grouting stop criteria (AT Järla Östra).

6.2.3 Sundstabacken

Figure 6.13 shows the percentage of boreholes that were stopped by volume, time or zero flow for each grouting class. No boreholes stopped by volume, which means that the boreholes did not need much grouting because of favourable conditions. However, some boreholes were injected with extra rounds because the prescribed pressure was not reached. The results show also that almost half of the holes were stopped by time and the other half did not take any grout, this might be interpreted in different ways. Either the crack frequency is too low, but the studies showed before that the crack frequency is 2-4 cracks/m, or that the fractures were too fine in which the grout could not penetrate. It could be also that the cracks are already sealed. Presence of filling materials such as clay can be also a reason. However, in geological mapping, no indication for large amounts of clay was observed that can hinder the penetration of the grout. For IK2 the percentage of boreholes that were stopped by time was 57%, which means that the grouting time was not really sufficient and therefore it is recommended to be increased. On the other hand, none of the boreholes were stopped by volume, which indicates that none of the fans required additional grouting rounds. But more than 40% of the boreholes in these fans were already sealed. The reason could be one of the interpretations mentioned above.

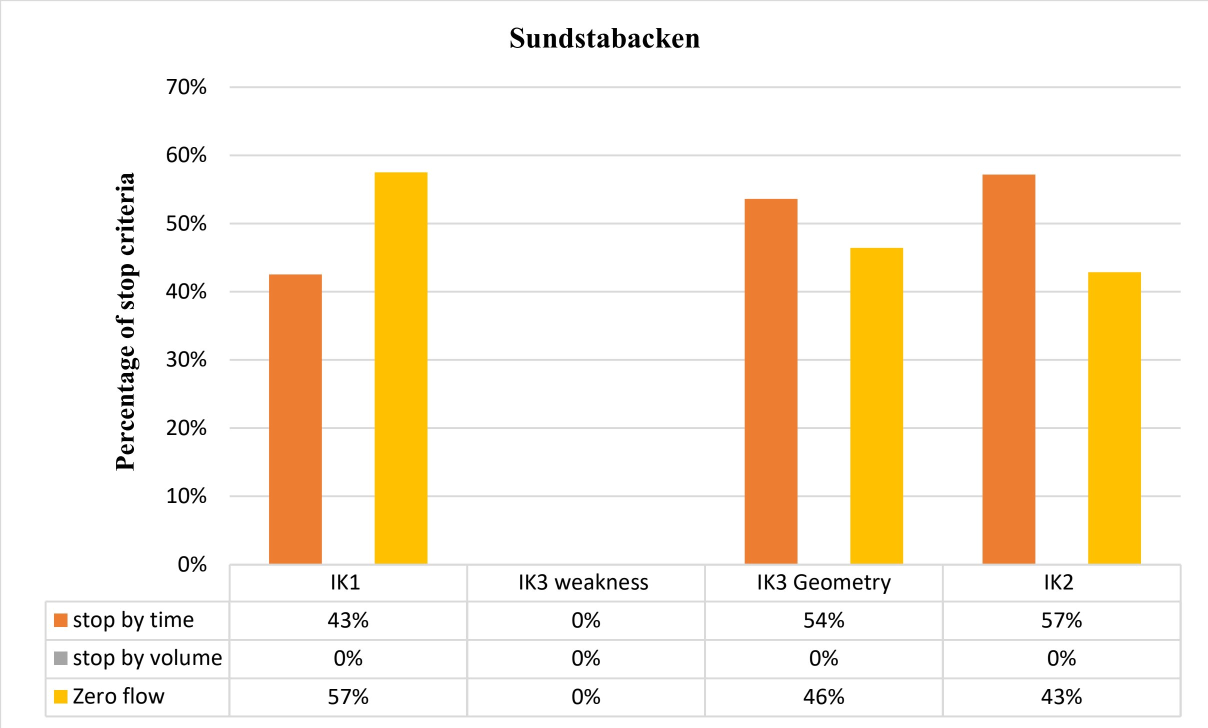
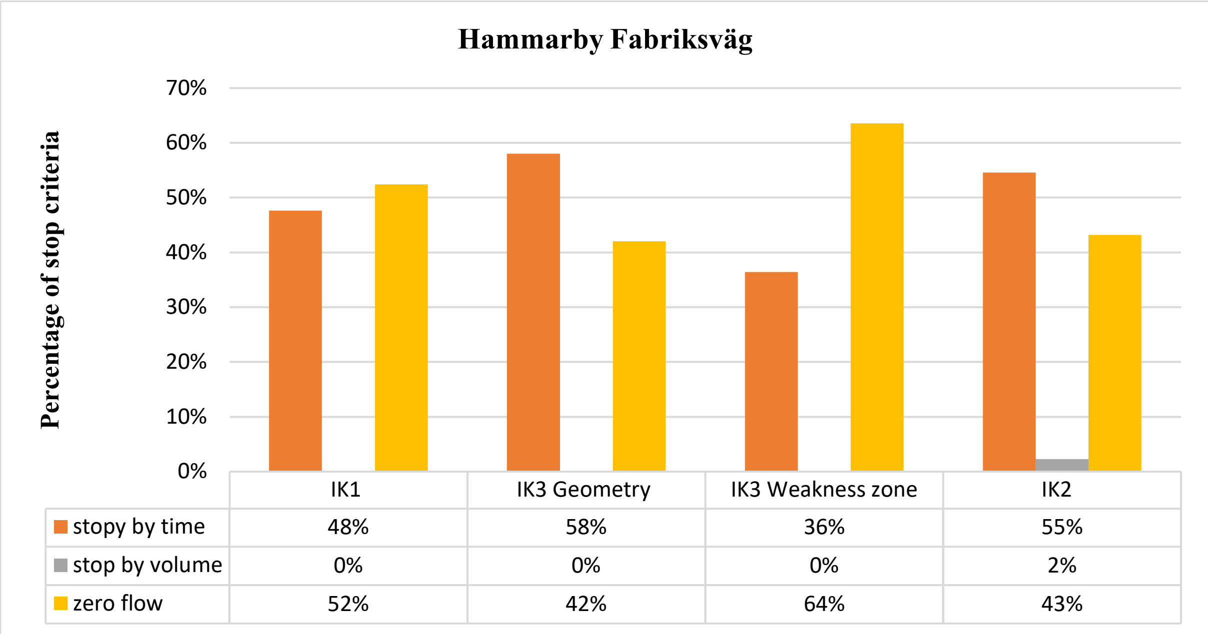


Figure 6.13. Comparison of grouting stop criteria (AT Sundstabacken).

6.2.4 Hammarby Fabriksväg

Figure 6.14 shows the percentage of boreholes that were stopped by volume, time or zero flow for each grouting class. Only 2% of the boreholes stopped by volume in IK2 zones; mainly was in surface rock area. Having 2% of the boreholes in IK2 in only two studied fans, that were stopped by volume, means that the surface rock domain in this tunnel might require more grouting time. But the judgements on the stop criteria will be made in the leakage chapter, where the measured flow after grouting is compared to the prognosed flow. As for IK3 weakness zone only 36% of the borehole were stopped by time, and thus 15 min are not sufficient, and it is highly recommended to increase the grouting time for this specific technical solution in such geological conditions. And 64% of the boreholes that took IK3 weakness zones were already sealed and did not take any grout. This percentage is really hight and was not really expected because the hydraulic conductivities based on water loss measurements for the ungrouted rock in that area had a range between $5*10^{-8}$ - $5*10^{-7}$ m/s. Thus generally, in all grouting classes the percentage of the boreholes that took zero flow was high, but according to studies performed in these areas, it did not have large crack fillings that can hinder the grout from flowing into the fractures and sealing it. However, most of the areas were in surface rock domain that has usually high crack intensity. Thus, it can be that some of the fractures in this tunnel were kind of connected to each other and by sealing these fractures the others were directly sealed during the grouting process.

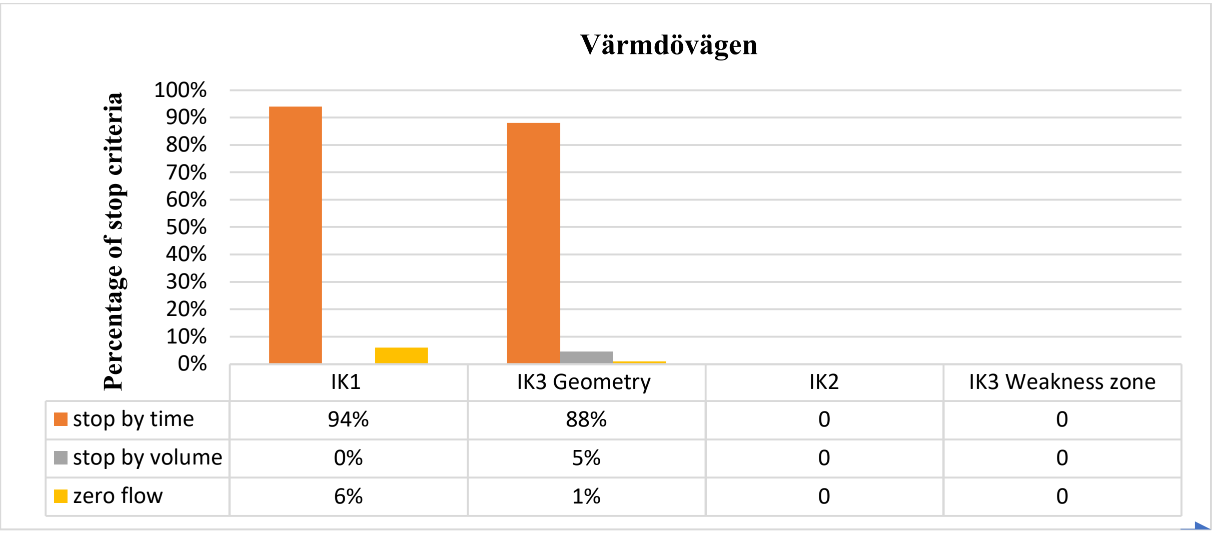


	IK1	IK3 Geometry	IK3 Weakness zone	IK2
stopy by time	48%	58%	36%	55%
stop by volume	0%	0%	0%	2%
zero flow	52%	42%	64%	43%

Figure 6.14. Comparison of grouting stop criteria (AT Hammarby Fabriksväg).

6.2.5 Värmdovägen

Figure 6.15 shows the percentage of boreholes that were stopped by volume, time or zero flow for each grouting class. Only 5% of the boreholes stopped by volume in IK3 geometry zones, which also indicates that this zone requires more grouting and accordingly the design stop criteria need to be reviewed. The recommendations on changing the stop criteria can be either confirmed or disregarded depending on the achieved sealing efficiency and the measured leakage after grouting which will be discussed later.



	IK1	IK3 Geometry	IK2	IK3 Weakness zone
stop by time	94%	88%	0	0
stop by volume	0%	5%	0	0
zero flow	6%	1%	0	0

Figure 6.15. Comparison of grouting stop criteria (AT Värmdovägen).

6.2.6 Skönviksvägen

Figure 6.16 shows the percentage of boreholes that were stopped by volume, time or zero flow for each grouting class. A total of 10% of the boreholes were stopped by volume in typical injection IK1 and IK2 zone, which indicates that there is a need to increase the maximum volume stop criteria, provided that the leakage requirements are not yet met.

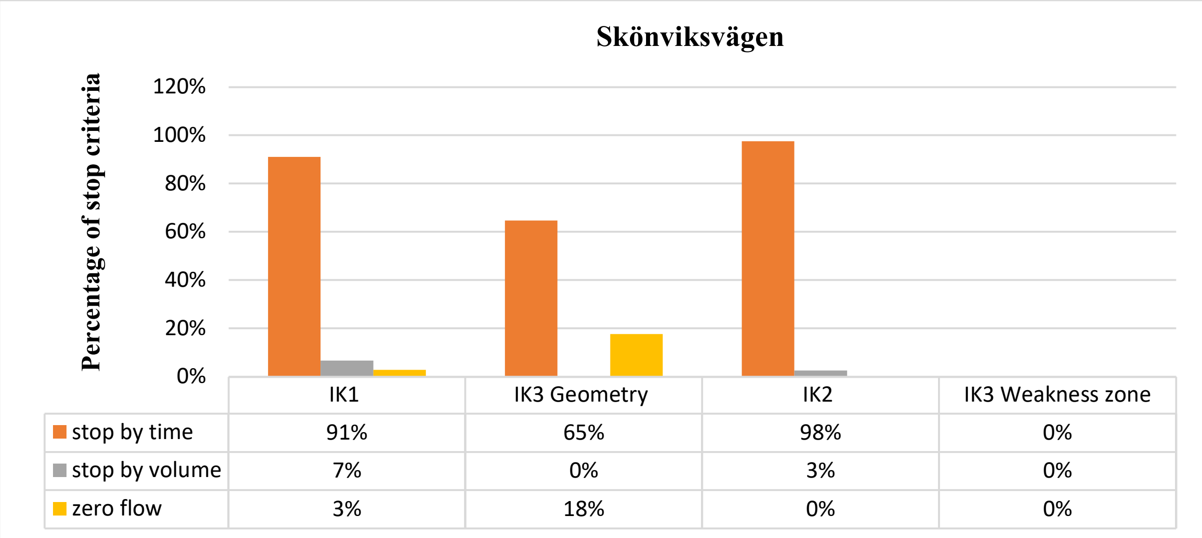


Figure 6.16.Comparison of grouting stop criteria (AT Skönviksvägen).

6.2.3 Summary of results

From the figures above and based on the authors' analysis, it is estimated that the average grout uptake in typical injection (IK1 and IK2) zones is almost 2 L/m with a standard deviation of 2 (excluding hole and hose filling volume). While in IK3 zones; IK3 geometry and IK3 weakness zones, there is no certain reference value that can be used for the average grout uptake. The standard deviation is high which makes it difficult to obtain a certain value. However, the average uptake and standard deviation values for each access tunnel can be used for similar geological conditions in the main tunnels or in other future projects. The reason behind this great variation in weakness zones is that weakness zones have been classified and evaluated using many parameters, crack orientation, crack filling and crack lengths. But to sum up the obtained results for IK3 zones we can say the following:

- For similar conditions like Londonviadukten the average grout uptake in weakness zones is 23 L/m. For some fans, the total grout volume was 52 L/m for 5 rounds. The grout uptake decreased from 21 to 0.2 L/m after 5 rounds. Whereas in other fans the total grout volume was 10.3 L/m, and the grout uptake decreased from 9 L/m to almost 1 L/m after only 2 rounds. This depends on the hydraulic conductivity of the analyzed area, keeping in mind that the conductivities can vary within the same weakness zone. As for IK3 geometry the average grout uptake is 4 L/m

- For Järla Östra the average grout uptake is 5.4 L/m in weakness zones and for IK3 geometry the 10.6 L/m. In this access tunnel,11 fans were in surface rock zone, which is known to be highly conductive. For similar conditions the average grout uptake in this area is 4 L/m.
- For similar conditions like Sundstabacken the average grout uptake is 1.8 L/m for IK3 geometry.
- For similar conditions like Hammarby Fabriksväg it is 1.5 L/m for weakness zones. As for IK3 geometry the average grout uptake is 1.3 L/m.
- For Varmdovägen the average grout uptake is 3.8 L/m for IK3 geometry.
- For Skönviksvägen the average grout uptake is 0.4 L/m for IK3 geometry.

To make better judgements on the reference values for typical injection. The average grout uptake was compared to the average grout uptake in City Line project. The comparison is shown in section 6.3.

6.3 Comparison between grout uptake with City Line project

In City Line projects, 2491 boreholes in typical grouting classes were analyzed for typical injection zones and the average uptake was 2L/m, excluding hole filling volume. Most of evaluated screens had only one grouting round and a few fans took two rounds. Water cement ratio for the grout mixture was 0.8. In Figure 6.17 below, the most important findings from City Line project are shown. The results in City Line showed that 90% of all holes took less than 5 L/m (100 L/ average hole length of 21 m). It was also found that 40% of the holes were dense, where they were only filled with hole filling volume and did not take grout to fill the fractures.

In this work, the extension of blue line project showed quite similar results for typical injection zones as in City Line. From the summary in section 6.2, the calculations and observations showed that the average grout uptake were estimated to be 2 L/m with a standard deviation of 2. This was estimated based on the authors analysis and using engineering judgement. Those results are compatible with the typical injection in City Line. Moreover, Figure 6.18 below shows the analysis of 340 boreholes taken from Londonviadukten and Sundstabacken in typical injection zones. The results show that in the extension of blue line project, 90% of the holes took less than 120 L over an average length of 24m holes, which means that 90% of the holes took less than 5 L/m as well. It is also shown that almost 38% of the holes were dense and did not take grout for fractures. These results are also compatible with the City Line project, which means that the design criteria for typical injection class in this project were good. However, the achieved sealing efficiency will govern if those criteria need to be changed or maybe optimized.

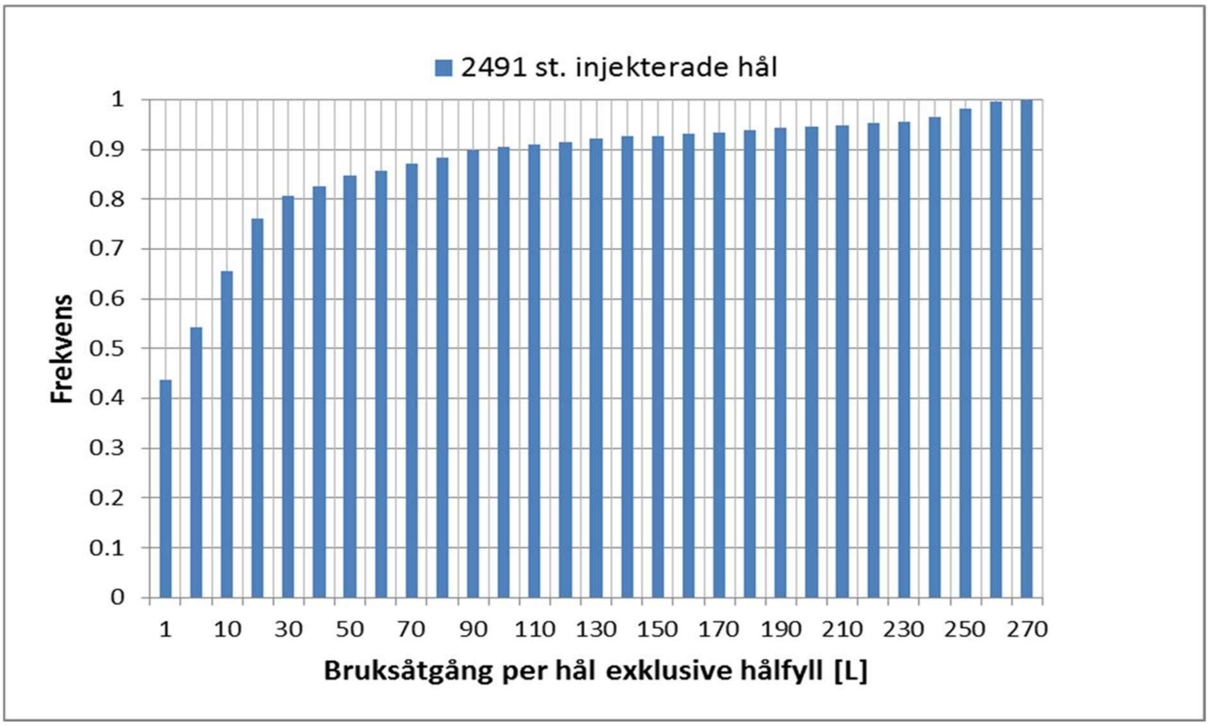


Figure 6.17.Evaluation of typical injection uptake in City line project.

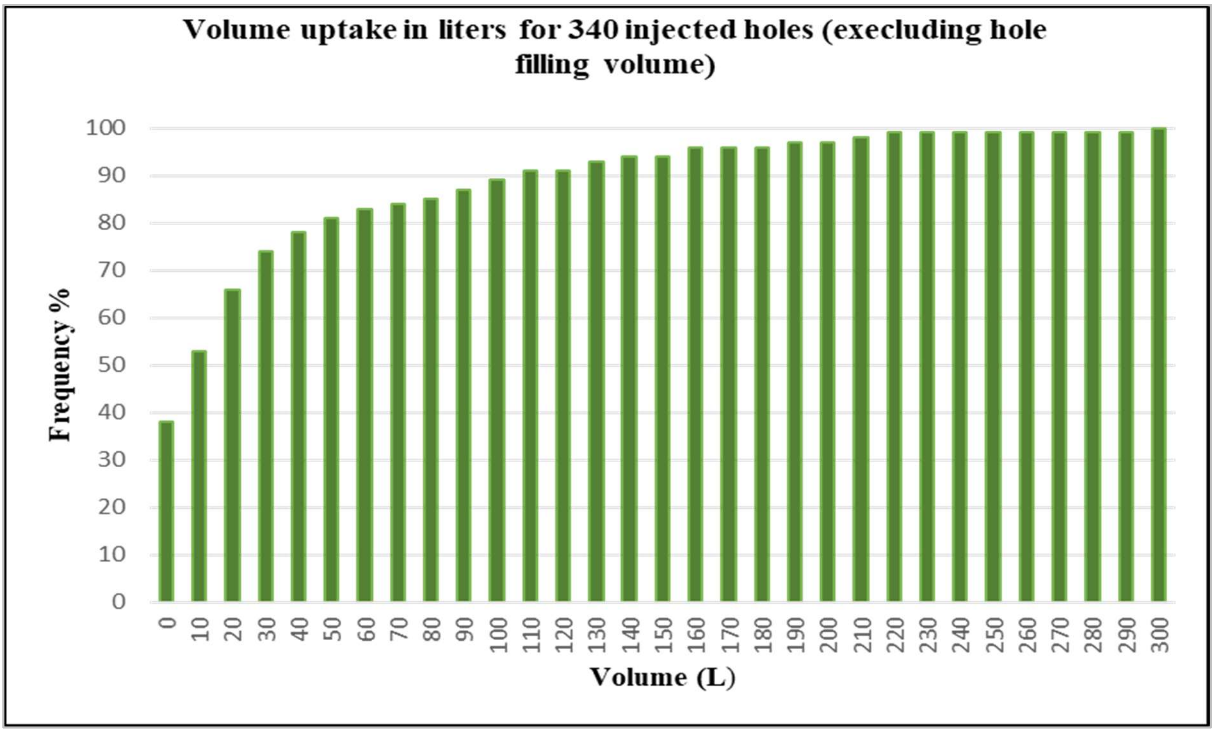


Figure 6.18. Evaluation of typical injection uptake in blue metro- line extension project.

6.4 Evaluation of water ingress after grouting

6.4.1 Comparison between measured flow and prognosed flow

The aim of this chapter is to make sure that the design requirements with regards to application of environmental court were not exceeded. This was done by comparing the measured flow after grouting with the prognosed flow (maximum allowable flow based on design documents). The prognosed flow was calculated to achieve a certain sealing efficiency using the design stop criteria, and to keep the accumulated maximum flow below the control value. The control value represents 90% of the maximum allowable flow. Thus, the measured leakage after grouting shall be less than the prognosed flow. Otherwise, new measures must be taken into consideration. However, it should be mentioned that not all the measured leakage values at the site could have been used for this comparison. Thus, using engineering judgment, we had to choose the most representative points only for comparison purposes. The other disregarded measured values had a high percentage of error due to measurements during rain times, leakage from bolts or other errors at the site.

6.4.1.1 Londonviadukten

As shown in Figure 6.19, the total measured flow is represented by a bar chart until the last evaluated section and compared to the prognosed flow and control value. The control value is represented by the upper limit of the colored zones. In the same figure, the measured flow after grouting was less than the prognosed flow, and a trend line of the measured flow values (dashed line) indicates that the real flow will continue to be under the prognosed flow which means that the accumulated flow will be less than the control value of 102 L/min. This means that the designed stop criteria in Londonviadukten give satisfactory results. It shall be however mentioned that in weakness zones, we only reached good results after 5 rounds of grouting, and the percentage of boreholes that stopped by volume were high, which indicates that the maximum volume as a stop criterion shall be increased to 1000 L instead of 500 L. At large depth fans (water pillar larger than 70m), the measured flow was also less than the prognosed which is good. But, at fan 607, the flow suddenly increased. This fan is located at a higher water depth (water pillar larger than 90m). This means that there are higher risks of having high leakage at zones located with large water depth of more than 90m. This is also another indicator that the stop criteria at large depths must be changed in Londonviadukten, only for fans that has a water pillar larger than 90m.

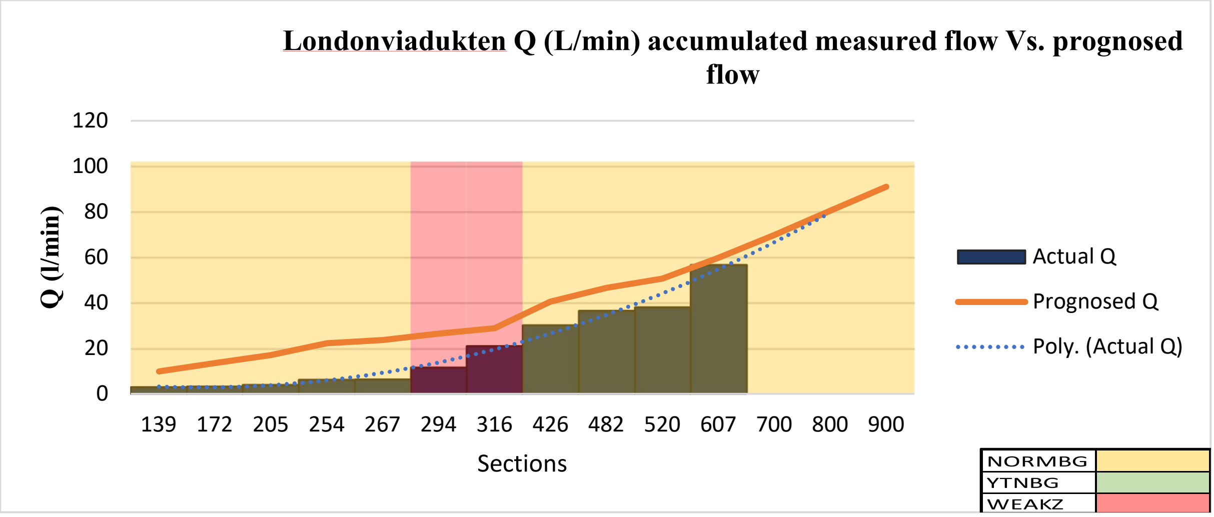


Figure 6.19. Londonviadukten accumulated measured flow Vs. prognosed flow in (L/min).

In Figure 6.20 below, the measured flow after grouting and the maximum allowable leakage were recalculated per 100m to check the how effective this grouting process can be for main tunnels. All the evaluated sections were under the allowable except for weakness zones which were from chainages 0+267 to 316. This means that if IK3 weakness zone was extended over a large distance and was injected using the same design stop criteria in the main tunnel, then a high leakage is expected which requires to stop the construction and perform post grouting. From the figure, it can be summarized the followings:

- In normal rock (+267 to 0+294), the leakage after grouting is exceeding the maximum allowable inflow according to the application to the environment court. Fans analyzed in this section are: 267 and 282. Fan 267 was injected with one round and fan 282 with three rounds because it needed extra grouting rounds. According to the geological mapping, these fans contain clay and crushed zones, which might be the reason got the high leakage values. This represents risks if these geological conditions extend over a larger distance.
- In weakness zone (+294 to 0+316), there is a high risk for high leakage if this area extends over a larger distance. However, it shall be mentioned that at fan 294, the leakage values after grouting were achieved after 5 rounds of grouting.

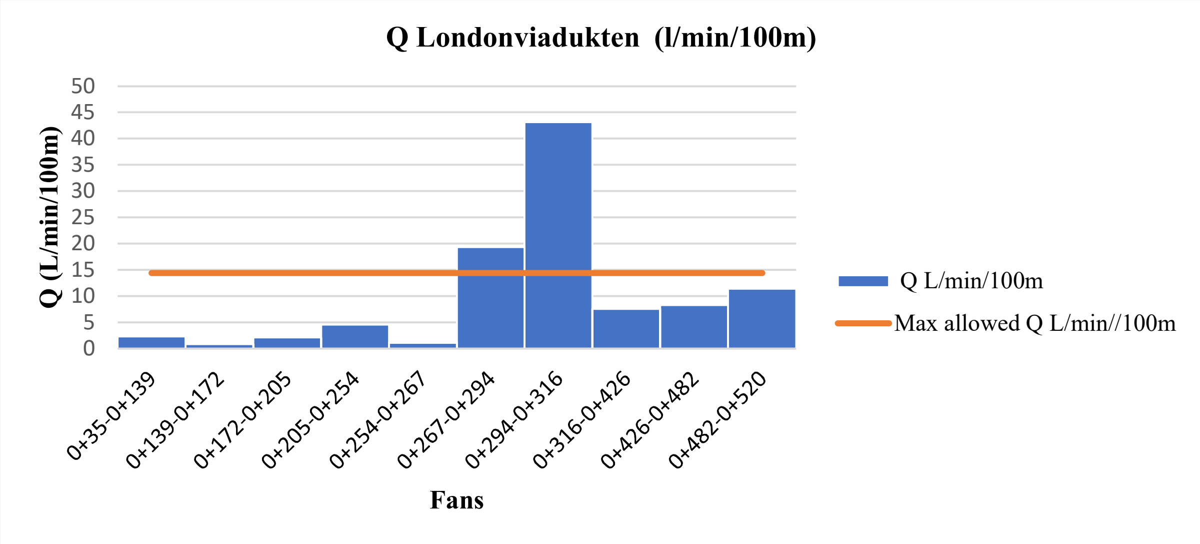


Figure 6.20. Londonviadukten measured flow Vs. maximum allowed flow in (L/min/100m).

6.4.1.2 Järla Östra

Few representative leakage measurements were chosen for evaluation at Järla Östra access tunnel as shown in Figure 6.21 below. The total accumulated flow until the last implemented section (280) is almost same as the prognosed flow. However, there is an indication that the real flow measurements can actually exceed the prognosed flow up to the last section in Järla Östra, which means that it can exceed the control value of 25.6 L/min. In such case, the stop criteria must be reviewed and adjusted. Also, as concluded from the previous chapter, 13% of holes were stopped by volume in surface rock domain in IK1 zones, which means that the stop injected volume in surface rock should have been increased or the typical injection class should have been changed to IK2.

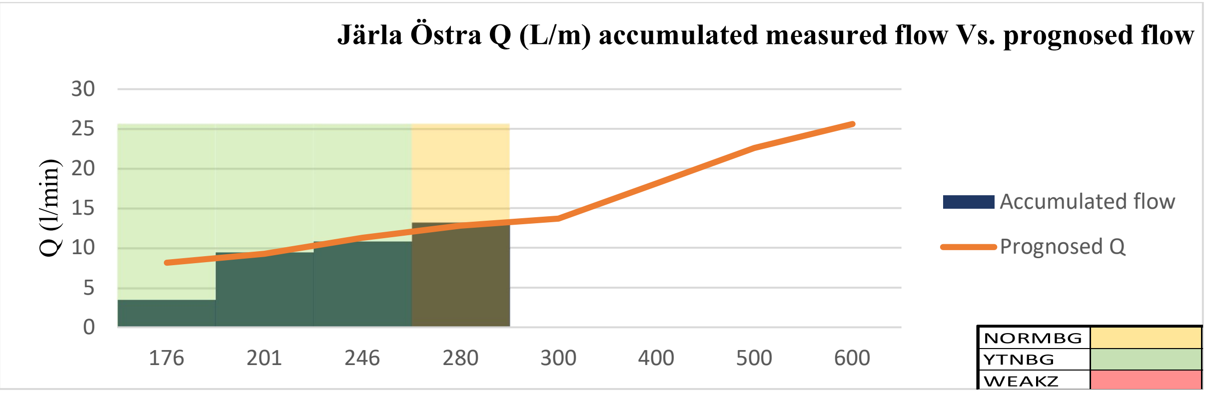


Figure 6.21.Järla Östra accumulated measured flow Vs. prognosed flow in (L/min).

In Figure 6.22 below again, the measured flow after grouting and the maximum allowable leakage were recalculated per 100 m to check the effectiveness of this grouting process to be either used or adjusted for main tunnels. All the evaluated sections were under the allowable except for chainages 0+176 to 0+201 (surface rock domain). This means that if the design stop criteria as shown in Appendix (A) were used, there will be a risk if the surface rock domain was extending over a larger span of 100 m or more in main tunnels. This assures that the stop criteria for surface rock domain must be improved by adding by prolonging the grouting time or performing additional grouting rounds. According to the hydrogeological investigations, the surface rock domains have high hydraulic conductivity, which is estimated to be $1*10^{-7}$ m/s for ungrouted zones. However, it could be higher or lower according to Figure 4.14 in Chapter 4. The leakage after grouting for surface rock zones is different at each section below. The measured leakage values were: 3.5, 6.0 and 1.4 L/min for each section below respectively.

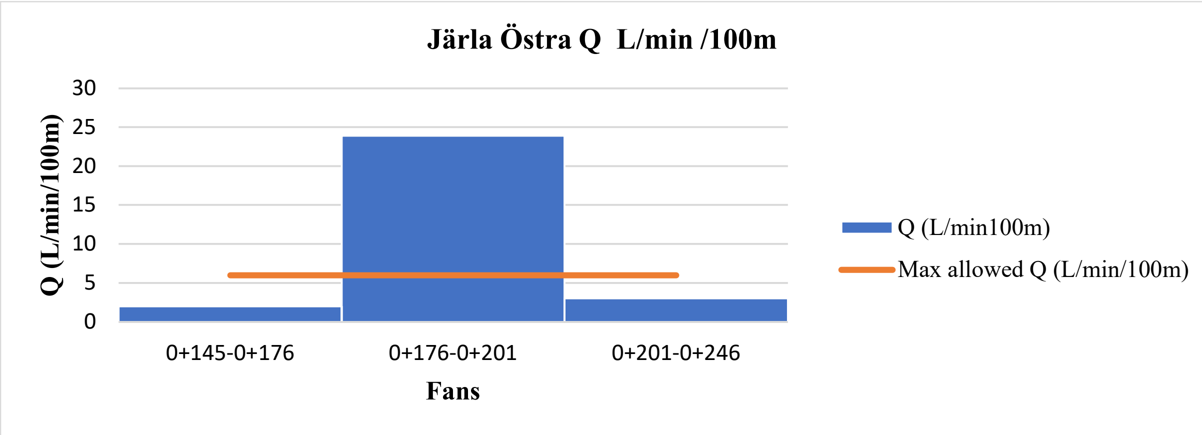


Figure 6.22.Järla Östra measured flow Vs. maximum allowed flow in (L/min/100m).

6.4.1.3 Sundstabacken

For Sundstabacken, as shown in Figure 6.23, the measured accumulated flow was way much lower than the prognosed flow and the control values. It was the most completed tunnel at the site and 96% of the sections were evaluated. The results here were really promising because they show no risk of exceeding the maximum allowable limit.

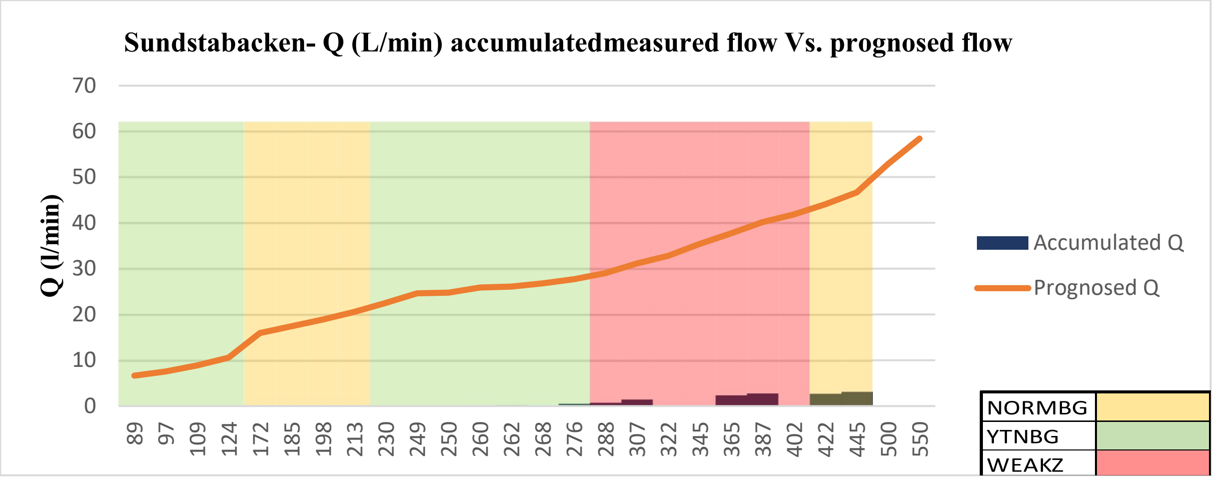


Figure 6.23.Sundstabacken accumulated measured flow Vs. prognosed flow in (L/min).

Thus, if the stop criteria used in this access tunnel is applied on the main tunnel, there is low risk that post grouting will be needed. The results here show that Sundstabacken had generally good geological conditions, which also justify the reasons why this tunnel had less grouting and that no boreholes were stopped by volume as was calculated in the previous chapter. Figure 6.24 below also shows that the risks in Sundstabacken were low. The existence of the weakness zone in Sundstabachen did not affect the grout uptake as it was in Londonviadukten, which can be justified by the fact that weakness zone in Londonviadukten had unfavorable conditions with rock class D, but weakness zone in Sundstabacken had better rock quality C. Also, the classification of weakness zones depends on other parametrs of cracks' endurance, direction and cracks' fillings.

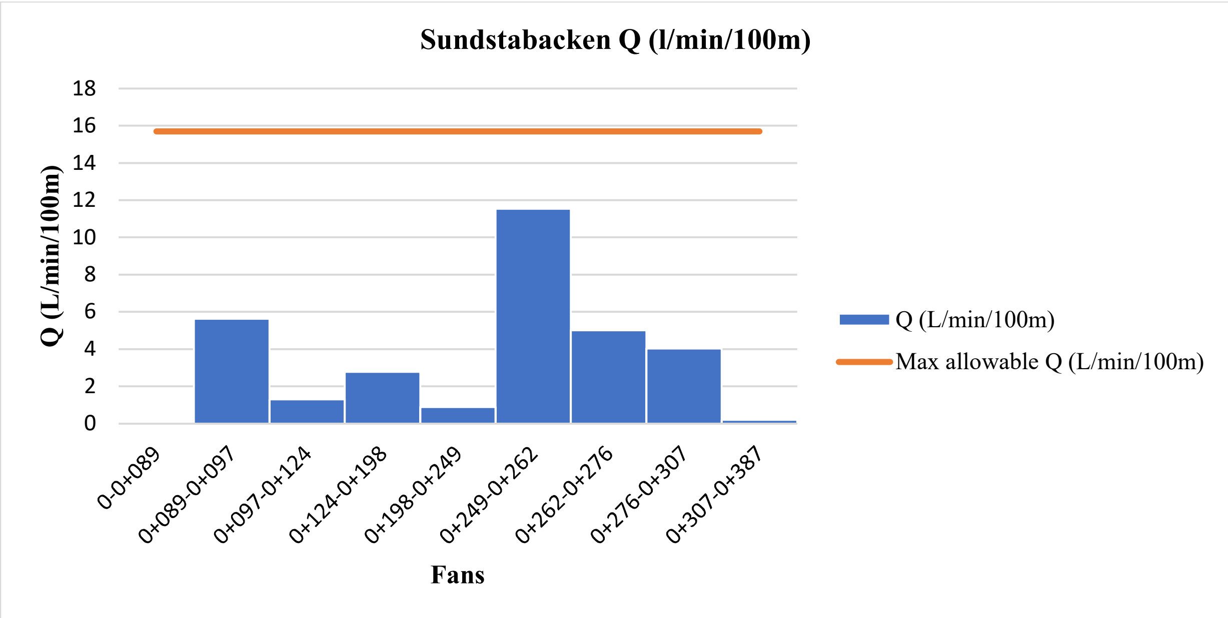


Figure 6.24. Sundstabacken measured flow Vs. maximum allowed flow in (L/min/100m).

6.4.1.4 Hammarby Fabriksväg

Few numbers of representative leakage measurements were chosen for evaluation at Hammarby Fabriksväg access tunnel as shown in Figure 6.25 below. The percentage of work done in this tunnel was only 35% at the time of analysis. The analyzed measurements were done in surface rock domain, which is known to have high crack intensity and thus high permeability. The results here were also promising because they show no risk of exceeding the maximum allowable limit. These fans are IK3 geometry zone, at which more than 50% of the holes stopped by time and the others did not take any flow, which assures that the design stop criteria for this access tunnel are quite good. Moreover, Figure 6.26 below shows that the risk for exceeding the requirements per 100 m is low. Therefore, no need to adjust the stop criteria in this access tunnel at least for the evaluated sections.

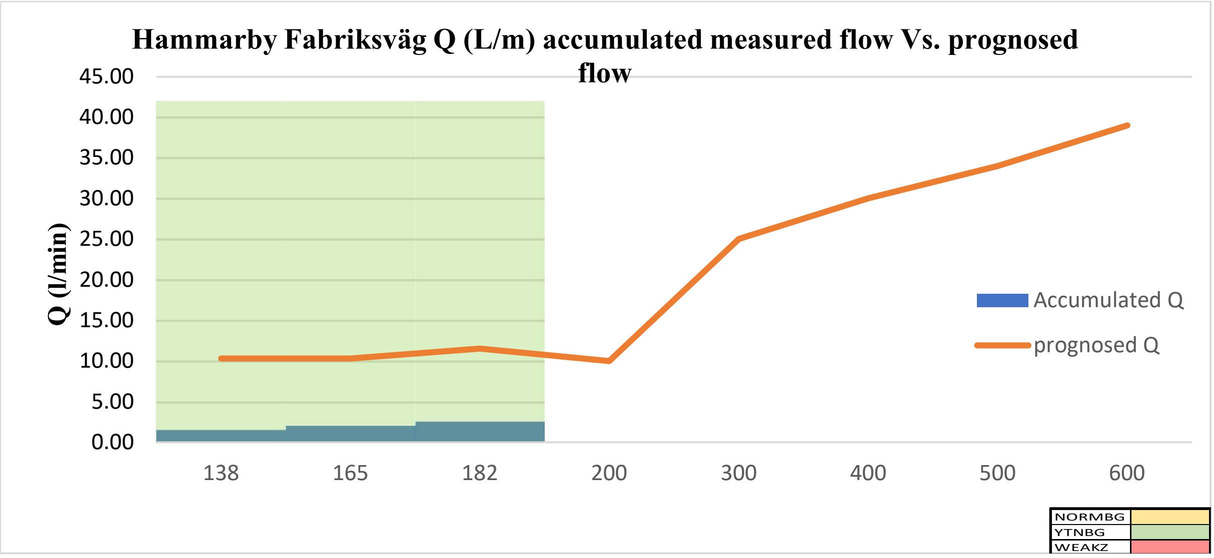


Figure 6.25. Hammarby Fabriksväg accumulated measured flow Vs. prognosed flow in (L/min).

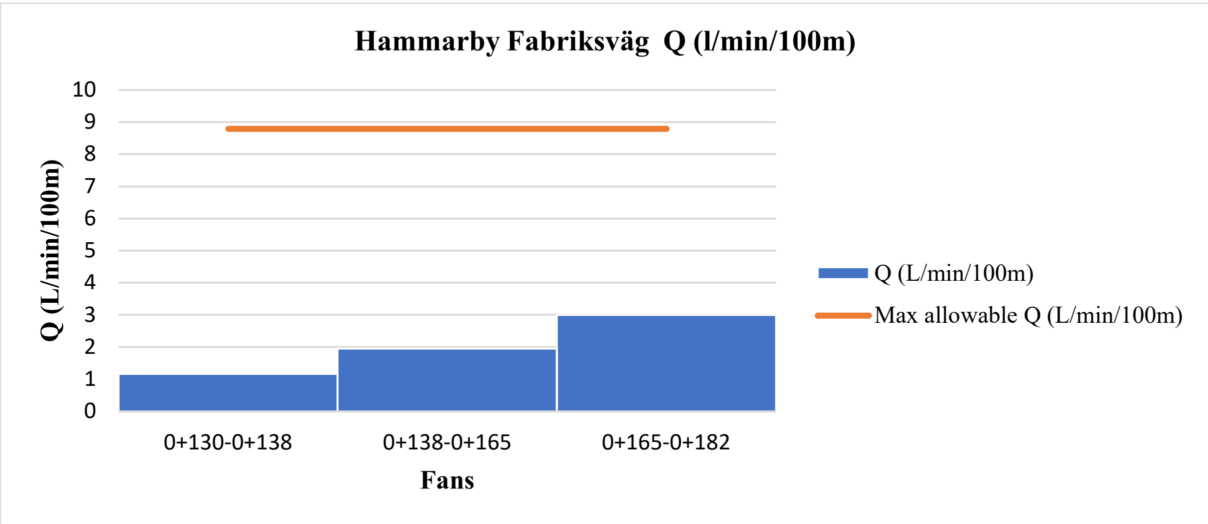


Figure 6.26.Hammarby Fabriksväg measured flow Vs. maximum allowed flow in (L/min/100m).

6.4.1.5 Värmdövägen

As shown in Figure 6.27 below, the representative measured leakage values were in normal rock domain. Few measurements were selected and considered representative because the percentage of evaluated sections at this tunnel is only 37%, whereas the other measured points had errors. The fans showed in the figure are in mostly IK1 zones and some were in IK3 geometry zones. It was seen before that 6% of the boreholes were stopped by volume in IK1 zones, and it is now shown that the measured leakage is very close to the prognosed flow and might exceed it in the next fans. Those results indicate that this normal rock area had higher hydraulic conductivity than the other normal rock areas, since the water loss measurements before grouting falls within a range as illustrated in chapter 4.5. Also, using Figure 6.28 below, in the fans from 0+128 to 0+141, the measured leakage exceeds the requirements if the span extends over a larger span, which means that there are some uncertainties in normal rock zones at Värmdövägen, and thus the design stop criteria need to be reviewed for this access tunnel. In these analyzed zones, it was shown that the grout uptake was generally between 1-3 L/m in Figure 6.9 and that the time stop criteria were applied in Figure 6.15. This leads to the necessity of making deeper investigation of the geological properties of this normal rock zone. According to the geological mappings, there were high number of altered/weathered cracks, little clay, and weaker rock mass quality than prognosed, i.e., rock class (B) instead of (A). In addition to this, there were high number of thin dense fractures. These conclusions from geological mapping can interpret the high leakage. Also, it might be that this zone has higher hydraulic conductivity before grouting than the estimated effective values, since that K_0 falls in a large range.

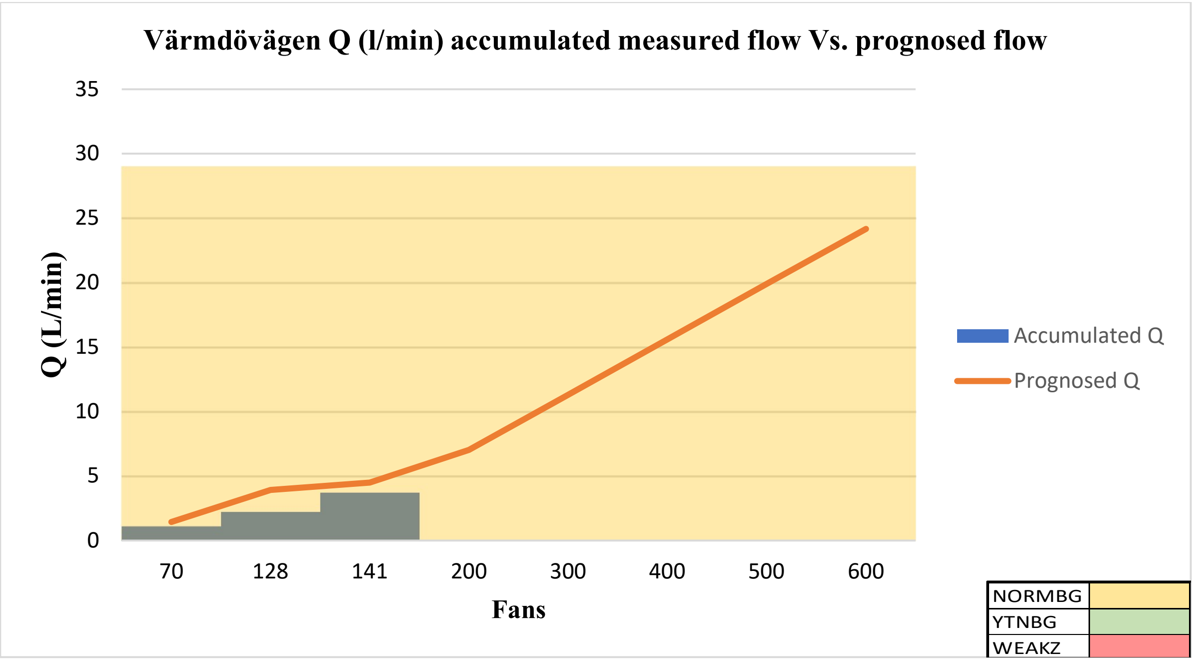


Figure 6.27. Värmdovägen accumulated measured flow Vs. prognosed flow in (L/min).

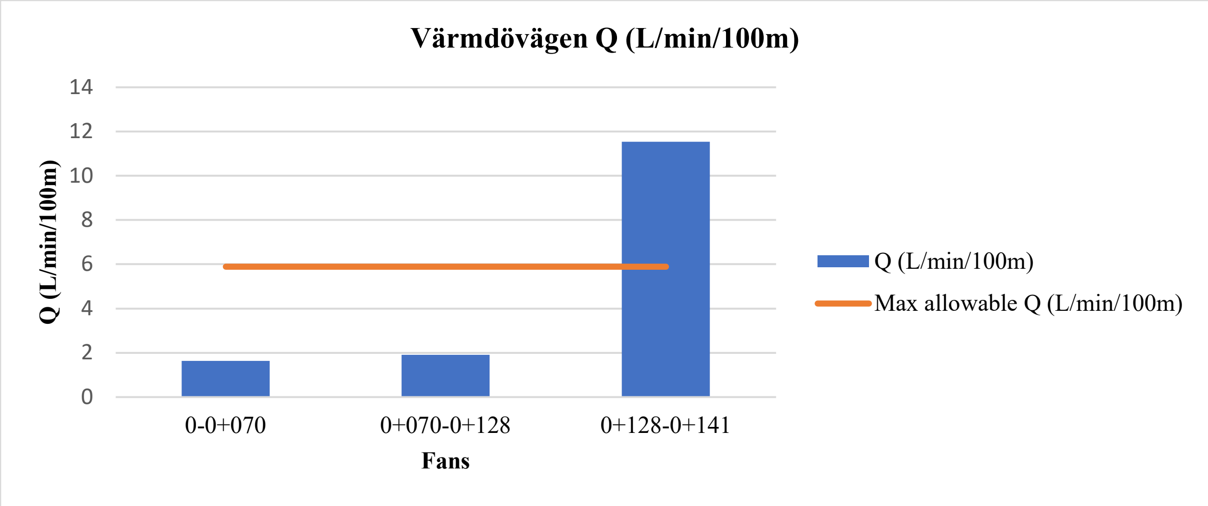


Figure 6.28.Värmdovägen measured flow Vs. maximum allowed flow in (L/min/100m).

6.4.1.6 Skönviksvägen

Only four leakage measurements were recorded by FUT at the time of conducting this study. Two representative measured leakage values, i.e., with less estimated errors, were chosen. In this access tunnel, the studied leakage measurements fall in weakness zone domains as shown in Figure 6.29. The fans are mainly in IK2 zones. It was seen before in Figure 6.16 that 3% of the boreholes were stopped by volume in IK2 zones, and it is now shown that the measured leakage is very close to the prognosed flow and might exceed it in the next fans. Those results indicate that it would be better apply in this weakness zone a specific technical solutions IK3 instead of IK2 with higher grout injected volume and time. In Figure 6.30, the measured leakage in weakness zones was still below the maximum permissible water inflow per 100 m, but since only two leakage measurements at two fans were evaluated, it is hard to judge if the stop criteria in Skönviksvägen was successful or not.

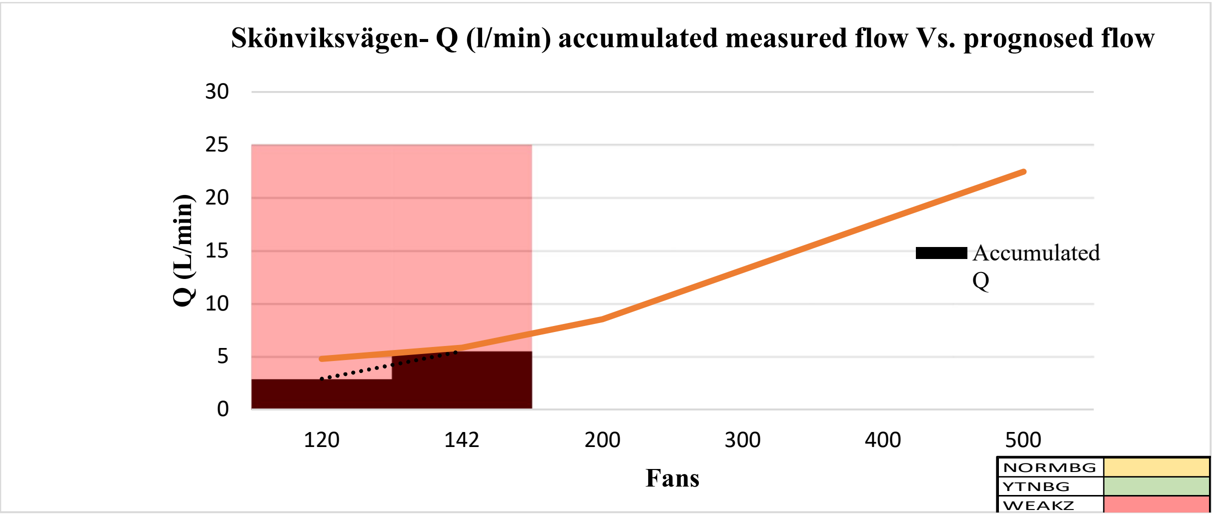


Figure 6.29. Skönviksvägen accumulated measured flow Vs. prognosed flow in (L/min).

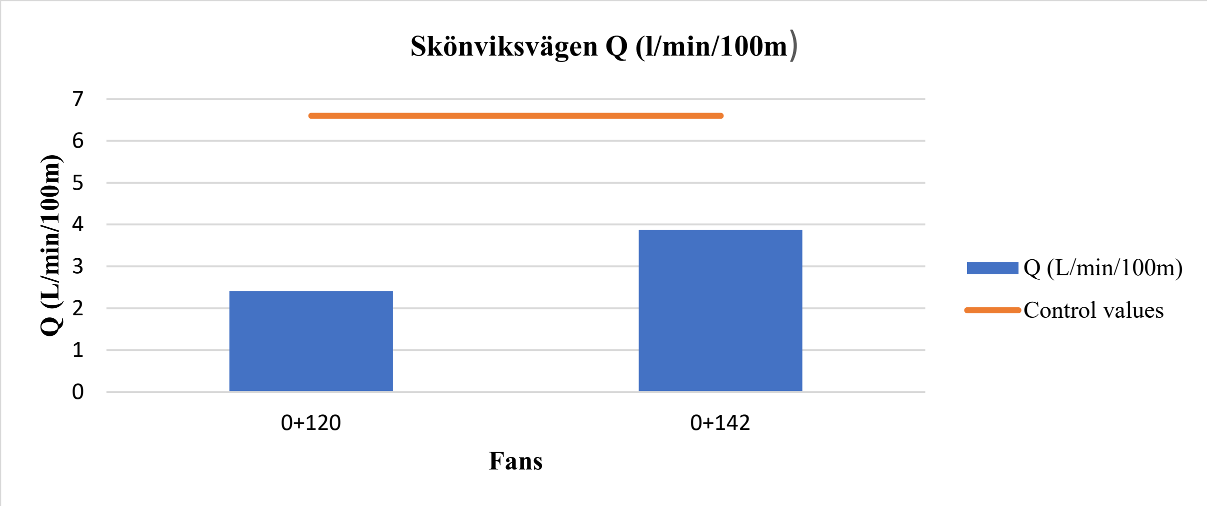


Figure 6.30.Skönviksvägen measured flow Vs. maximum allowed flow in (L/min/100m).

6.4.2 Summary of results

Generally, the design stop criteira shown in Appendix (A) chapters A.1 and A.2 are achieving good results, where all of the leakage measurements so far were below or very close to the prognosed flow and thus below the control values. The accumulated inlekage vs. the prognosed accumulated inleakage for the last evaluated section are summarized below:

- In Londonviadukten, the accumulated leakage was 56.8 L/min which is lower than the prognosed leakage of 59.8 L/min at the last evaluated section. The measured leakage values were following and satisfying the prognosed leakage. The achieved results satisfy the requirements in the application to the environmet court, i.e. maximum permissible water leakage is 14.4 L/min/100 m. But, if weakness zones would be extentded over a larger span, then it is recommended either to change the designed stop criteria The results were achieved by following the designed stop criteria in Appendix A for typical injection zones and weakness zones. However, weakness zones only satisfied the requirement after 2-5 rounds of grouting. Grout uptake in weakness zones reach up to 59 L/m execluding hole and hose filling volume.

- In Järla Östra, the accumulated leakage was 13.25 L/min which is slightly higher than the prognosed leakage of 12.8 L/min at the last evaluated section. Surface rock domain was dominant in the analysed sections, which can be the reason ehy leakage after grouting was high. Surface rocks usually have a hydraulic conductivity between $7*10^{-7}$ up to $1*10^{-5}$ or even more as shown in Figure 4.13 in chapter 4.5. The span of hydraulic conductivites for surface rock zones have a standard deviation of $1.6*10^{-6}$. Having surface rock extended over a larger span might cause problems and will exceed the requirement according to the application to the environmental court (6 L/min/100m).

- In Sundstabacken, the accumulated leakage was 3.2 L/min which is much lower than the prognosed leakage of 46.7 L/min at the last evaluated section. The measured leakage values were much lower than the prognosed leakage. The achieved results satisfy the requirements in the application to the environmet court, i.e. maximum permissible water leakage is 15.7 L/min/100m. In this access tunnel, it was prognosed that rock class D will appear which actually did not apear according to geological mapping, which can be the reason for achieving such good results and high sealing effeciency. Crack intensity in this access tunnel was normal compared to other access tunnels. Thus, it can be that this area has lower conductivity than other areas in the other access tunnels.

- In Hammarby Fabriksväg, the accumulated leakage was 2.6 L/min which is lower than the prognosed leakage of 11.5 L/min at the last evaluated section. The measured leakage values were following and satisfying the prognosed leakage. The achieved results satisfy the requirements in the application to the environmet court, i.e. maximum permissible water leakage is 8.8 L/min/100m.

- In Värmdövägen, the accumulated leakage was 3.75 L/min which is lower than the prognosed leakage of 4.5 L/min. The measured leakage values were following and satisfying the prognosed leakage. So far, the achieved results satisfied the requirements according to the environent court, i.e. maximum permissible water leakage is 5.9 L/min/100m. But, if this normal rock domain extended over 100m as between fans 0+128 and +0141, then the requirements might be exceeded. As mentioned before, there is an indication that this normal rock zone here has higher hydraulic conductivity than the estimated effective hydraulic conductivity. Also, due to the fact that the actual rock mass quality was weaker than prognosed, and the existence of thin dense cracks, then it can be said that this access tunnel has higher risks in normal rock zones compared to other tunnels.

- In Skönviksvägen, the accumulated leakage was 5.5 L/min which is slightly lower than the prognosed leakage of 5.8 L/min at the last evaluated section. The measured leakage values were following and satisfying the prognosed leakage. The achieved results satisfy the requirements in the application to the environmet court, i.e. maximum permissible water leakage is 6.6 L/min/100m

Based on the summary, the design stop criteria can be used for other similar projects and in main tunnels work. However, some recommendations are made regarding stop criteria for some fans to better optimize the grouting process and assure achieving required sealing effeciency.

Recommendations:

1- Stop criteria in typical injection class can be used in future projects, taking into consideration additional grouting rounds and mix change in case the prescribed pressure is not reached, or the number of boreholes that are stopped by volume is higher than a certain number.
2- In weaknesss zones, with similar conditions as in Londonviadukten that has poor rock quality of class D, with high unfavorable conditions regarding cracks filllings, the stop criteria are recommended to be changed where the maximum volume can be set to be 1000 L instead of 500 L.Complementary grouting actions must all of the time be taken into account.
3- For injection at large depths (large water pillar), it was noticed that when the water pillar is higher than 90 m, the leakage measurements were increased significantly. Thus, it is recommended to increase the maximum volume as a stop criteria to be 500 L at large depths (more than 90 m)instead of 300 L.There is a high possibility of erosion, i.e. the yield stress of grout is not sufficient to resist water pressure, which means grout penetrations and spread is not sufficient. Therefore, larger spread is needed and grouting time is also recommended to be increased.
4- For surface rock zones with high hydraulic conductivity, such as in Järla Östra, the stop criteria are recommended to be adjusted since the real leakage measurements were so close to the prognosed flow, which might lead to exceeding the requirements for leakage according to the application to the land and environmental court. Typical injection for surface rocks is recommended to be IK2 instead of IK1. In case, IK1 is a must, then it is recommended to increase the grouting time, for example to 15 minutes instead of 12 minutes, with larger maximum stop volume criteria of 500 L instead of 300 L.

6.5 Evaluation of sealing efficiency

The estimated flow before grouting (L/min) was calculated using the design effective hydraulic conductivities (K_o) from water loss measurements in chapter 4 Table 4.9. By comparing the estimated flow before grouting with the measured flow after grouting, results showed that the measured flow after grouting exceeds the estimated flow before grouting for some fans as shown in Figures 6.31 to 6.36. This is not acceptable ,because grouting process aims to seal the fractures and reduce the water flow to a permissible level. These unlogical results can justified with many reasons. First reason can be that the measured leakage values are not correct and thus the process of measuring the water flow after grouting shall be improved at the site. Second probable reason is that the estimated flow before grouting was not correct because it was calculated using effective hydraulic condutivity. Hydraulic conductivity before grouting is not known as a specific number but it falls within a range of values for each domain as mentioned earlier. Thus, using only effective hydraulic conductivity does not give useful values to calculate the sealing efficiency to evaluate the grouting process effectiveness. Therefore, it is recommended to make better estimation of the hydraulic conductivity before grouting. The conductivity must be verified with observations and investigations.

Sealing effeciency can be calculated using equation (5-1) in chapter 5, but with having uncertainities in quantifying K_o ,it was difficult to calculate the sealing efficiency and compare it with the required values since in some fans K_o was larger than K_i.

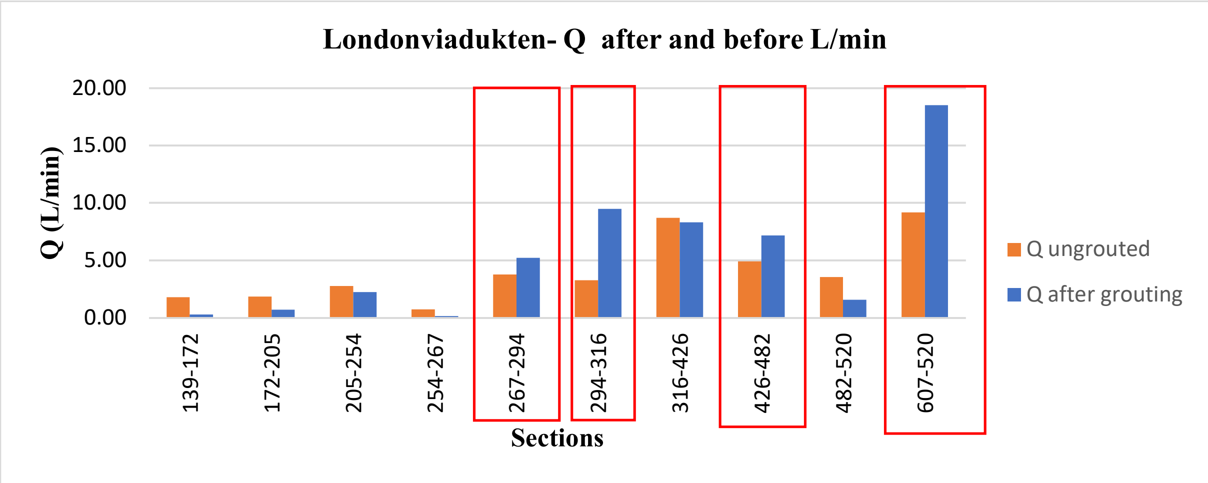


Figure 6.31.Londonviadukten estimated flow before grouting Vs. The measured flow after grouting for evaluated sections.

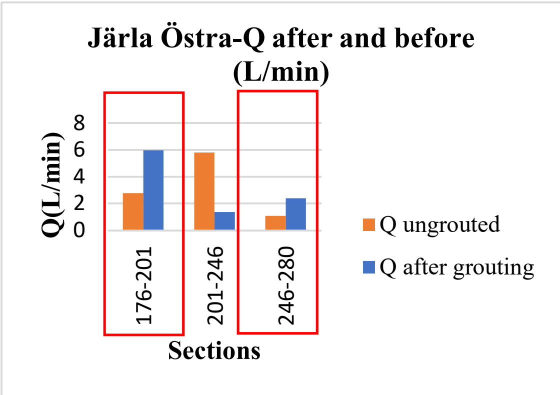


Figure 6.32. Järka Östra estimated flow before grouting Vs. The measured flow after grouting for evaluated sections.

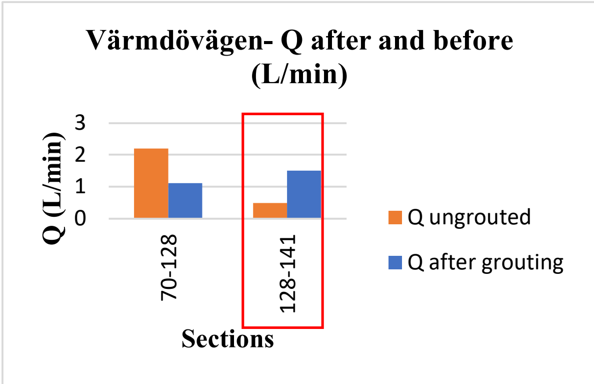
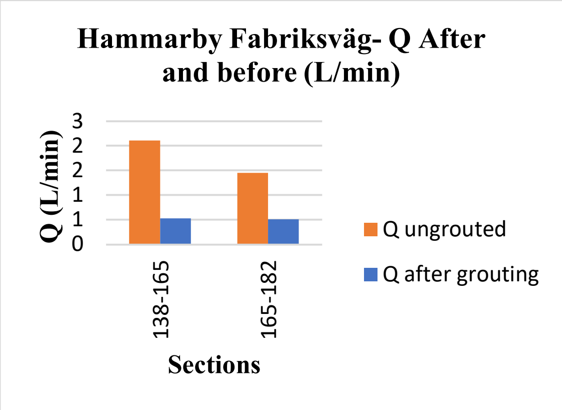


Figure 6.33. Värmdovägen estimated flow before grouting Vs. The measured flow after grouting for evaluated sections.



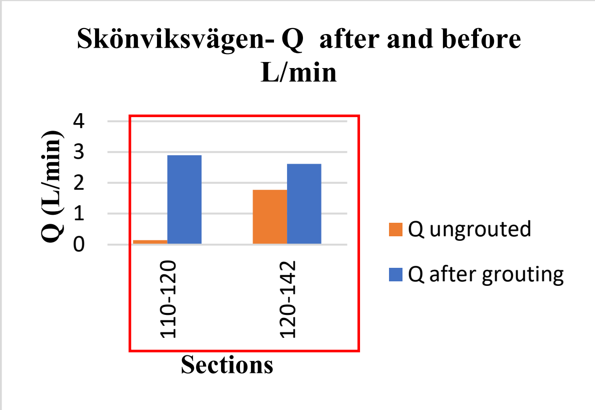


Figure 6.34. Hammarby Fäbriksväg estimated flow before grouting Vs. The measured flow after grouting for evaluated sections.

Figure 6.35. Skönviksvägen estimated flow before grouting Vs. The measured flow after grouting for evaluated sections.

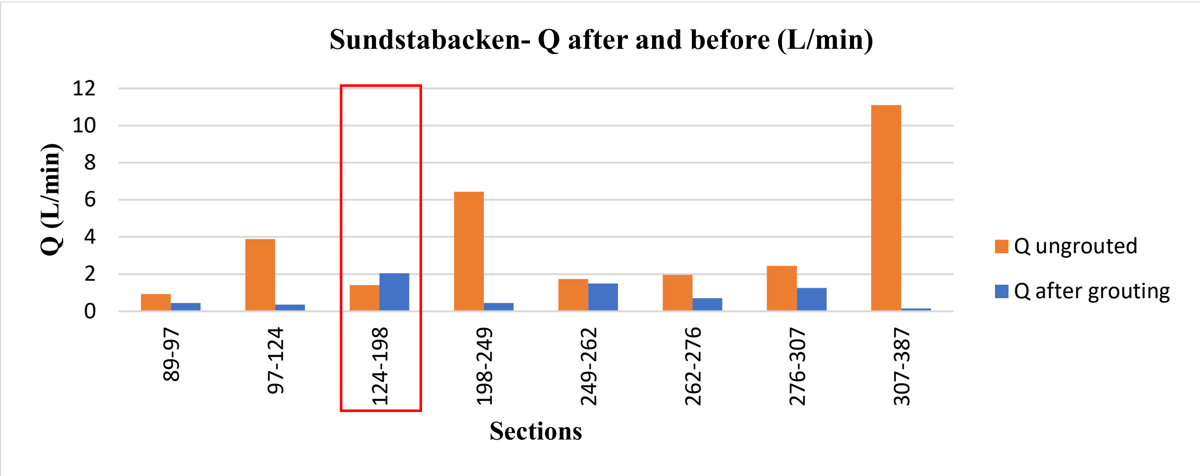


Figure 6.36. Sundstabacken estimated flow before grouting Vs. The measured flow after grouting for evaluated sections.

To solve this problem, estimations for new effective hydraulic conductivities (k_o) were made using the values in Figures 4.11 to 4.14 in chapter 4. Some of the effective values were mainly replaced by arithmetic mean values to make a rough estimation of the achieved hydraulic conductivities after grouting using equation 3-1 in chapter 3. Figures 6.37 to 6.42 show the corrected estimated flow before grouting. The results obtained here were more logical since the flow after grouting is lower than the estimated flow before grouting.

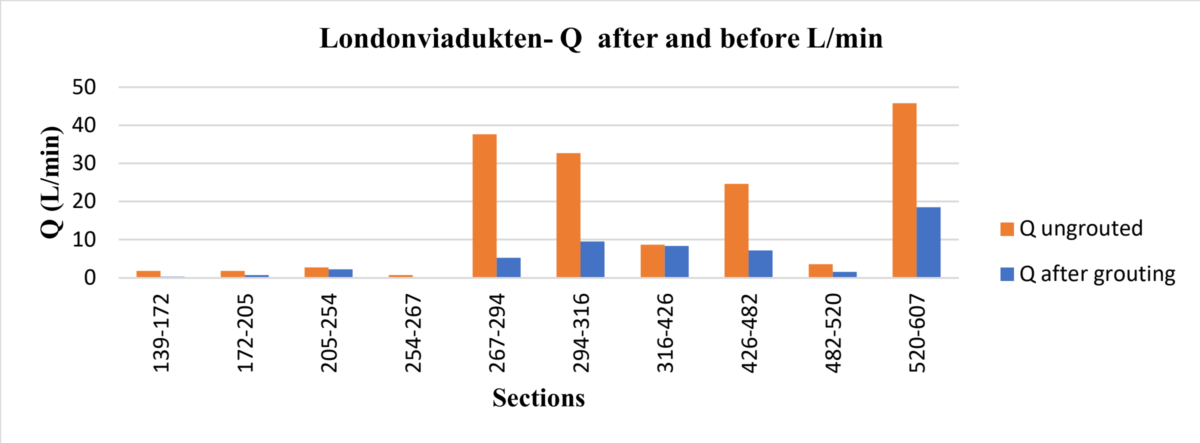


Figure 6.37. Estimated flow before grouting Vs measured flow after grouting for evaluated sections in AT Londonviadukten.

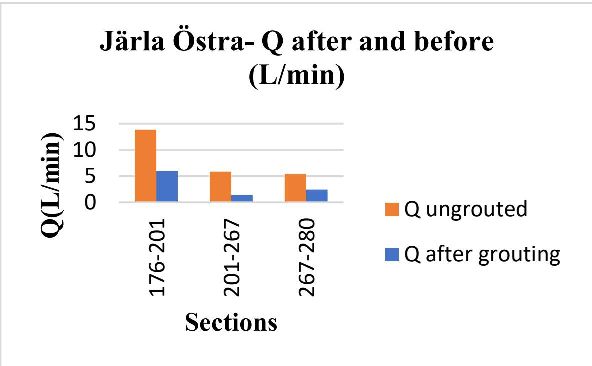


Figure 6.39. Estimated flow before grouting Vs measured flow after grouting for evaluated sections in AT Londonviadukten

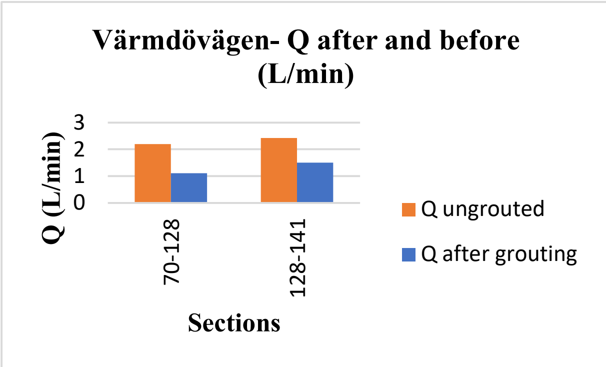


Figure 6.41. Estimated flow before grouting Vs measured flow after grouting for evaluated sections in Järla Östra.

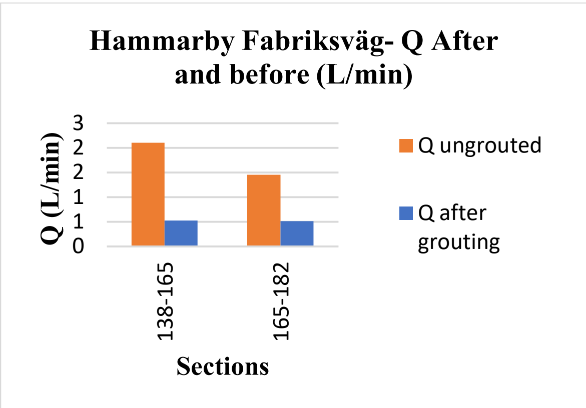


Figure 6.38. Estimated flow before grouting Vs measured flow after grouting for evaluated sections in Hammarby Fabriksväg.

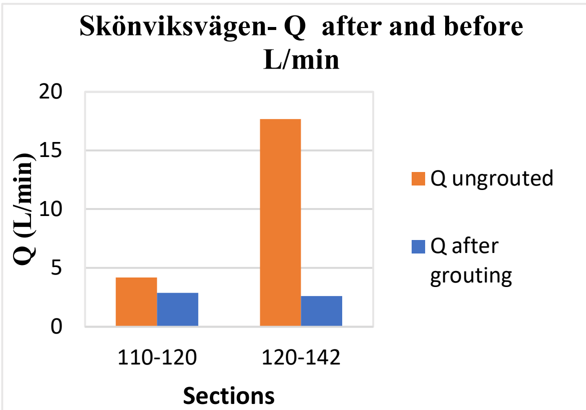


Figure 6.40. Estimated flow before grouting Vs measured flow after grouting for evaluated sections in Skönviksvägen.

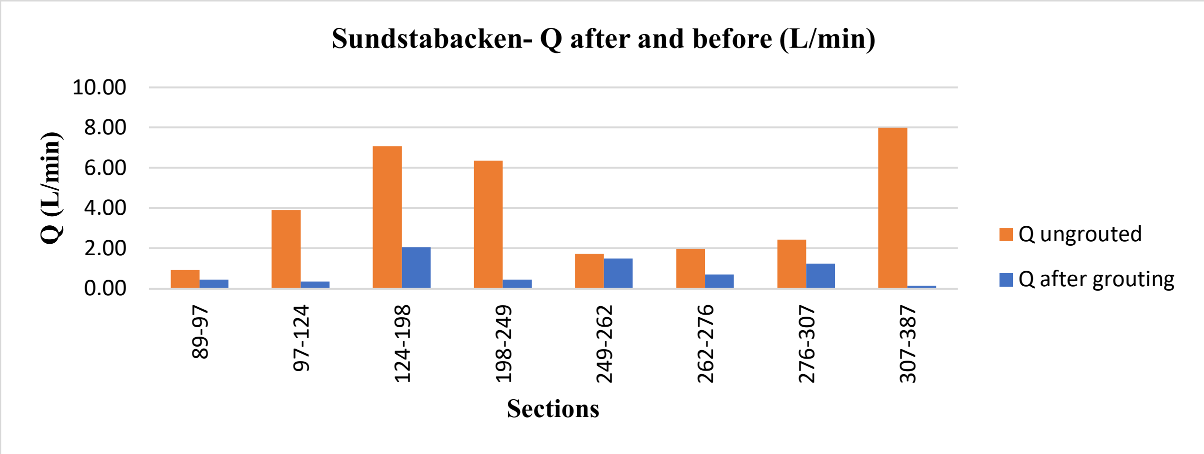


Figure 6.42. Estimated flow before grouting Vs measured flow after grouting for evaluated sections in Sundstabacken.

The estimated achieved hydraulic conductivity was then calculated for each fan as shown in Table 6.7 below. Based on design documents, the expected achieved hydraulic conductivity after grouting in Stockholm area is usually between $1*10^{-8}$ to $2*10^{-9}$. Most of the achieved hydraulic conductivities fall within this range except some fans which achieved a lower hydraulic conductivity (good results) and other fans that achieved higher hydraulic conductivity. The achieved hydraulic conductivities as shown in the table fall between $9.7*10^{-10}$ and $1.03*10^{-8}$. It can be also noticed that in the very unfavorable conditions in the weakness zone of Londonviadukten, the hydraulic conductivity after grouting was achieved after the highest number of rounds (5 rounds). Other zones had a smaller number of rounds. Mostly one round for typical injectrion cases, and two to three rounds in weakness zones. The general conclusion is that it is recommended to make better investigations and controls on the water loss measurements before grouting to have better values closer to the real values. Then, it would be possible to calculate sealing efficiency values. The achieved sealing efficiency was estimated using equation (5-1) and also shown in Table 6.7. By comparing the achieved sealing efficiency with the required sealing efficiency in Table 5.8 in chapter 5, it can be seen that most of the fans had achieved higher sealing efficiency than required. While, in few fans (highlighted in yellow below), the achieved sealing efficiency was lower than required. However, the achieved sealing efficiency here are not trusted. The reason behind this is that they are estimated based on our assumed values of K_0 since the design values of K_0 gave illogical results as was explained in this chapter. Thus, the sealing efficiency can be actually higher or lower.

Table 6.7: Achieved hydraulic conductivity after grouting for each section in the 6-access tunnels. Yellow highlighted fans achieved lower sealing efficiency than required.

Section	Hydrogeological Domain	Assumed effective K_0 based on authors' analysis (m/s)	Ahcieved hydraulic conductivities after grouting (estimated) (m/s)	Estimated achieved Sealing efficiency (%)
Londonviadukten				
139-172	NORMBG	2,0E-08	3,97E-10	98%
172-205	NORMBG	2,0E-08	1,22E-09	94%
205-254	NORMBG	2,0E-08	6,29E-09	69%
254-267	NORMBG	2,0E-08	5,22E-10	97%
267-294	NORMBG until 285 then WEAKZ	5,0E-07	8,51E-09	98%
294-316	WEAKZ	5,0E-07	2,09E-08	96%
316-426	WEAKZ until 320 then NORMBG	2,0E-08	1,39E-08	31%
426-482	NORMBG	1,0E-07	3,91E-09	96%
482-520	NORMBG	2,0E-08	1,43E-09	93%
520-607	NORMBG	1,0E-07	6,11E-09	94%
Järla Östra				
176-201	YTBG	5,00E-07	4,59E-08	91%
201-267	YTBG until 240 thenWEAKZ	1,00E-07	3,80E-09	96%
267-280	NORMBG	1,00E-07	8,78E-09	91%
Sundstabacken				
89-97	YTBG	1,00E-07	1,10E-08	89%

97-124	YTBG	1,00E-07	1,21E-09	99%
124-198	NORMBG	1,00E-07	5,32E-09	95%
198-249	NORMBG until 214 then YTBG	1,00E-07	9,72E-10	99%
249-262	YTBG	1,00E-07	4,54E-08	55%
262-276	YTBG	1,00E-07	6,41E-09	94%
276-307	YTBG until 280 then WEAKZ	5,00E-08	5,66E-09	89%
307-387	WEAKZ	5,00E-08	1,10E-10	99%
Hammarby Fabriksväg				
138-165	YTBG	1,00E-07	4,63E-09	95%
165-182	YTBG	1,00E-07	7,15E-09	93%
Värmdövägen				
70-128	NORMBG	2,00E-08	2,12E-09	89%
128-141	NORMBG	1,00E-07	1,60E-08	84%
Skönviksvägen				
110-120	NORMBG	3E-07	6,59E-08	78%
120-142	WEAKZ	5E-07	1,03E-08	98%

6.6. Correlation between hydrogeological domains and average grout uptake

The evaluation of the grout uptake in relation to the hydrogeological domains are presented in Figures 6.43 to 6.45 below. The evaluation was performed for all fans in all access tunnels for normal rock, weakness zones and surface rock. The grout uptake represents the total volume per meter, excluding hole and hose filling volume.

In Figure 6.43 for normal rock conditions, 53 fans were evaluated, 96% of the boreholes took 4 L/m or less. Afterwards, the uptake was constant until that 100% of boreholes took 13 L/m, however the boreholes that took 13 L/m were in normal rock conditions where there was a passage near existing tunnels in IK3 zones.

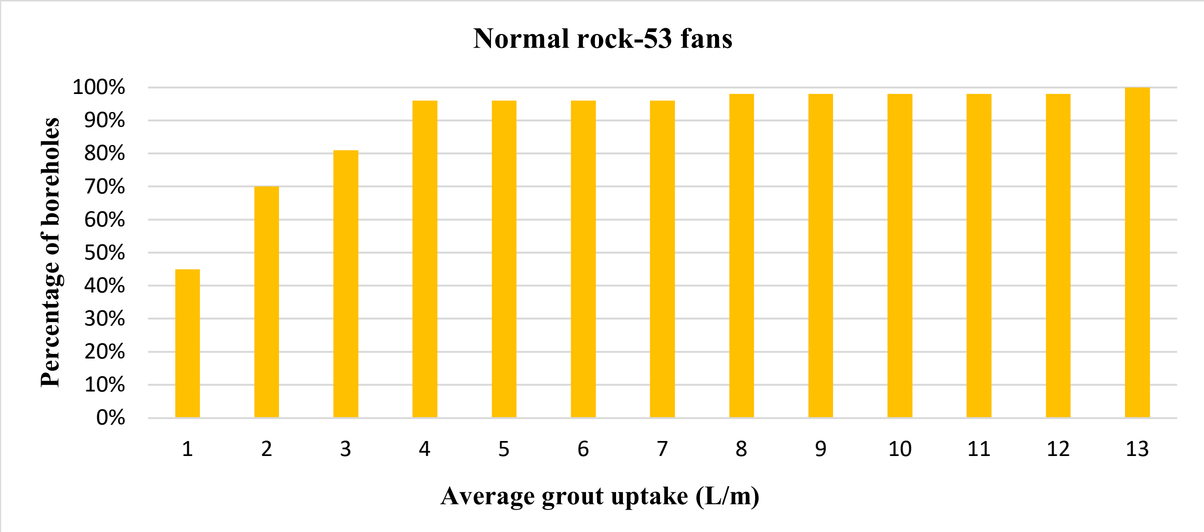


Figure 6.43.Average grout uptake for normal rock.

In Figure 6.44 for weakness zones, 14 fans were evaluated, 88% of boreholes took 18 L/m or less. Afterwards, the uptake was constant until that 100% of boreholes took 53 L/m or less. The extreme boreholes that took this high grout uptake were in Londonviadukten, where this weakness zones had very unfavorable parameters regarding clay filling and very poor rock quality. Therefore, it took 5 rounds of grouting to achieve the required sealing.

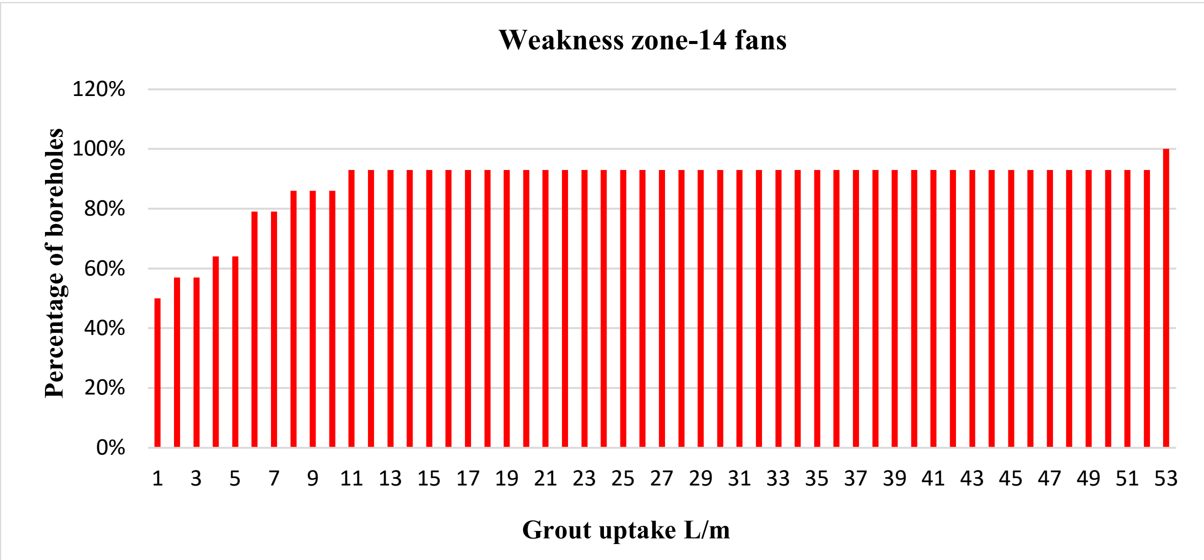


Figure 6.44. Average grout uptake for rock in weakness zone.

In Figure 6.45 for surface rock conditions, 46 fans were evaluated, 94% of the boreholes took 6 L/m or less. Afterwards, the uptake was constant until that 100% of boreholes took 11 L/m or less. However, the boreholes that took more than 6 L/m were mostly in the pre-cut (påslag) zones.

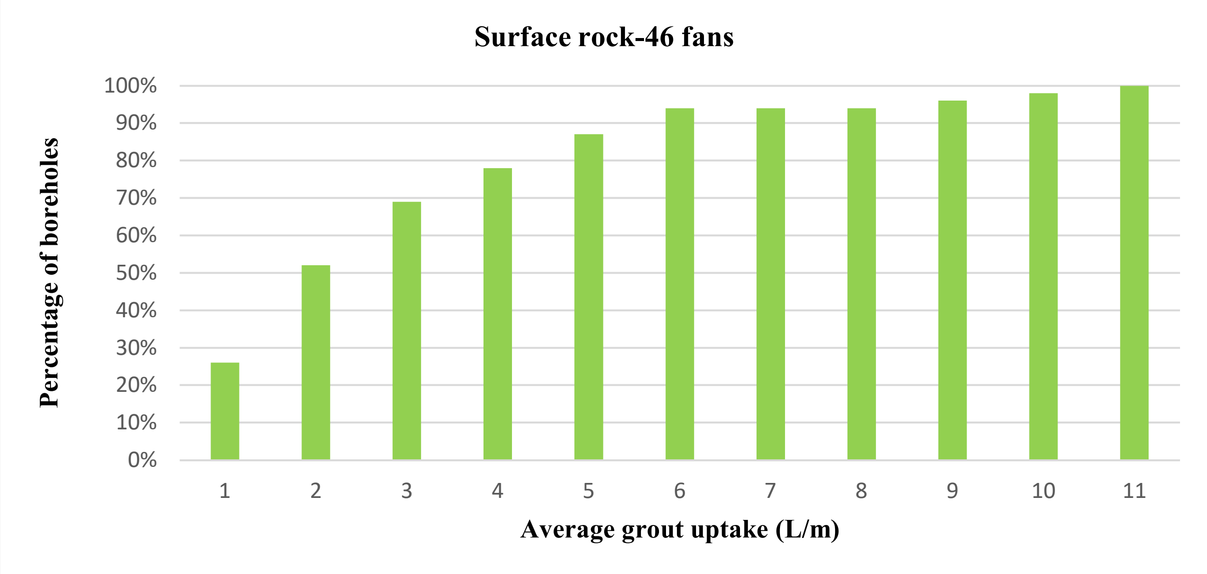


Figure 6.45. Average grout uptake for surface rock.

6.7 Evaluation of grouting data using Real Time Grouting Control Method

6.7.1. Fan 403-Londonviadukten

Data from borehole 16 in Londonviadukten was taken for application. The zone is IK1 zone, with good rock quality class A and falls withing the normal rock zone. The data from the pressure flow loggings were plotted as shown in Figure 6.46 below, and then used to determine the flow dimensionality as a first step in the analysis. The applied stop criteria in this borehole were time. The injection was stopped after around 10 minutes after the borehole filling volume, i.e., 87 L. The dimensionality is shown in Figure 6.47, where the results show that the flow in this borehole is likely to be 1D flow.

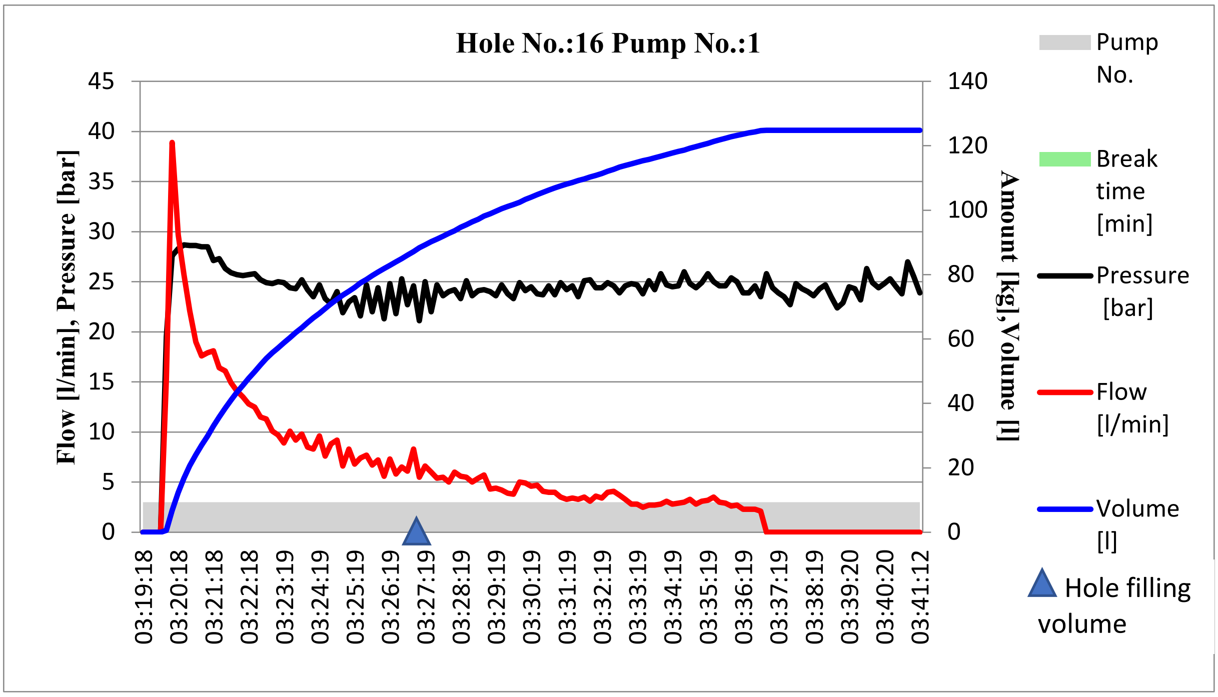


Figure 6.46.Flow and pressure distribution for hole number 16 in fan 403 as a function of time.

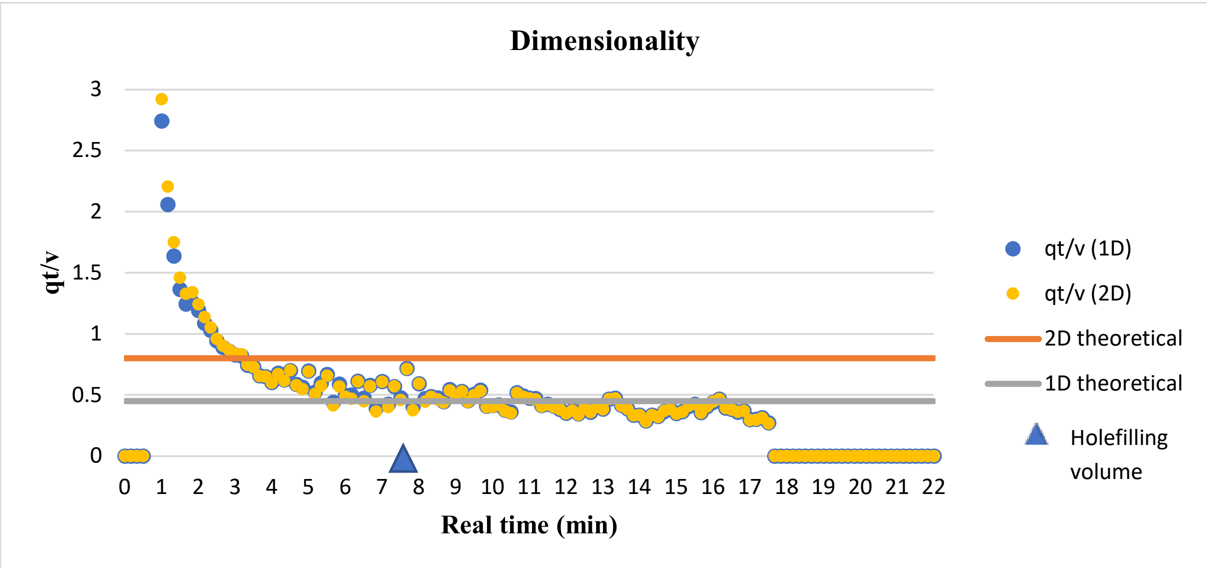


Figure 6.47. Dimensionality indicates that the flow in hole number 16 is likely 1D.

The grouting process continued for almost 10 minutes after the borehole filling, starting from minute 7.7 until the minute 17.7 because the flow was stopped afterwards. The penetration length was then estimated and shown in Figure 6.48 below. The results are satisyting and show that the achieved penetration length was within the limits. The grout penetrated the critical fractures of b_{crit} of 90 µm with 4.6m at the end of the process, which is satisfying because the minimum limit of pentration was set to be 3m according to the design documents. Also, the maximum penetration length was 9.2 m, which also gives good results and satsifies the upper limit that should not be exceeded. In previous sections, the grout in this fan was generally good and the leakage after grouting was less than the prognosed flow, thus achieves good sealing effeciency to reduce the risks of exceeding the permissible water inflow according to the application submitted to the environemtal court. In Figure 6.48 below, we can notice that at minute 15, which is almost 8 minutes after the borehole filling volume, the penetration legth was satisfying and the process could have been stopped earlier. However, the sealing requirements shall be checked to judge if after 8 minutes of grouting, the sealing effeciency was achieved or not, especially that RTGC has many simplifications and assumes that the fracture is a straight void between two plates, excluding the real geological conditions and the shape and inclination of the cracks. Thus, RTGC shows positive results and indications that the grout and stop criteria can be actually optimized, provided that the required sealing effeciency is achieved. Accordingly, RTGC can be used in trail grouting process to optimize the grouting time and volume and come up with new stop criteria. This has been already shown also for some test boreholes in other studies and papers as in Kobayashi et. al. 2008, and in Rafi 2010.

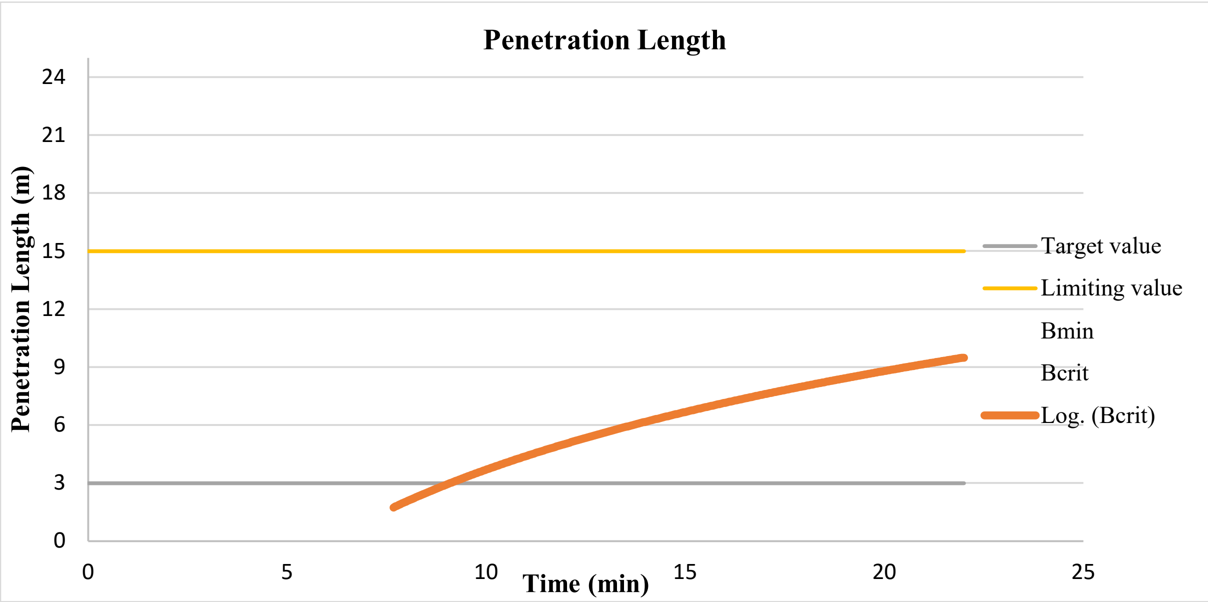


Figure 6.48. The achieved penetration length vs. grouting time.

Since the flow in this borehole is a 1D flow, the recorded flow vs. the $\frac{dI_D}{dt_D} \cdot \frac{1}{t_0} \cdot \left(\frac{\Delta p}{2\tau_0}\right) \cdot \sum wb^2$ was plotted as shown in Figure 6.49. The slope of the regression line represents $\sum wb^2$ in 1D case, noting that the regression line must pass through the zero point. As shown in this case, the $\sum wb^2$ factor is 0.0023, where (w) represents the width of the 1D channel, which is usually taken as 10m.The term represents the fractures' apertures, where the aperture of single joints cannot be evaluated unless the distribution of fractures is known e.g. as a Pareto or lognormal distribution. This factor was then used to calculate the estimated flow and compare it with the real measured flow as shown in Figure 6.50. The results show that the estimated flow is close to the real flow, which assures the validity of RTGC method in good geological conditions.

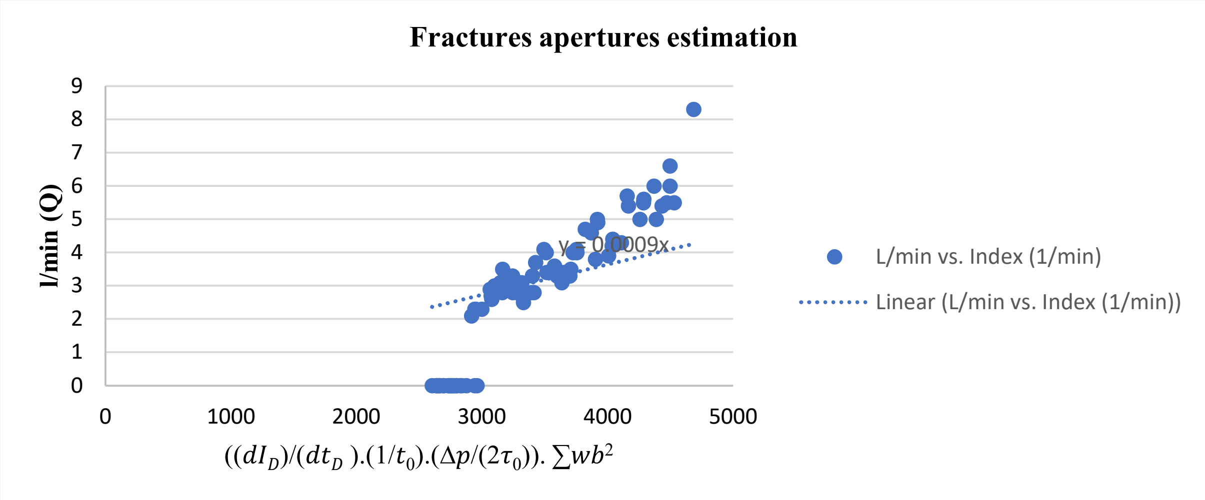


Figure 6.49.Estimation of the fracture's apertures.

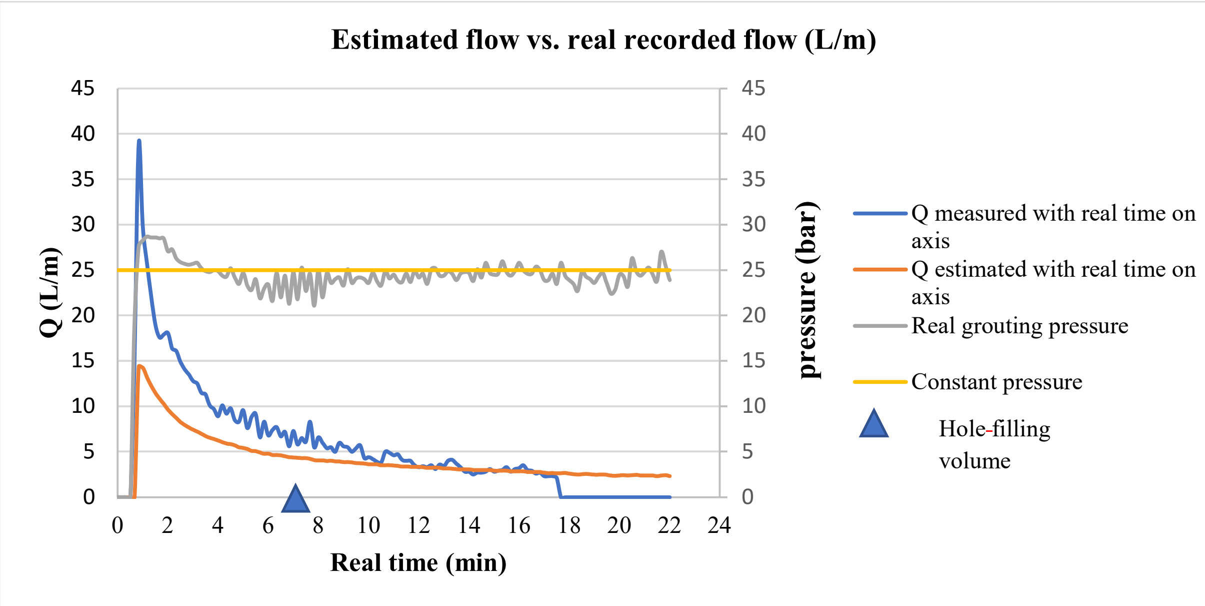


Figure 6.50.Estimated flow Vs. Real recorded flow.

6.7.2. Fan 294 (weakness zone)-Londonviadukten

Data from borehole 24 in Londonviadukten was taken for application, knowing that this hole was injected in four rounds. The zone is IK3 zone, with poor rock quality (class D) and falls within the weakness zone Folkungagatan. The data from the pressure flow loggings were plotted for the first three rounds and shown in Figures 6.51 to 6.53 below. The data was used to determine the flow dimensionality as a first step in the analysis. The total grout time in these three rounds was around one hour and 18 minutes, including borehole and hose filling time. In round one, the borehole filling volume of 51.5 L was reached after approximately 2 minutes, then the borehole consumed the maximum allowed volume of 500 L using mixture one (vct 0.8) and thus stopped by volume. Therefore, the mixture was changed to denser grout with vct of 0.5. This can be seen when the volume uptake line is constant at 500 L, which is when the mixture was changed in the same round 1. Round 2 and 3 were injected only using mixture one (vct of 0.8). The grout uptake in round 2 and 3 was 76 and 199 L, respectively. In round 2 and 3, the injection was stopped by time, at which the injection continued for 15min. It shall be mentioned that in Figures 6.52 and 6.53, the volume does not start from zero because the recorded volume represents the total accumulated volume.

In round 1, the dimensionality is presented in Figure 6.54, with a time span of 47 minutes. Since the estimated dimensionality is 3D, then RTGC here cannot be applied because RTGC was only developed for 1D and 2D flow. The dimensionality of the flow was however of interest to check how the dimensionality will change in each round. Therefore, the dimensionality was also estimated for round 2 and round 3 for this borehole and shown in Figures 6.55 and 6.56. The Figures show that the flow started and 2D and continued at 1D after few minutes in both rounds. In such cases, RTGC can be applied to estimate the penetration length and compare it to required spread.

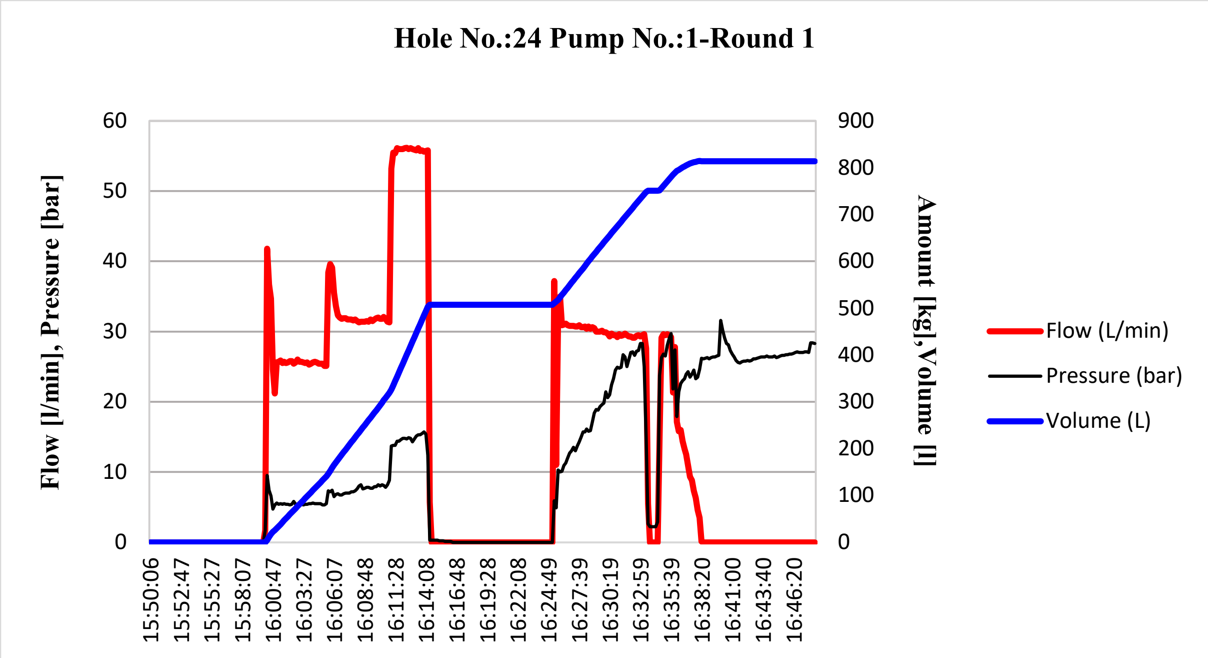


Figure 6.51: Flow and pressure distribution for hole number 24 in fan 294 as a function of time in the first round.

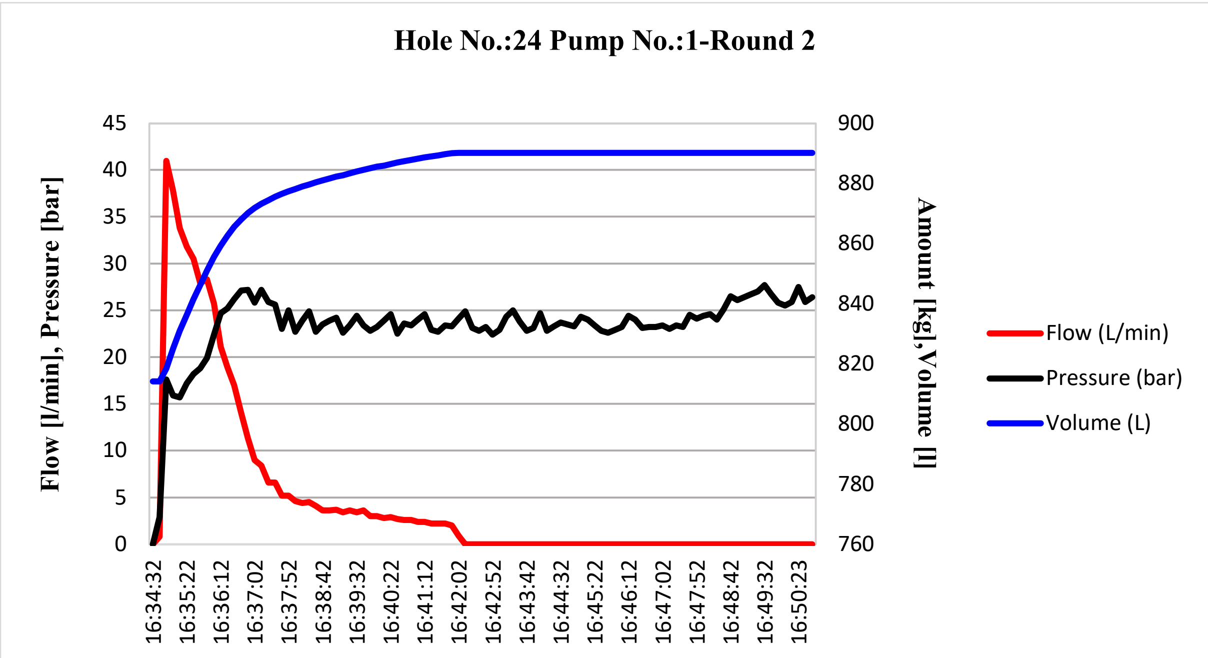


Figure 6.52: Flow and pressure distribution for hole number 24 in fan 294 as a function of time in the second round.

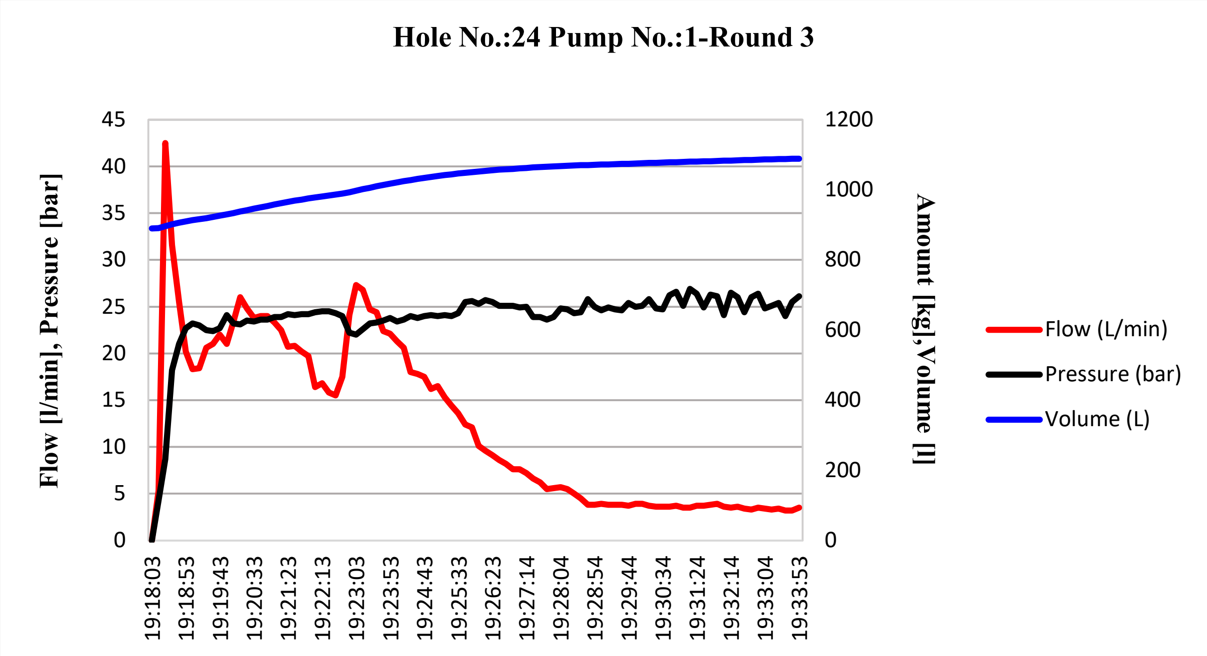


Figure 6.53: Flow and pressure distribution for hole number 24 in fan 294 as a function of time in the third round.

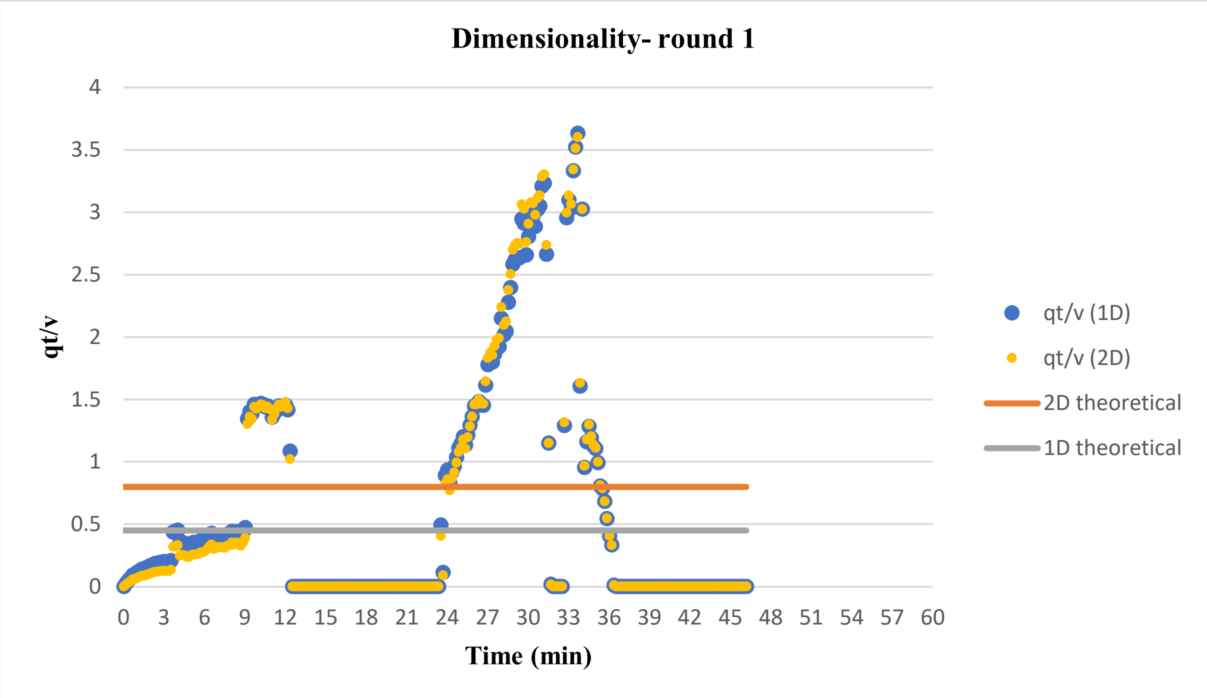


Figure 6.54. Dimensionality indicates that the flow in hole number 24 is likely 3D in the first round.

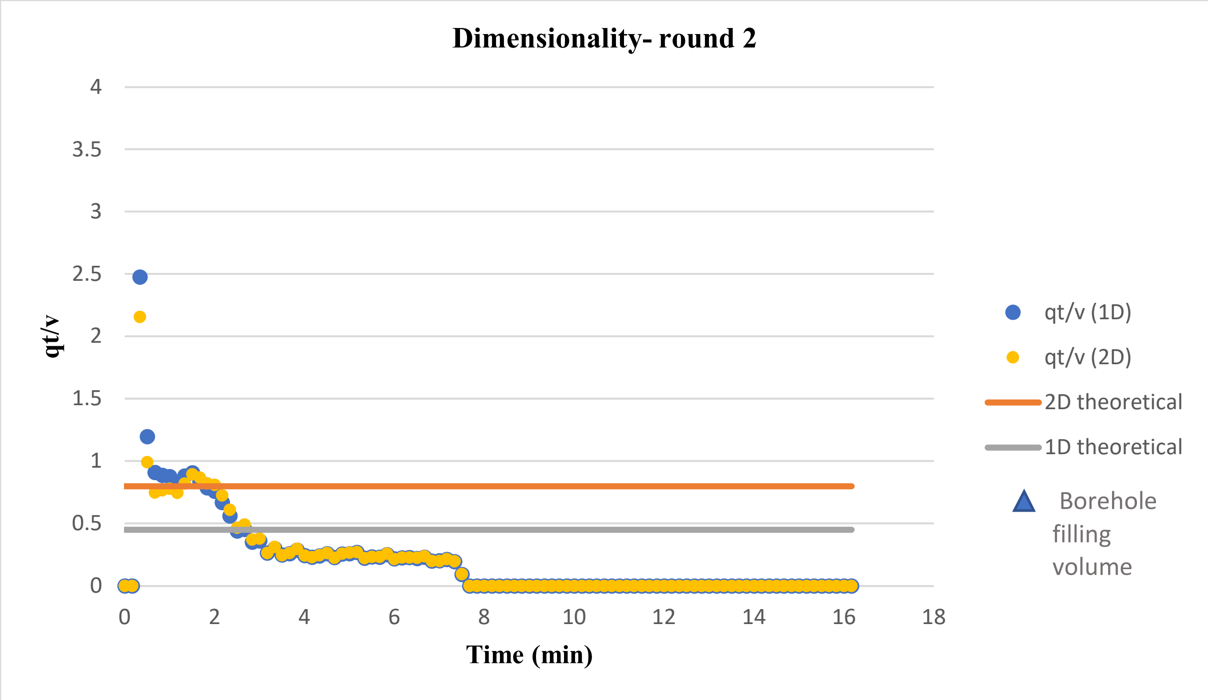


Figure 6.55: Dimensionality indicates that the flow in hole number 24 is likely 1D in the second round.

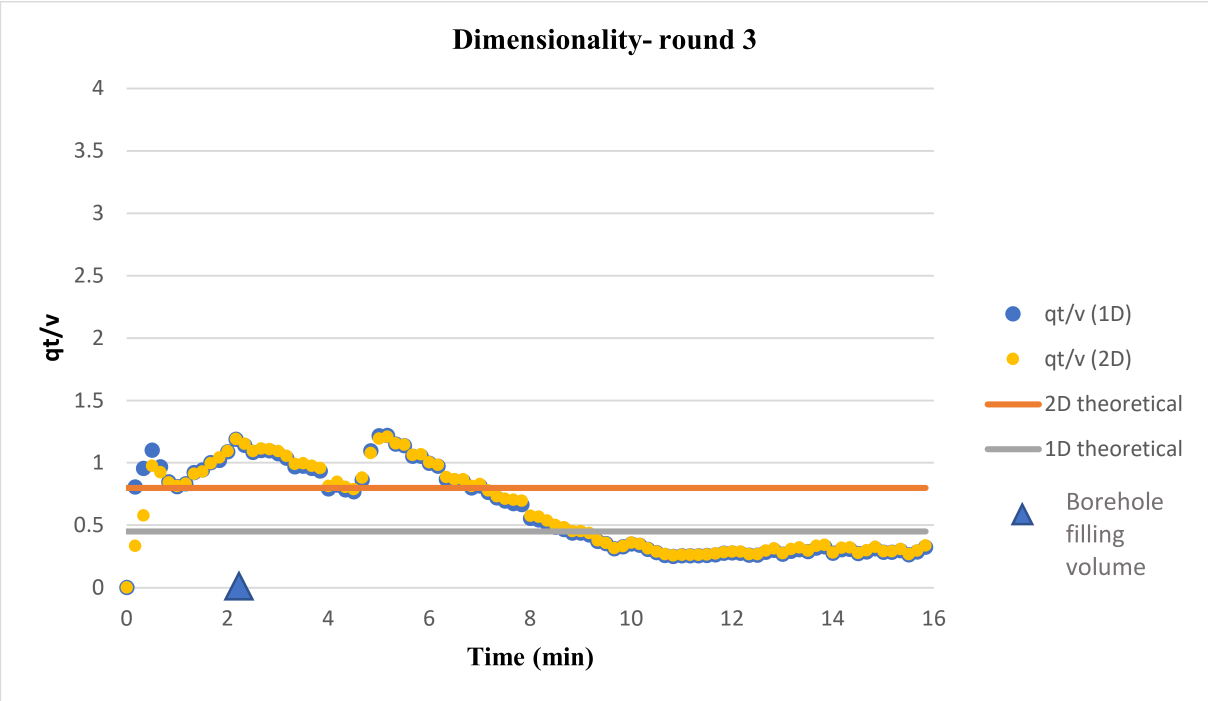


Figure 6.56: Dimensionality indicates that the flow in hole number 24 is likely 1D in the third round.

In Figures 6.57 and 6.58, the fourth-round logging data and dimensionality are presented, i.e., last round for borehole 24. The dimensionality is estimated to be 1D at the last round and RTGC can be applied to estimate the spread. In such unfavorable conditions, RTGC applies only when the flow dimensionality changes to 2D or 1D. But for the first round, RTGC is not applicable because of the 3D flow.

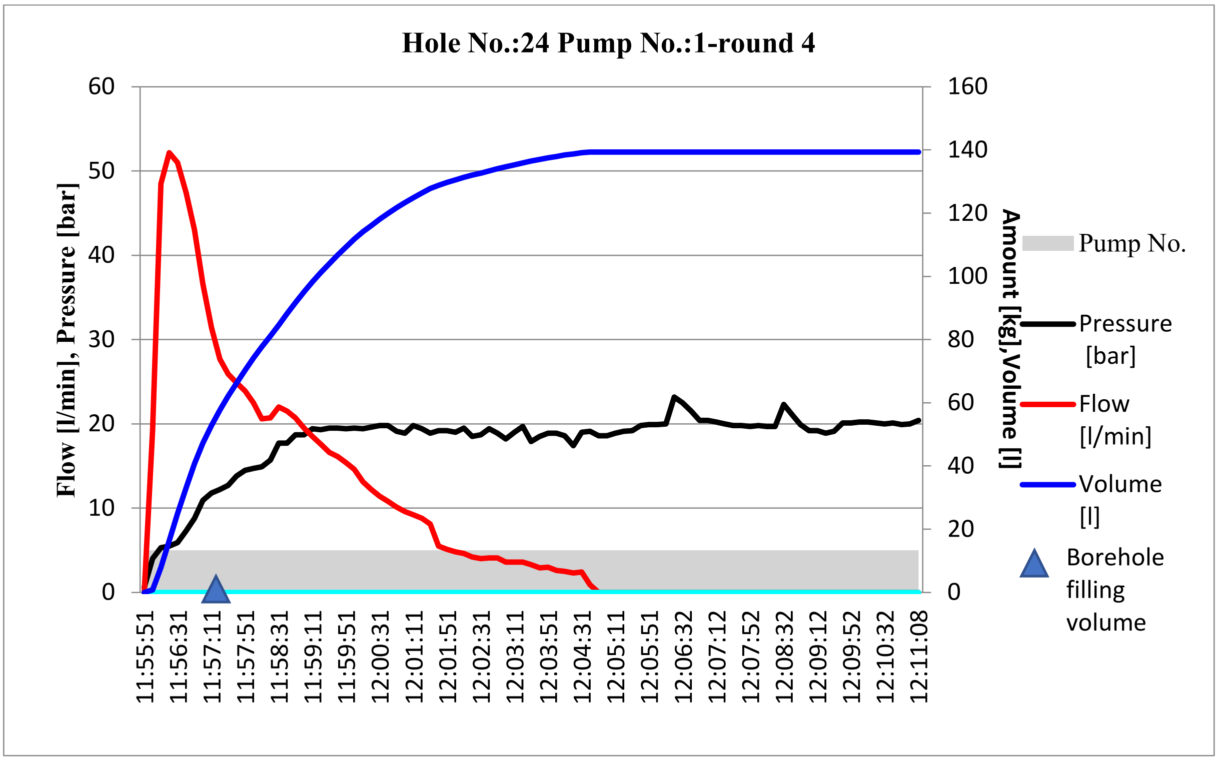


Figure 6.57: Flow and pressure distribution for hole number 24 in fan 294 as a function of time in the fourth round.

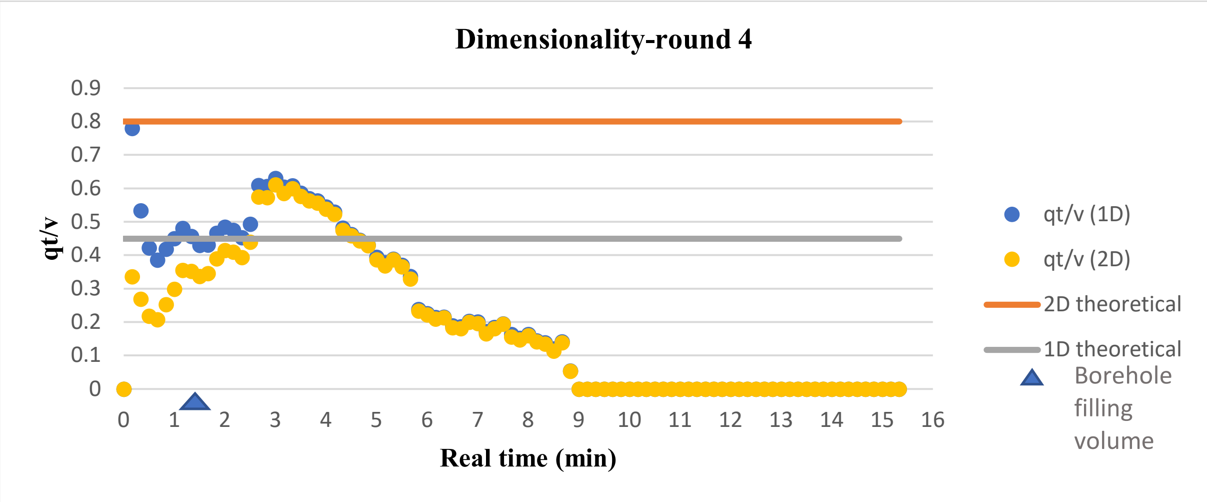


Figure 6.58: Dimensionality indicates that the flow in hole number 24 is likely 1D in the fourth round.

6.7.3. Fan 487-Londonviadukten

Another attempt for analysis was made for borehole 10 in fan 487, which is injected using specific solutions IK3, because of the large water depth at this fan, i.e., water pillar higher than 70 m. Figure 6.59 shows the logging data of borehole 10. Though the geological conditions were generally classified to be good, with rock class BKA in the design phase, the analysis of dimensionality show that the flow was 3D as shown in Figure 6.60, which means that RTGC cannot be applied to estimate the penetration length. More investigations are done in this section, and the geological mapping showed later that this borehole passes in the right wall of the fan, which is full of clay and crushed zones. Thus, this interprets the flow dimensionality in this borehole. This concludes that the classification of geological conditions is general and sensitive to certain parameters and can be changed even in few meters, the matter that can affect the design process due to the high uncertainty.

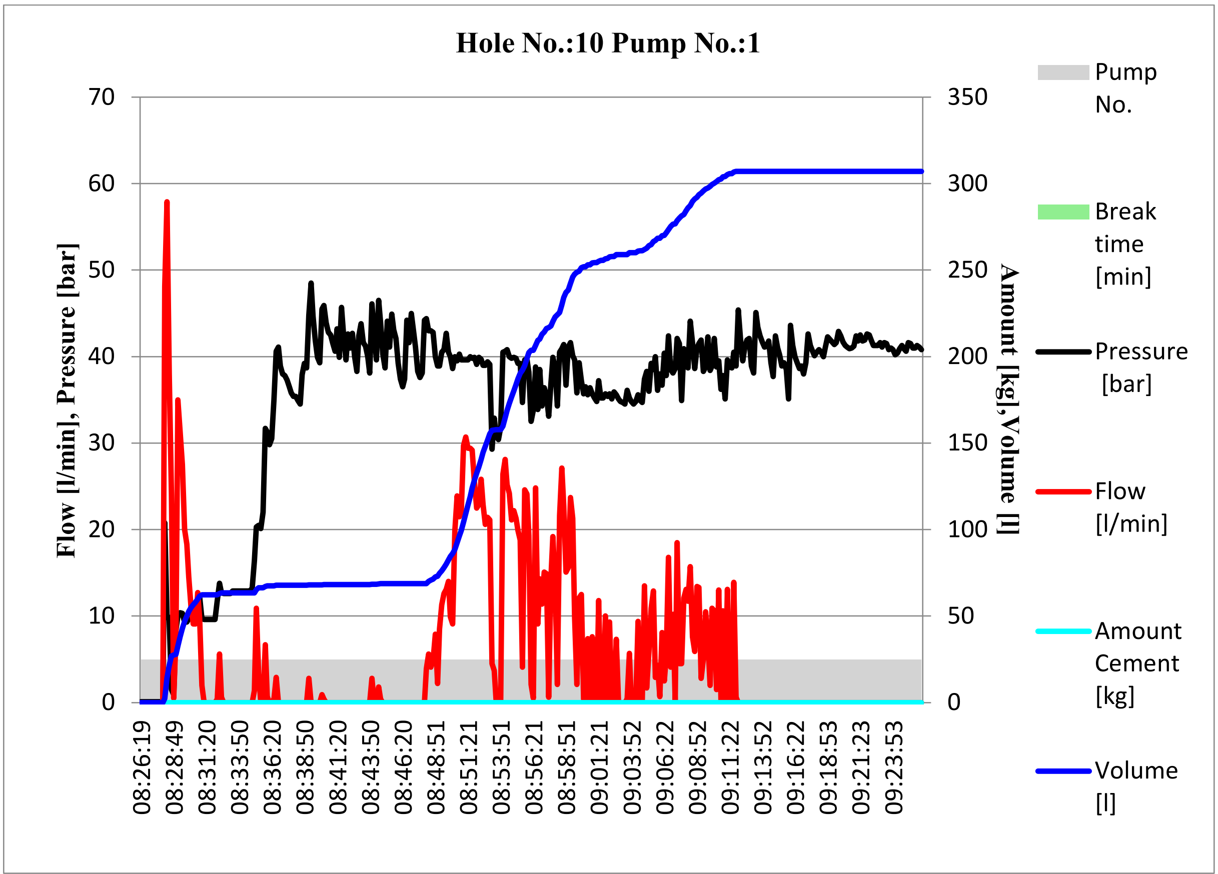


Figure 6.59. Flow and pressure distribution for hole number 10 in fan 487 as a function of time.

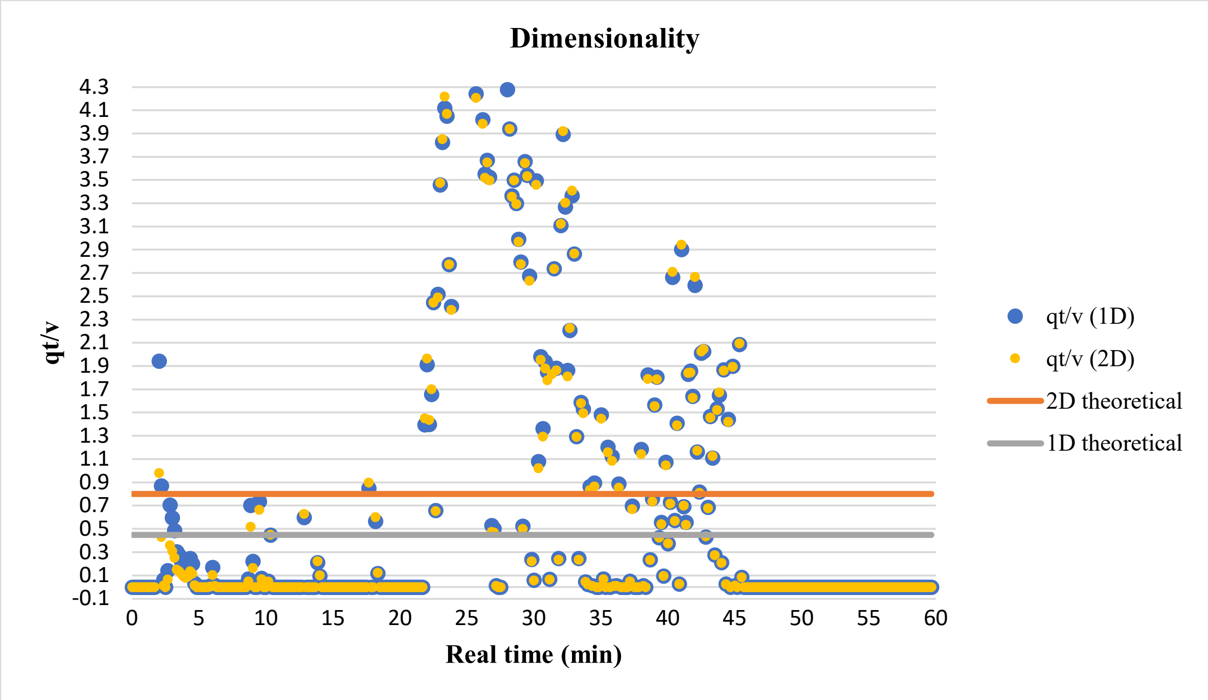


Figure 6.60. Dimensionality indicates that the flow in hole number 10 is 3D.

7. Conclusion

The assessment done in this project forms a documented experience for grouting evaluation in access tunnels for the extension of the Blue Metro Line in Sweden. The documented grouting design with the suggested recommendations can be used for other projects with similar conditions. However, since unfavorable conditions concept is wide and depends on many factors, there cannot be a specific stop criterion. Thus, the design and recommendations of stop criteria for weakness zones and unfavorable conditions, can be used for other projects only with similar conditions. In such projects, trial grouting can be a good way to optimize the grouting work and to establish relevant stop criteria. In addition, performing systematic investigations for conductivities is recommended for verifications and control. In this way, sealing efficiency and achieved conductivities after grouting can be better calculated and monitored.

7.1 Evaluation of grouting design and executed grouting works

Based on the evaluation of grouting data from the six access tunnels, following conclusions can be made:

1- Sealing efficiency values were estimated based on the authors' calculations of hydraulic conductivities for ungrouted zones. Therefore, the presented values can be either higher or lower. It was not possible to precisely calculate the sealing efficiency values since the designed effective hydraulic conductivities were not useful at all fans. The reason behind this is that the hydraulic conductivity values fall within a high range in each hydrogeological domain. Based on our assumptions for hydraulic conductivities for ungrouted rock:

- The achieved sealing efficiency for normal rock varies between (69-99) %. The estimated hydraulic conductivity after grouting was between ($9.7*10^{-10}$-$1.6*10^{-8)}$ m/s, considering the conductivity for ungrouted rock between ($2*10^{-8}$ - $3*10^{-7}$) m/s.
- The achieved sealing efficiency for weakness zones varies between (31-99 %. The estimated hydraulic conductivity after grouting was between ($1.1*10^{-10}$-$2.9*10^{-8}$) m/s, considering the conductivity for ungrouted rock between ($5*10^{-8}$ - $5*10^{-7}$) m/s. It shall be mentioned that the achieved sealing efficiency in weakness zones were different with a high variation depending on each zone's geological properties, regarding most unfavorable parameters i.e., crack filling, direction of dominant cracks, direction of fractures Vs. tunnel directions, and crack endurance.
- In Londonviadukten, the weakness zones had rock class D and needed 5 rounds of injection to achieve the required results of good sealing. While, in Sundstabacken, it was prognosed that rock class D will appear. However, rock class D was not found and thus these better geological conditions helped in getting higher sealing efficiency here than in Londonviadukten with a smaller number of injections rounds.
- The achieved sealing efficiency in surface rock varies between (55-99) %. The estimated hydraulic conductivity after grouting was between ($7.2*10^{-9}$-$4.5*10^{-8)}$ m/s, considering the conductivity for ungrouted rock between ($5*10^{-8}$ - $5*10^{-7}$) m/s.

2- For the access tunnels, the requirement for leakage according to the application to the land and environmental court were all met so far for the evaluated fans.

- It was shown that in Sundstabacken, the measured leakage after grouting was very low compared to the prognosed flow. Whereas in the other tunnels, the measured leakage after grouting was very close to the prognosed flow.
- In Londonviadukten, the accumulated leakage was 56.8 L/min which is lower than the prognosed leakage of 59.8 L/min at the last evaluated section. For normal rock the inleakage after grouting varied between 1-10 L/min-100 m. In weaknesszones the leakage after grouting varied between 20 -45 L/min-100 m.
- In Järla Östra, the accumulated leakage was 13.25 L/min which is slightly higher than the prognosed leakage of 12.8 L/min at the last evaluated section. For normal rock the inleakage after grouting was 2 L/min-100 m. For surface rock the leakage after grouting was 24 L/min-100 m.
- In Sundstabacken, the accumulated leakage was 3.2 L/min which is much lower than the prognosed leakage of 46.7 L/min at the last evaluated section. The measured leakage after grouting was less than 3 L/min -100 m.
- In Hammarby Fabriksväg, the accumulated leakage was 2.6 L/min which is lower than the prognosed leakage of 11.5 L/min at the last evaluated section. For normal rock the leakage after grouting varied between 1-3 l/min-100 m.
- In Värmdövägen, the accumulated leakage was 3.75 L/min which is lower than the prognosed leakage of 4.5 L/min. For normal rock the leakage after grouting varied between 2 -12 L/min-100 m.
- In Skönviksvägen, the accumulated leakage was 5.5 L/min which is slightly lower than the prognosed leakage of 5.8 L/min at the last evaluated section. For weaknesszone the leakage after grouting varied between 2 -4 L/min-100 m.

3- Based on the evaluations of grouting uptake and stop criteria, it was estimated that for typical injection zones (IK1 and IK2), the average grout uptake is around 2 L/m, excluding hole and hose filling volumes, with a standard deviation of 2. Whereas, in IK3 zones, it was not possible to come up with a reference value for the grout uptake per m, because IK3 zones were classified based on many factors and was divided to IK3 based on geometry and IK3 based on weakness zones.

- In IK1, 3-57% of boreholes showed zero grout take, which implies that those boreholes were already sealed or filled with filling materials. Also, we should note that the lowest percentages in the zero flow boreholes were usually found in surface rock and weakness zones. This is the reason behind having high leakage in these areas. The percentage of boreholes that were stopped by time stop criteria were at least 43% and at maximum 94% in all access tunnels. Whereas the percentage of boreholes that were stopped by volume stop criteria were in some tunnels 0%, but maximum 13% in the others.
- In IK2, 43% of boreholes showed zero grout take. Which implies that those boreholes were already sealed or filled with filling materials. The percentage of boreholes that were stopped

by time stop criteria were at least 55% and at maximum 57% in all access tunnels. Whereas the percentage of boreholes that were stopped by volume stop criteria were maximum 2%.

- In IK3 due to weakness zones, 0-64% of boreholes showed zero grout take. Which implies that those boreholes were already sealed or filled with filling materials. The percentage of boreholes that were stopped by time stop criteria were at least 0% and at maximum 99% in all access tunnels. Whereas the percentage of boreholes that were stopped by volume stop criteria were in some tunnels 0%, but maximum 13% in the others.
- In IK3 geometry which covers situations where connection between tunnels exist, low rock coverage, precut at the beginning of each tunnel, and other situations such as passing near existing structures (distance <15m), and injection at large depth (water pillar >70 m), 1-46 % of boreholes showed zero grout take. Which implies that those boreholes were already sealed or filled with filling materials. The percentage of boreholes that were stopped by time stop criteria were at least 42% and at maximum 89% in all access tunnels. Whereas the percentage of boreholes that were stopped by volume stop criteria were in some tunnels 0%, but maximum 12% in the others.

4- In this work, the RTGC was applied on good geological conditions in fan 403 in Londonviadukten. RTGC proved its applicability to calculate the penetration length and to predict the flow. This method can be used in performing trial grouting in different geological conditions to determine relevant stop criteria while reducing time and costs. For some fans in weakness zones the dimensionality changes from 3D to 1D during grouting after several rounds, which indicates that the fractures are sealed by grouting. However, in other fans in weakness zones, the dimensionality of the flow was 3D and RTGC could not be applied since it was developed for only 1D and 2D cases. Therefore, it would be good if future research can be made to develop the RTGC method to cover 3D cases.

7.2 Recommendation for stop criteria

1- In normal rock conditions, the grouting time and grouting volume stop criteria are recommended to stay as stated in the design stop criteria in IK1 and IK2 classes. However, in cases where thin dense fractures are present as in Värmdövägen, more investigations shall be made, and the grouting time shall be increased.

2- In weakness zones, stop criteria can differ from one section to another depending on the assessed parameters. In Londonviadukten, the weakness zones took maximum 5 rounds of grouting with a high percentage of boreholes that were stopped by volume. Therefore, the stop criteria are recommended to be adjusted so that the maximum volume stop criterion is 1000 L instead of 500 L. For grouting time, 15 min were sufficient to meet the design requirements, taking into consideration same complementary actions as stated in design. While other weakness zones such as in Sundstabacken, designed stop criteria were satisfying because of the good rock quality conditions compared to Londonviaadukten.

3- In surface rock conditions, such as in Järla Östra, the stop criteria are recommended to be adjusted since the real leakage measurements were so close to the prognosed flow, which might lead to exceeding the requirement for leakage according to the application to the land and environmental court. The recommendations are to increase the grouting time, for example to15 minutes instead of 12 minutes, with larger maximum stop volume criteria of 500L instead of 300L.

4- In zones that were injected with IK3 due to Geometry, i.e., situations where connections between tunnels exist, low rock coverage, precut at the beginning of each tunnel, and other situations such as passing near existing structures (distance <15m), and injection at large depth (water pillar >70 m), the designed stop criteria were generally satisfied for each case. However, for injection at large water pillar depth, it was noticed that when the water pillar is higher than 90m, the leakage measurements were increased significantly. Thus, it is recommended to increase the maximum volume as a stop criterion to be 500L at large depths of more than 90m instead of 300 L. Grouting time can be increased because of the high possibility of erosion happening which will affect the penetration length. Therefore, time is recommended to be increased to 20 minutes instead of 15.
5- Criteria for additional grouting shall be based on the assessment of achieved conductivity and leakage.

7.3 Recommendation for follow up of grouting works

1- Improving measurement process to get accurate leakage measurements. The way that helps controlling the leakage and analyzing the results in relation to the hydraulic conductivity for ungrouted zones. Accordingly, makes better estimation of the achieved sealing efficiency. This is based on the challenges that were faced in this work, to determine representative leakage measurement values after grouting.
2- Improving execution of water loss measurements to better estimate the conductivity of ungrouted rock with higher accuracy. This can help in better dividing the zones into hydrogeological domains with similar properties.
3- Verification of conductivity after grouting by water loss measurement shall be made.
4- Continuous evaluation of the prognosed leakage and measured leakage at site.

8. Recommendation for future works

1- Evaluation of observational method approach during grouting is recommended to make a better assessment of the geological conditions.
2- Analysis of water loss measurements to estimate the conductivities of ungrouted rock must be made.
3- Evaluation of grouting design for large geometries, e.g., station areas.
4- Analysis of jacking.
5- Comparing the results with other big projects, e.g, Förbifart Stockholm.
6- Developing RTGC method to study 3D flow cases, as it is a likely method to optimize the grout process and to inspect the designed stop criteria.
7- More studies and analysis to be done on the penetrability by using cement-based grout materials with different additives.
8- Conduct research to check the effect of reaction of chloride filling with the cement base grout.

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Documents retrieved from FUT database

It must be noted that many references were retrieved from FUT webforum and database to be able to perform this grout evaluation. A compilation of the documents used is listed below, however they are not cited in the text to avoid disruption of information flow.

Dimensionering av typinjektering för bygghandling. 2017-10-04, revision date 2017-12-20. Granskad av, utförare: Marie von Matérn Godkänd av, utförare: Thomas Andtfolk Granskad av, beställare: Lars Rosengren Fastställd av, beställare: Magnus Lundin. 1331-P11-24-00006

Nacka och Söderort-2330 Bygghandling-Hydrogeologisk domänindelning för berggrundens vattenförande förmåga. 2018-04-27. Granskad av, utförare: Magnus Zetterlund, Norconsult AB. Godkänd av, utförare: Nils Outters, Sweco/TYPSA. 2330-B21-24-00001

Nacka och söderort-7713 Arbetstunnel Londonviadukten-Projekteringsförutsättningar – Bergkonstruktioner. 2019-06-12. Granskad av, utförare: Robert Swindell, Sweco/TYPSA. Godkänd av, utförare: Nils Outters, Sweco/TYPSA. Granskad av, Beställare: Lars-Olof Dahlström. Godkänd/Fastställd av, Beställare: Magnus Lundin. 7713-B21-24-00002

Nacka och söderort-7713 Arbetstunnel Londonviadukten-Ingenjörsgeologisk prognos. 2019-06-12. Granskad av, utförare: Robert Swindell, Sweco/TYPSA. Godkänd av, utförare: Nils Outters, Sweco/TYPSA. Granskad av, Beställare: Lars-Olof Dahlström . Godkänd/Fastställd av, Beställare: Magnus Lundin. 7713-B21-24-00003.

Nacka och Söderort-7713 Arbetstunnel Londonviadukten-Projekteringsrapport Bergkonstruktioner. 2019-06-12. Granskad av, utförare: Carl-Olof Söder, Sweco/TYPSA. Godkänd av, utförare: Nils Outters, Sweco/TYPSA. Granskad av, Beställare: Lars-Olof Dahlström. Godkänd/Fastställd av, Beställare: Magnus Lundin. 7713-B21-24-00004.

Nacka och söderort-87132 Arbetstunnel Järla Östra-Projekteringsförutsättningar -Bergkonstruktioner. 2019-08-06. Granskad av, utförare: Robert Swindell, Sweco/TYPSA. Godkänd av, utförare: Nils Outters, Sweco/TYPSA. Granskad av, beställare: Lars-Olof Dahlströ. Handläggare, beställare: Theresa Millqvist, FUT. Godkänd/fastställd av beställare: Magnus Lundin. 87132-B21-24-00001.

Nacka och söderort-87132 Arbetstunnel Järla Östra-Ingenjörsgeologisk prognos. 2017-12-18. Granskad av, utförare: Robert Swindell, Sweco/TYPSA. Godkänd av, utförare: Nils Outters, Sweco/TYPSA. Granskad av, beställare: Lars-Olof Dahlström. Handläggare, beställare: Theresa Millqvist, FUT. Godkänd/fastställd av beställare: Magnus Lundin. 87132-B21-24-00003.

Nacka och Söderort-87132 Arbetstunnel Järla Östra-Projekteringsrapport Bergkonstruktioner. 2019-08-06. Granskad av, utförare: Lars Hässler, Sweco/TYPSA. Godkänd av, utförare: Nils Outters, Sweco/TYPSA. Granskad av, beställare: Lars-Olof Dahlström. Handläggare, beställare: Theresa Millqvist, FUT. Godkänd/fastställd av beställare: Magnus Lundin, FUT2-15. 87132-B21-24-00004

Nacka och söderort-9714 Arbetstunnel Sundstabacken-Projekteringsförutsättningar – Bergkonstruktioner. 2019-03-08. Granskad av, utförare: Robert Swindell, Sweco/TYPSA. Godkänd av, utförare: Nils Outters, Sweco/TYPSA. Granskad av, beställare: Lars-Olof Dahlström. Handläggare, beställare: Dan Palmgren, FUT. Godkänd/fastställd av, beställare: Magnus Lundin, FUT2-17. 9714-B21-24-00001.

Nacka och Söderort-9714 Arbetstunnel Sundstabacken-Projekteringsrapport Bergkonstruktioner. 2019-03-29. Granskad av, utförare: Lars Hässler, Sweco/TYPSA . Godkänd av, utförare: Nils Outters, Sweco/TYPSA. Granskad av, beställare: Lars-Olof Dahlström. Handläggare, beställare: Theresa Millqvist, FUT .Godkänd/fastställd av, beställare: Magnus Lundin, FUT2-17. 9714-B21-24-00005.

Nacka och söderort-9714 Arbetstunnel Sundstabacken-Ingenjörsgeologisk prognos. 2019-03-28. Granskad av, utförare: Robert Swindell, Sweco/TYPSA. Godkänd av, utförare: Nils Outters, Sweco/TYPSA. Granskad av, beställare: Lars-Olof Dahlström. Handläggare, beställare: Theresa Millqvist, FUT. Godkänd/fastställd av, beställare: Magnus Lundin, FUT2-17. 9714-B21-24-00006.

Nacka och söderort-7714 Arbetstunnel Hammarbyet Fabriksväg inkl. FBA-Projekteringsförutsättningar – Bergkonstruktioner. 2020-07-10. Granskad av, utförare: Robert Swindell, Sweco/TYPSA. Godkänd av, utförare: Nils Outters, Sweco/TYPSA. Granskad av, beställare: Lars-Olof Dahlström. Handläggare, beställare: Theresa Millqvist, FUT . Godkänd/Fastställd av, beställare: Magnus Lundin, FUT2-13. 7714-B21-24-00002.

Nacka och söderort-7714 Arbetstunnel Hammarby Fabriksväg inkl. FBA-Ingenjörsgeologisk prognos. 2020-07-10. Granskad av, utförare: Robert Swindell, Sweco/TYPSA. Godkänd av, utförare: Nils Outters, Sweco/TYPSA. Kontrollerad av, utförare: Lars-Olof Dahlström .Handläggare, beställare: Theresa Millqvist, FUT. Godkänd/Fastställd av, beställare: Magnus Lundin, FUT2-13. 7714-B21-24-00003.

Nacka och Söderort-7714 Arbetstunnel Hammarby Fabriksväg inkl. FBA-Projekteringsrapport Bergkonstruktioner. 2020-07-10. Granskad av, utförare: Carl-Olof Söder, Sweco/TYPSA. Godkänd av, utförare: Nils Outters, Sweco/TYPSA. Kontrollerad av, utförare: Lars-Olof Dahlström. Handläggare, beställare: Theresa Millqvist, FUT. Godkänd/Fastställd av, beställare: Magnus Lundin, FUT2-13. 7714-B21-24-00004.

Nacka och söderort-87131 Arbetstunnel Värmdövägen-Projekteringsförutsättningar – Bergkonstruktioner. 2018-03-29. Granskad av, utförare: Robert Swindell, Sweco. Godkänd av, utförare: Nils Outters, Sweco. Granskad av, beställare: Lars-Olof Dahlström. Handläggare, beställare: Theresa Millqvist, FUT. Godkänd/fastställd av, beställare: Andreas Burghauser, FUT2-14. 87131-B21-24-00001.

Nacka och söderort-87131 Arbetstunnel Värmdövägen-Ingenjörsgeologisk prognos. 2018-10-23. Granskad av, utförare: Robert Swindell, Sweco /TYPSA . Godkänd av, utförare: Nils Outters, Sweco/TYPSA. Granskad av, beställare: Lars-Olof Dahlström. Handläggare, beställare: Theresa Millqvist, FUT. Godkänd/fastställd av, beställare: Andreas Burghauser, FUT2-14. 87131-B21-24-00003.

Nacka och Söderort-87131 Arbetstunnel Värmdövägen-Projekteringsrapport Bergkonstruktioner. 2019-08-06. Granskad av, utförare: Carl-Olof Söder, Sweco/TYPSA . Godkänd av, utförare: Nils Outters, Sweco/TYPSA. Granskad av, beställare: Lars-Olof Dahlström. Handläggare, beställare: Theresa Millqvist, FUT. Godkänd/fastställd av, beställare: Andreas Burghauser, FUT2-14. 87131-B21-24-00004.

Nacka and the south-87133 Arbetstunnel Skönviksvägen-Projekteringsförutsättningar – Bergkonstruktioner. 2018-07-10. Granskad av, utförare: Robert Swindell, Sweco . Godkänd av, utförare: Nils Outters, Sweco. Granskad av, beställare: Lars-Olof Dahlström. Handläggare, beställare: Theresa Millqvist, FUT. Godkänd/Fastställd av, beställare: Magnus Lundin, FUT2-16. 87133-B21-24-00001.

Nacka och söderort-87133 Arbetstunnel Skönviksvägen-Ingenjörsgeologisk prognos. 2019-06-12. Granskad av, utförare: Robert Swindell, Sweco/TYPSA . Godkänd av, utförare: Nils Outters, Sweco/TYPSA . Granskad av, beställare: Lars-Olof Dahlström. Handläggare, beställare: Theresa Millqvist, FUT. Godkänd/Fastställd av, beställare: Magnus Lundin, FUT2-16. 87133-B21-24-00003.

Nacka och Söderort-87133 Arbetstunnel Skönviksvägen-Projekteringsrapport Bergkonstruktioner. 2018-07-11. Granskad av, utförare: Carl-Olof Söder, Sweco . Godkänd av, utförare: Nils Outters, Sweco/TYPSA. Granskad av, beställare: Lars-Olof Dahlström. Handläggare, beställare: Theresa Millqvist, FUT. Godkänd/Fastställd av, beställare: Magnus Lundin, FUT2-16. 87133-B21-24-00004.

A. Appendix

A.1 Typical grouting design fan geometry

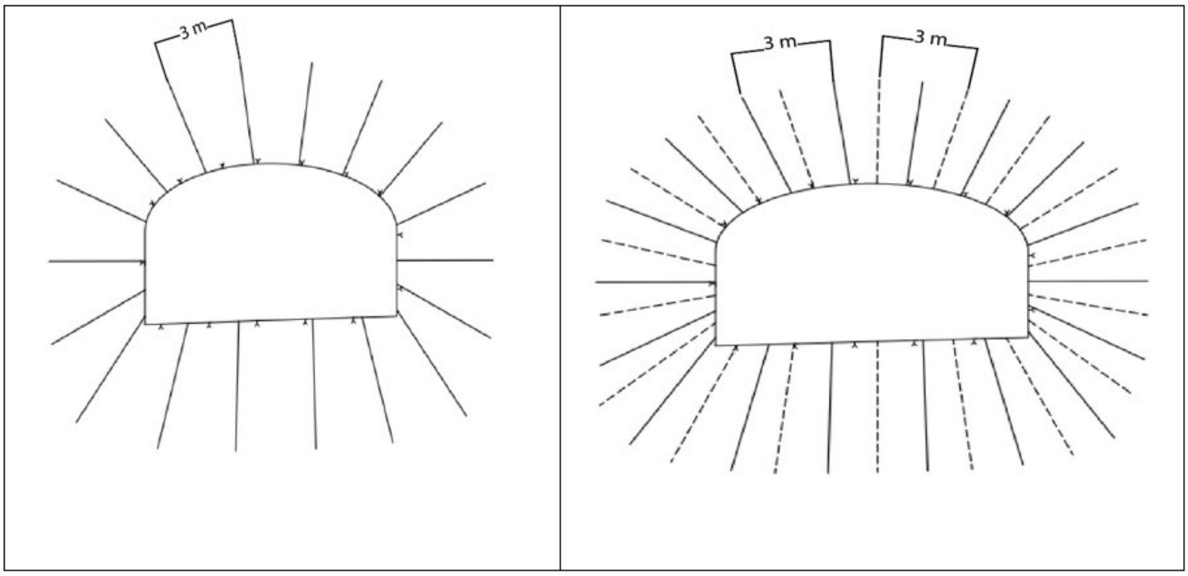

Figure A.1. Tip distance in a typical grouting fan.

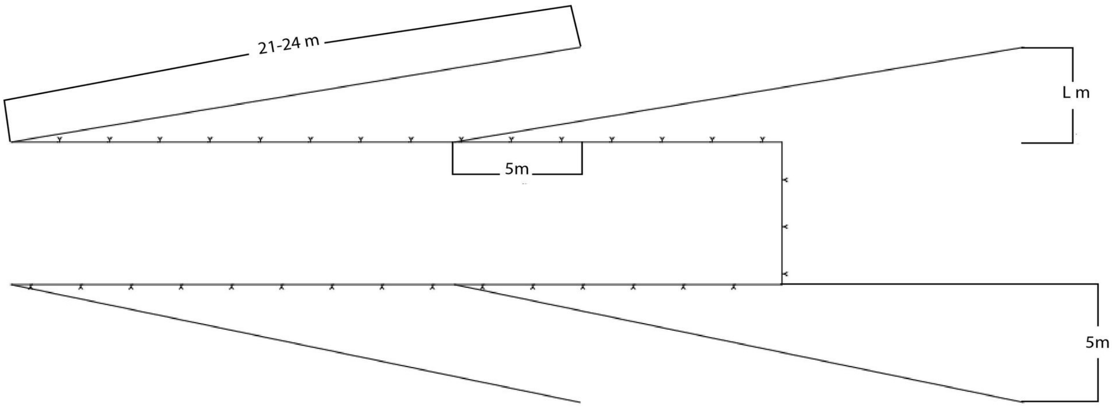

Figure A.2. Boreole average length (21-24m), overlap 5m, and stick of 5m in a typical grouting fan.

A.2 IK3 design fan geometries and stop criteria

- Londonviadukten (weakness zones)

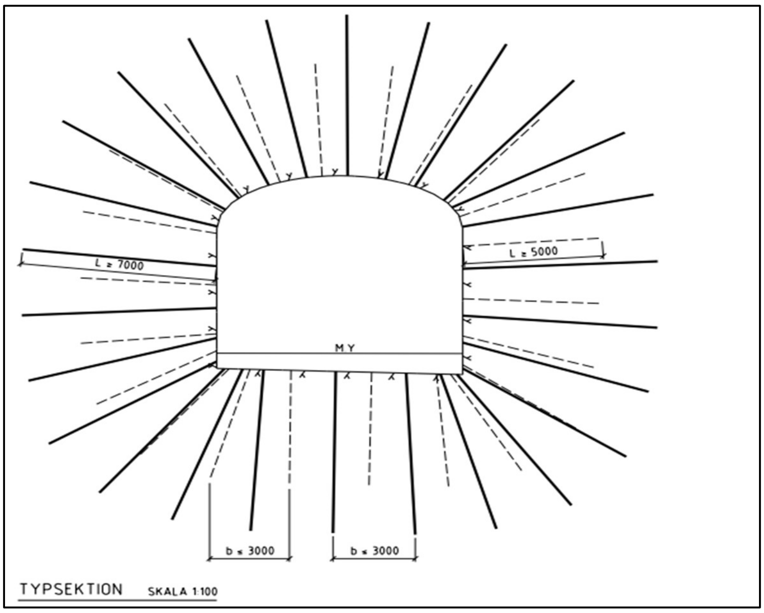

Figure A.3.Specific grouting fan in weakness zones.

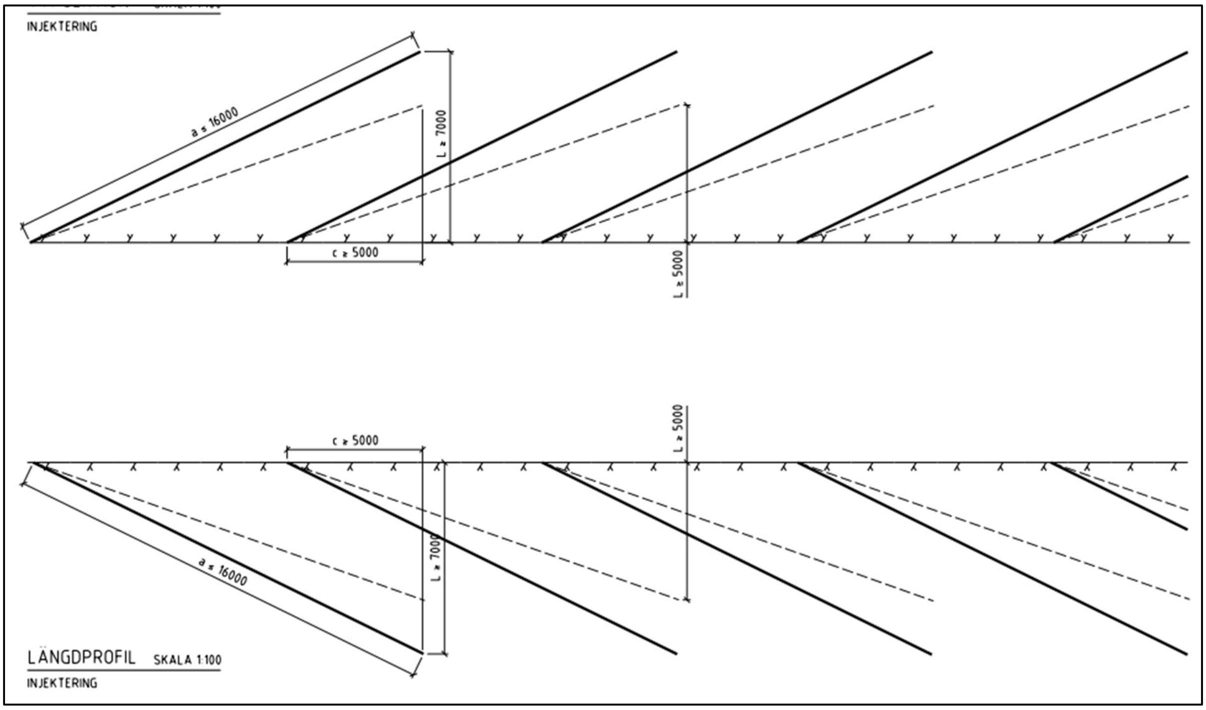

Figure A.4. Borehole's length, overlap, and stick in specific grouting fan in weakness zones.

Table A.1. Grouting pressure required based on design documents (1331).

Grouting Pressure [MPa]	Groundwater pressure / Rock cover [m]
1.0	10 m ≤ Groundwater pressure < 15 m
1.5	15 m ≤ Groundwater pressure< 20 m
2.0	20 m ≤ Groundwater pressure≤ 70 m

Table A.2. Complementary actions.

Situation	Action
The stop conditions for injected volume are reached before the stop conditions for injected time for mixture 2	Grouting must be carried out with utility mixture wct=0,5 immediately after end of grout in the current hole and let until grouting pressure is achieved.
Volume stop conditions are reached before the time stop condition for mixture 1	Grouting should be done immediately and continue with mixture 2. Stop conditions form mixture 2 are: 10 min injected time, 250 L injected volume in a 16 m borehole.

Table A.3.Stop criteria round 1

Grouting Pressure [MPa]	Instructions for stop conditions	Time [min]	Injected volume [Liter]
1.0	Grouting into the current borehole should be interrupted when the conditions for injected time or injected volume are reached	15	500 liters per 16 m borehole
1.5		15	500 liters per 16 m borehole
2.0		15	500 liters per 16 m borehole

Table A.4. Stop criteria round 2.

Grouting Pressure [MPa]	Instructions for stop conditions	Time [min]	Injected volume [Liter]
1.0	Grouting into the current borehole should be interrupted when the conditions for injected time or injected volume are reached	15	500 liters per 16 m borehole
1.5		15	500 liters per 16 m borehole
2.0		15	500 liters per 16 m borehole

- Londonviadukten (large depths)

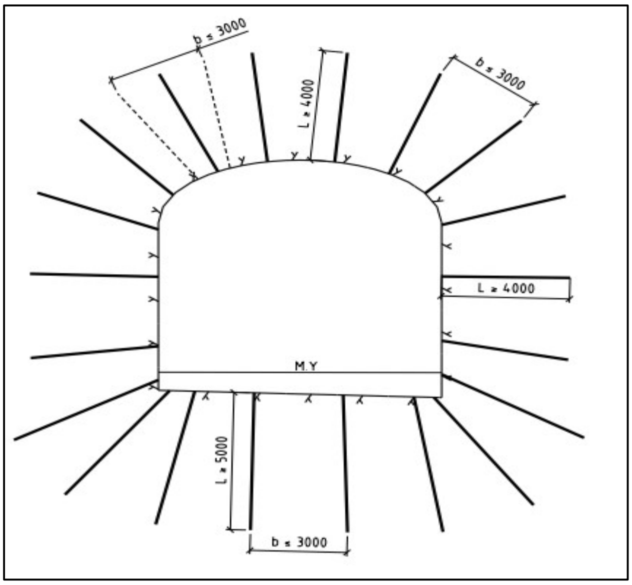

Figure A.5.Specific grouting fan in large depth zone.

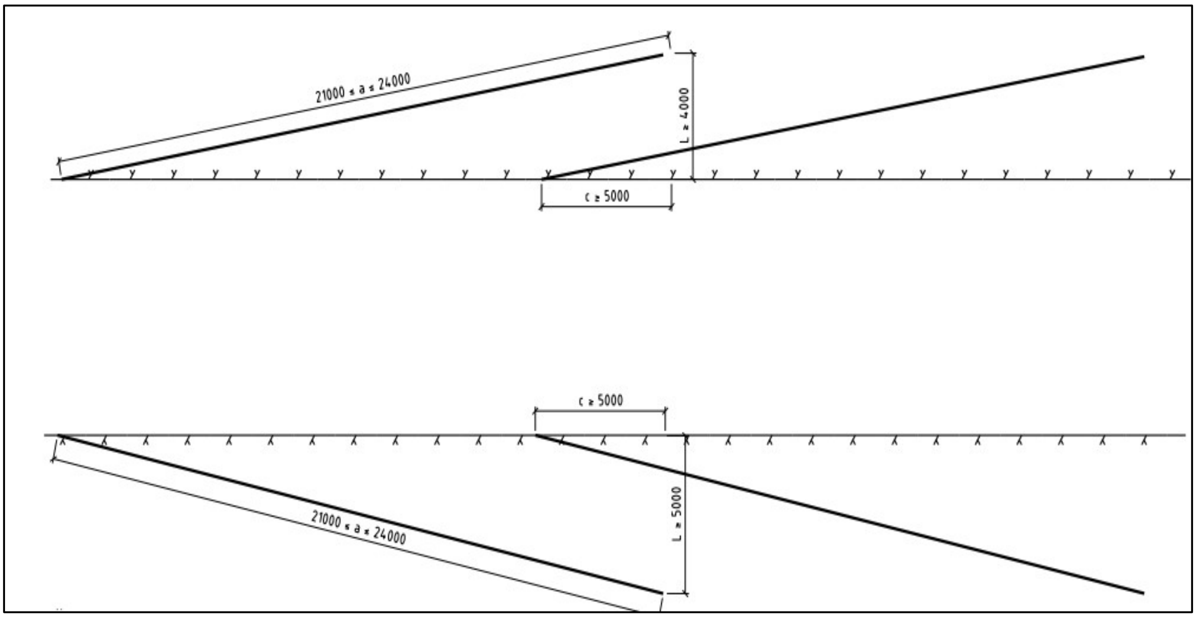

Figure A.6.Borehole's length, overlap, and stick in specific grouting fan in large depth zones.

Table A.5.Stop criteria

Grouting Pressure [MPa]	Instructions for stop conditions	Time [min]	Injected volume [Liter]
3.5	Grouting into the current borehole should be interrupted when the conditions for injected time or injected volume are reached	15	300 liters

Table A.6.Complementary actions

Unwanted event	Action
Prescribed grouting pressure is not achieved.	Grouting must be carried out with utility mixture wct=0,5 immediately after end of grout in the current hole.
The stop conditions for injected volume are reached before the stop conditions for injected time.	See technical description facility.

- Järla Östra (weakness zones, Birkavägen)

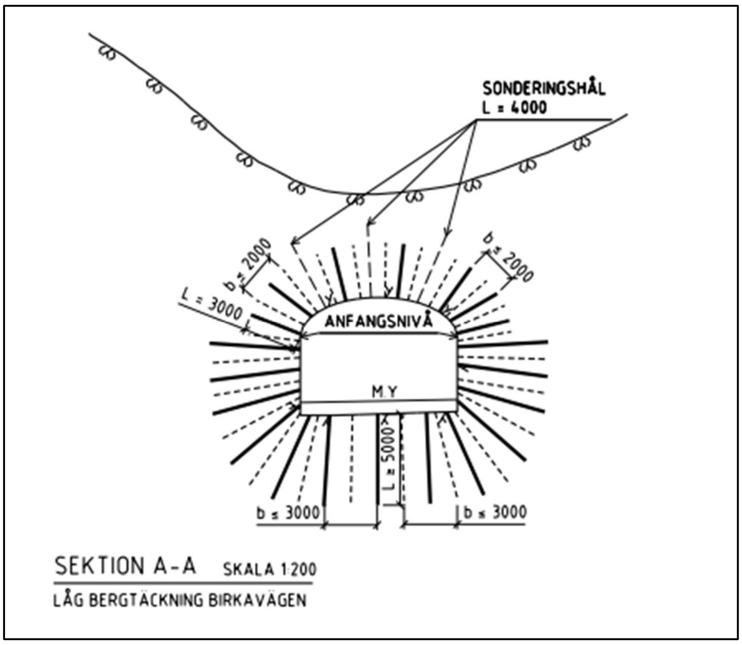

Figure A.7. Specific grouting fan in weakness zones.

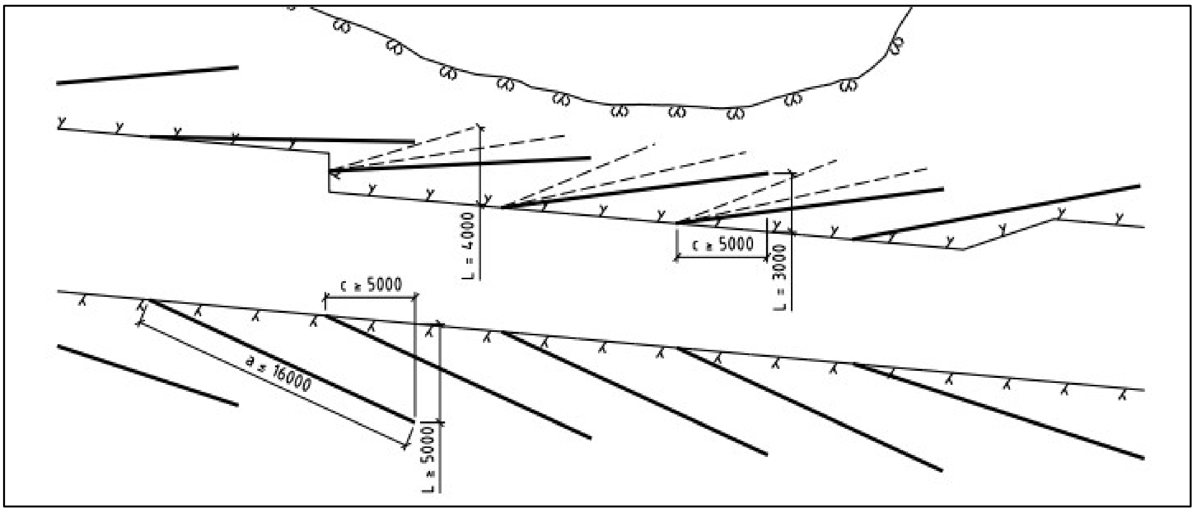

Figure A.8. Borehole's length, overlap, and stick in specific grouting fan in weakness zones.

Table A.7.Stop criteria.

Grouting Pressure [MPa]	Instructions for stop conditions	Time [min]	Injected volume [Liter]
0.5	Grouting into the current borehole should be interrupted when the conditions for injected time or injected volume are reached	15	250 liters per 16 m borehole
1.0		15	250 liters per 16 m borehole

Table A.8.Complementary actions.

Unwanted event	Action
Prescribed grouting pressure is not achieved.	Grouting must be carried out with utility mixture wct=0,5 immediately after end of grout in the current hole.
The stop conditions for injected volume are reached before the stop conditions for injected time.	See technical description facility.

- **Sundstabacken (weakness zone)**

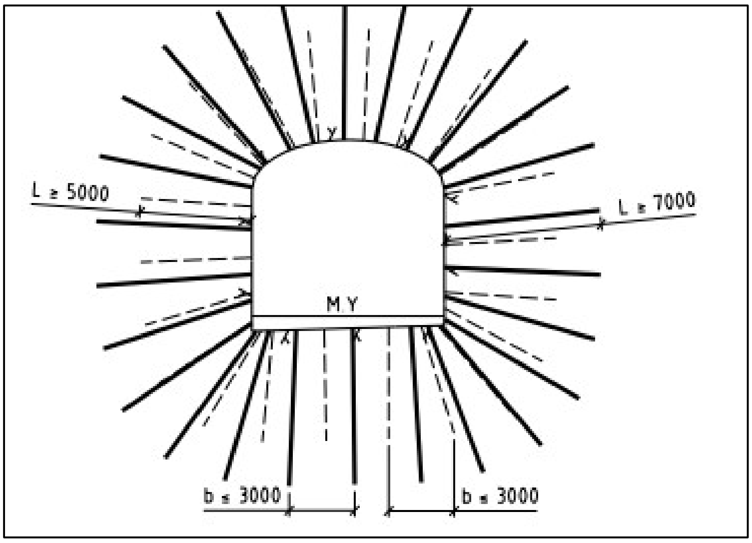

Figure A.9. Specific grouting fan in weakness zones.

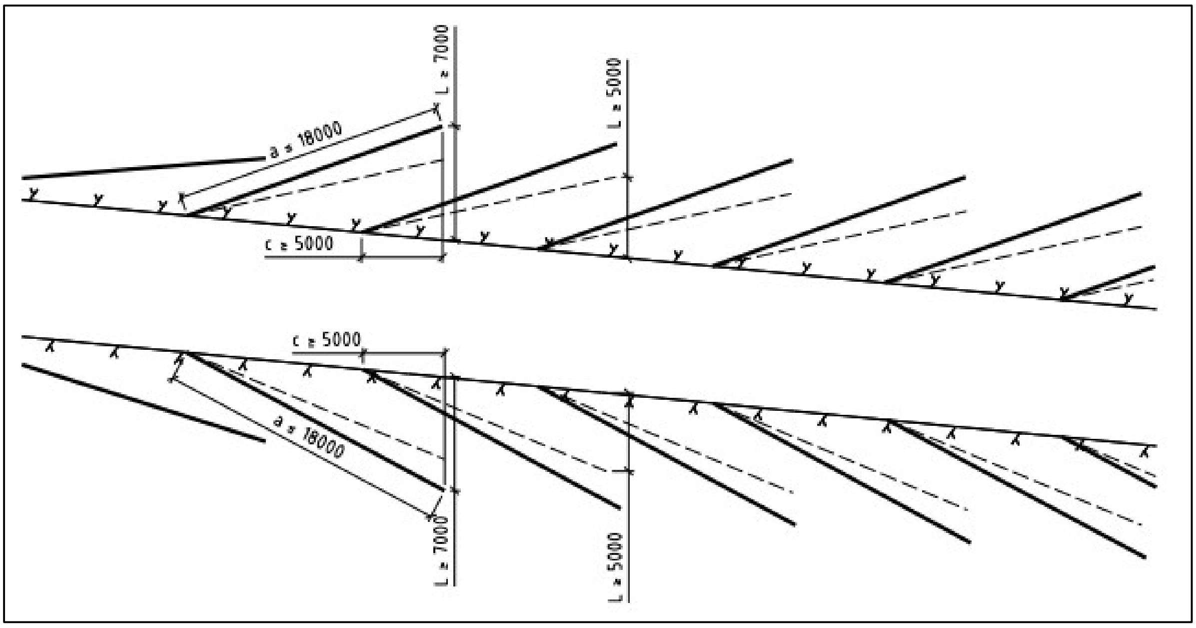

Figure A.10. Borehole's length, overlap, and stick in specific grouting fan in weakness zones.

Table A.9. Grouting pressure required based on design documents (1331).

Grouting Pressure [MPa]	Groundwater pressure / Rock cover [m]
1.0	10 m ≤ Groundwater pressure < 15 m
1.5	15 m ≤ Groundwater pressure< 20 m
2.0	20 m ≤ Groundwater pressure≤ 70 m

Table A.10.Stop criteria round 1.

Grouting Pressure [MPa]	Instructions for stop conditions	Time [min]	Injected volume [Liter]
1.0	Grouting into the current borehole should be interrupted when the conditions for injected time or injected volume are reached.	15	560 liters per 18 m borehole
1.5		15	560 liters per 18 m borehole
2.0		15	560 liters per 18 m borehole

Table A.11.Stop criteria round 2.

Grouting Pressure [MPa]	Instructions for stop conditions	Time [min]	Injected volume [Liter]
1.0	Grouting into the current borehole should be interrupted when the conditions for injected time or injected volume are reached.	15	560 liters per 18 m borehole
1.5		15	560 liters per 18 m borehole
2.0		15	560 liters per 18 m borehole

Table A.12.Complemenatry actions.

Situation	Action
The stop conditions for injected volume are reached before the stop conditions for injected time for mixture 2.	Grouting must be carried out with utility mixture wct=0,5 immediately after end of grout in the current hole and let until grouting pressure is achieved.
Volume stop conditions are reached before the time stop condition for mixture 1.	Grouting should be done immediately and continue with mixture 2. Stop conditions form mixture 2 are: 10 min injected time, 250 L injected volume in an 18 m borehole.

- Sundstabacken (passage near existing structure)

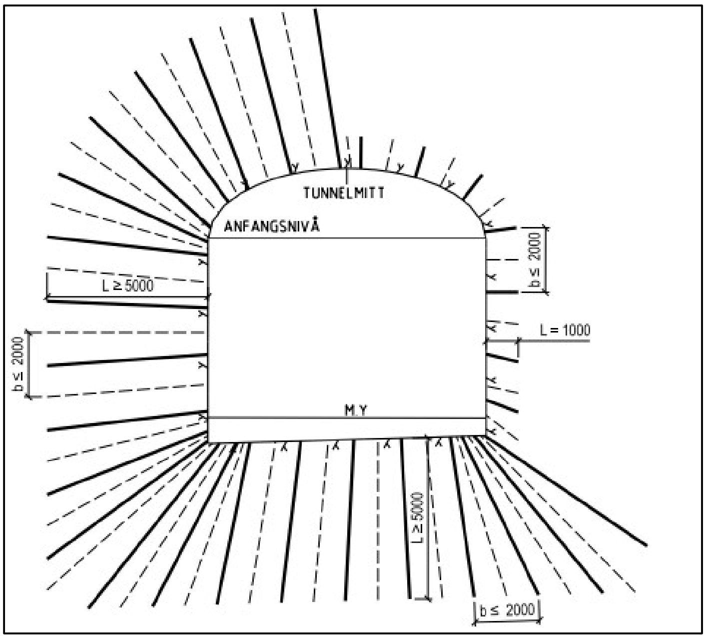

Figure A.11. Specific grouting fan in passage near existing structure zone.

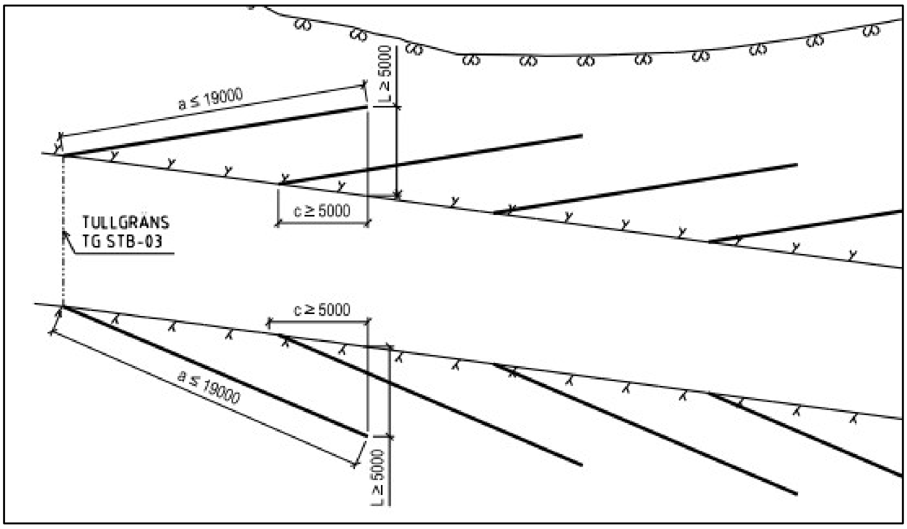

Figure A.12. Borehole’s length, overlap, and stick in specific grouting fan in passage near existing structure zone.

Table A.13.Stop criteria.

Grouting Pressure [MPa]	Instructions for stop conditions	Time [min]	Injected volume [Liter]
0.5	Grouting into the current borehole should be interrupted when the conditions for injected time or injected volume are reached.	15	175
1.0		15	250

Table A.14.Complemantary actions.

Unwanted event	Action
Prescribed grouting pressure is not achieved.	Grouting must be carried out with utility mixture wct=0,5 immediately after end of grout in the current hole.
The stop conditions for injected volume are reached before the stop conditions for injected time.	See technical description facility.

- Hammarby Fabriksväg

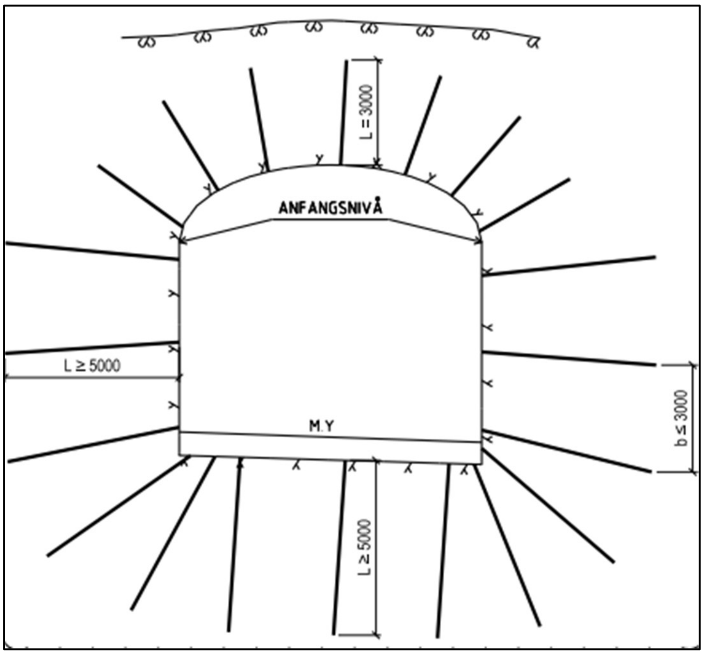

Figure A.13. Specific grouting fan.

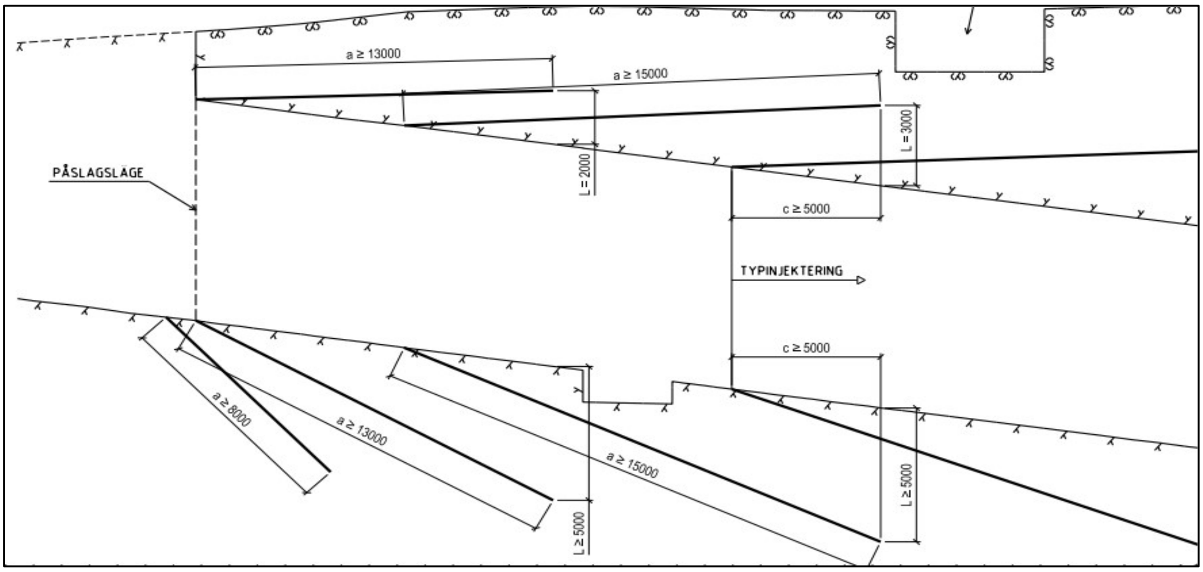

Figure A.14. Borehole's length, overlap, and stick in specific grouting fan.

Table A.15.Stop criteria.

Grouting Pressure [MPa]	Instructions for stop conditions	Time [min]	Injected volume [Liter]
0.5	Grouting into the current borehole should be interrupted when the conditions for injected time or injected volume are reached.	15	225
1.0		15	225

Table A.16.Complemantary actions.

Unwanted event	Action
Prescribed grouting pressure is not achieved.	Grouting must be carried out with utility mixture wct=0,5 immediately after end of grout in the current hole.
The stop conditions for injected volume are reached before the stop conditions for injected time.	See technical description facility.

- Värmdövägen (connection to the main tunnel)

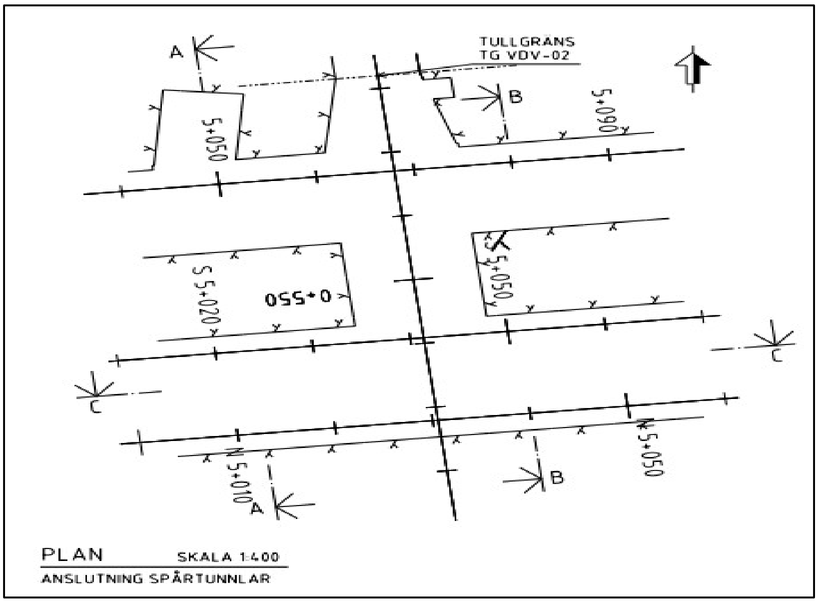

Figure A.15.Plan view for connection to main tunnel.

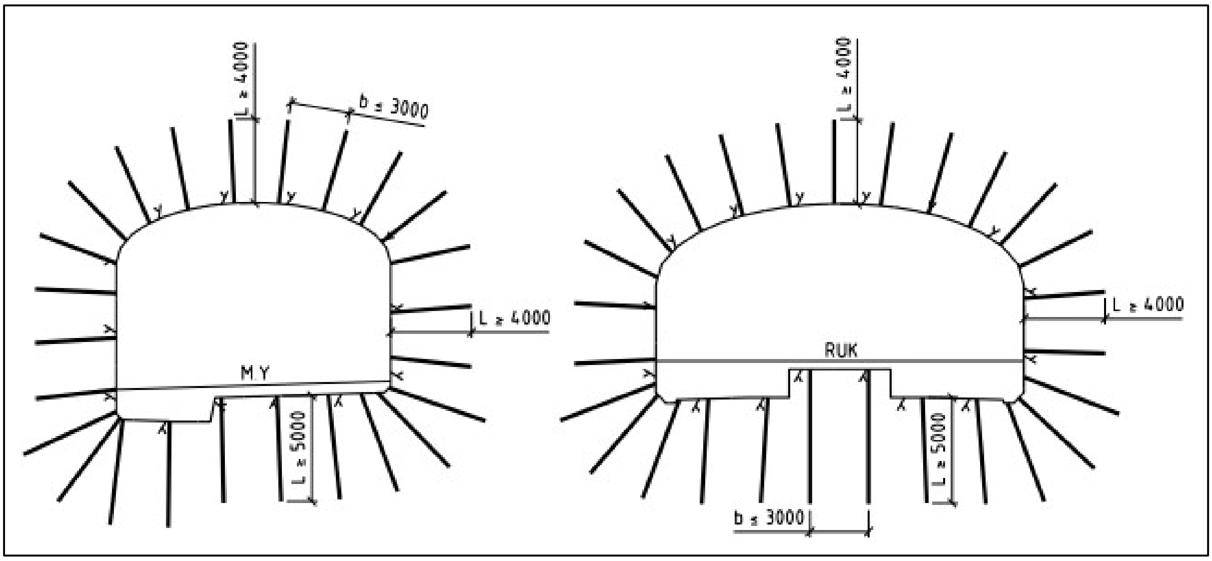

Figure A.16. Specific grouting fan for the connection to the main tunnel.

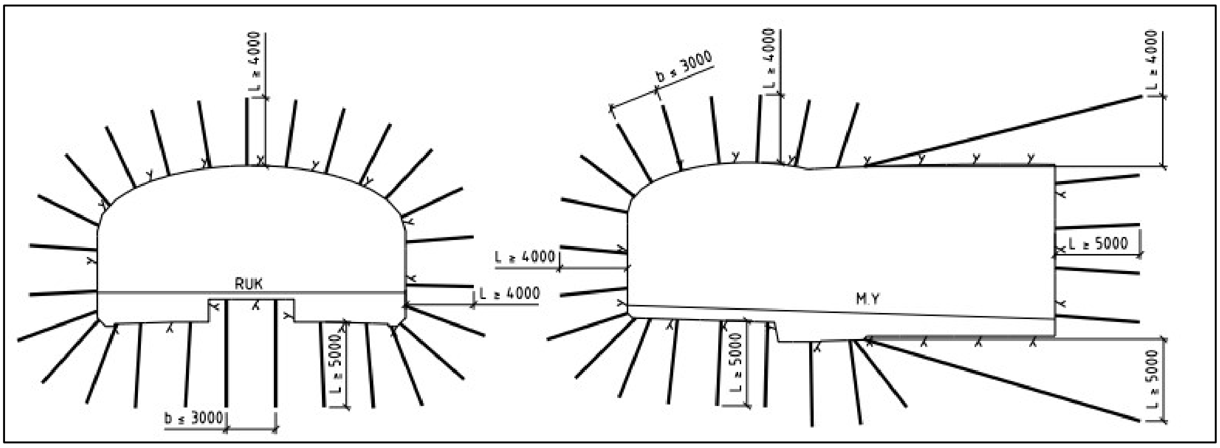

Figure A.17.Borehole's length, overlap, and stick in specific grouting fan in case of connection to main tunnel.

Table A.17.Grouting pressure required based on design documents (1331).

Grouting Pressure* [MPa]	Groundwater pressure / Rock cover [m]
1.0	10 m ≤ Groundwater pressure < 15 m
1.5	15 m ≤ Groundwater pressure< 20 m
2.0	20 m ≤ Groundwater pressure≤ 70 m

Table A.18.Stop criteria.

Grouting Pressure [MPa]	Instructions for stop conditions	Time [min]	Injected volume [Liter]
1.0	Grouting into the current borehole should be interrupted when the conditions for injected time or injected volume are reached.	12	350
1.5		10	300
2.0		10	300

Table A.19.Complemantary actions.

Event	Action
Prescribed grouting pressure is not achieved.	Grouting must be carried out with utility mixture wct=0,5 immediately after end of grout in the current hole.
The stop conditions for injected volume are reached before the stop conditions for injected time.	See technical description facility.

- Skönviksvägen (påslag)

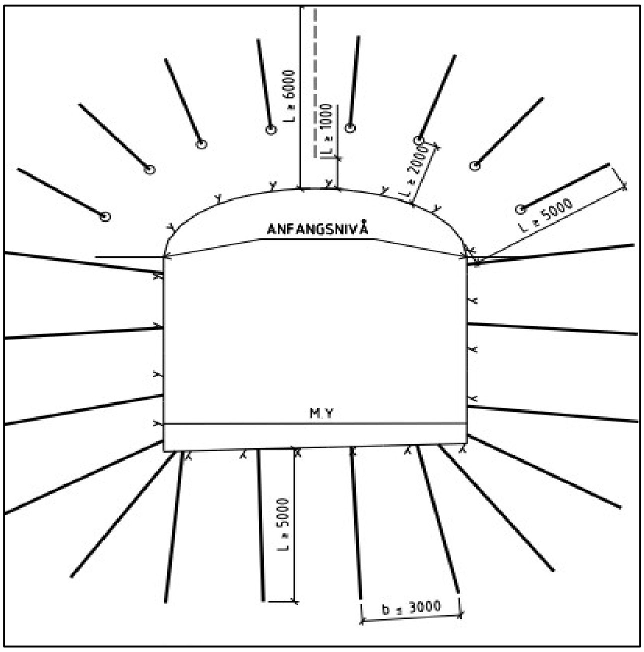

Figure A.18. Specific grouting fan at the pre-cut.

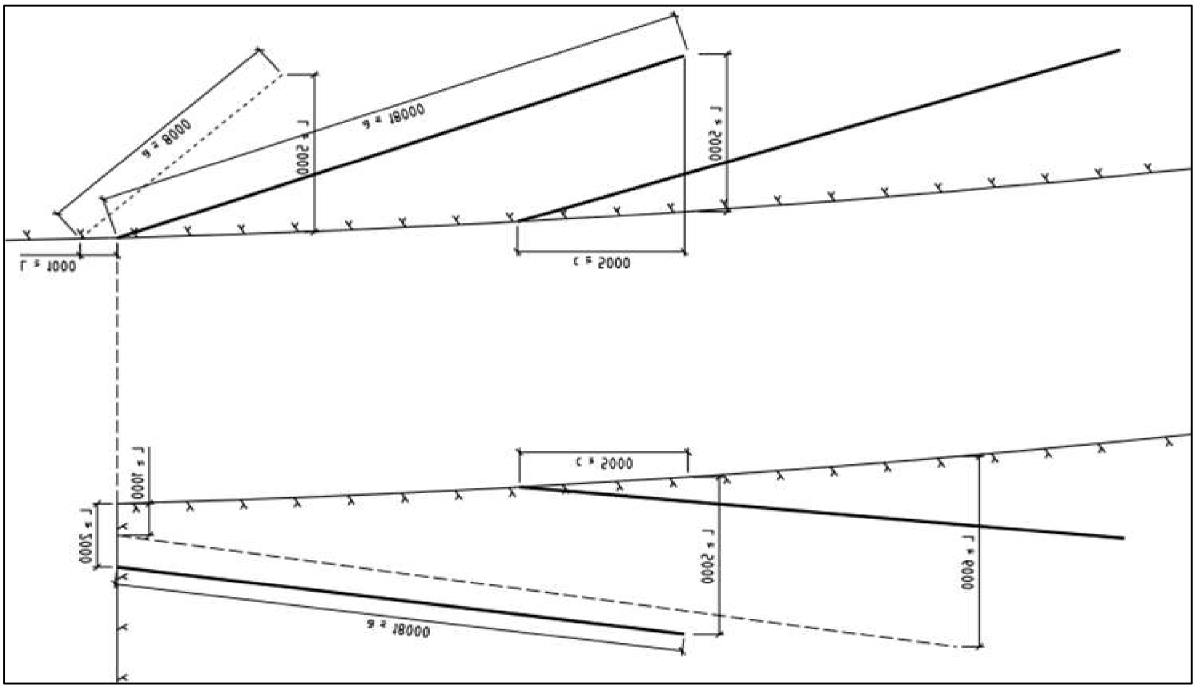

Figure A.19. Borehole's length, overlap, and stick in specific grouting fan at the pre-cut.

Table A.20.Stop criteria.

Grouting Pressure [MPa]	Instructions for stop conditions	Time [min]	Injected volume [Liter]
0.5	Grouting into the current borehole should be interrupted when the conditions for injected time or injected volume are reached.	15	300
1.0		12	300

Table A.21.Complemantary actions.

Event	Action
Prescribed grouting pressure is not achieved.	Grouting must be carried out with utility mixture wct=0,5 immediately after end of grout in the current hole.
The stop conditions for injected volume are reached before the stop conditions for injected time.	See technical description facility.

A.3 Weakness zones tables (IK3)

- Green: Favorable conditions
- Light green: relatively favorable
- Orange: relatively unfavorable
- Red: unfavorable

7 Londonviadukten

Table A.22.Lomdonviadukten-evaluation of geological conditions.

	Length measurements start km	**Length measurements stop km**	**Number of dominant crack-directions**	**Crack direction vs. tunnel**	**Cracks assessed endurance**	**Occurrence of crack-filling**	**Overall assessment**
AT1	0+036	0+046					
	0+046	0+085					
	0+085	0+115					
	0+115	0+285					
	0+285	0+320					
	0+320	0+440					
	0+440	0+821					
	Connection: N-track, S-track, Platform room, Service tunnel						

8 Järla Östra

Table A.23.Järla Östra-evaluation of geological conditions.

Length-measuring start lm	Length-measuring stop lm	Hydro-geological domain	Number of dominant crack-directions	Crack direction vs. tunnel	Cracks assessed endurance	Occurrence of crack-filling	Overall assessment
0+052	0+090	YTNBG					
0+090	0+240	YTNBG					
0+240	0+270	WEAKZ					
0+272	0+373	NORBG					
0+373	0+405	WEAKZ					
0+405	0+425	NORBG					
0+425	0+495	NORBG					
Connection: N-track, S-track, Platform room, Service tunnel		NORBG					

9 Sundstabacken

Table A.24.Sundstabacken-evaluation of geological conditions.

Length-measurement start lm	Length-measurement stop lm	Hydro-geological domain	Number of dominant crack-directions	Crack direction vs. tunnel	Cracks assessed endurance	Occurrence of crack-filling	Overall assessment
0+050	0+140	YTNBG					
0+140	0+214	NORBG					
0+214	0+280	YTNBG					
0+280	0+294	SVAGZ					
0+294	0+340	SVAGZ					
0+340	0+418	SVAGZ					
0+418	0+461	NORBG					

Connection: N-track, S-track, Platform room, Service tunnel	NORBG					

10 Hammarby Fabriksväg

Table A.25.Hammarby Fabriksväg-evaluation of geological conditions.

Length-measuring start km	**Length-measuring stop km**	**Hydro-geological domain**	**Number of dominant crack-directions**	**Crack direction vs. tunnel**	**Cracks assessed endurance**	**Occurrence of crack-filling**	**Overall assessment**
0+103	0+150	YTNBG					
0+150	0+200	YTNBG					
0+200	0+220	YTNBG					
0+220	0+250	SVAGZ					
Ventilationstunnel		YTNBG					
0+250	0+320	SVAGZ					
0+320	0+380	SVAGZ					
0+380	0+400	YTNBG					
0+400	0+490	NORBG					
0+490	0+530	NORBG					
0+530	0+578	NORBG					
Connection: N-track, S-track, Platform room, Service tunnel		NORBG					

11 Värmdövägen

Table A.26.Värmdovägen-evaluation of geological conditions.

Length-measuring start km	Length-measuring stop km	Hydro-geological domain	Number of dominant crack-directions	Crack direction vs. tunnel	Cracks assessed endurance	Occurrence of crack-filling	Overall assessment
0+027.5	0+060	YTNBG					
0+060	0+270	NORBG					
0+270	0+325	WEAKZ					
0+325	0+575	NORBG					
Connection: N-track, S-track, Platform room, Service tunnel		NORBG					

12 Skönviksvägen

Table A.27.Skönviksvägen-evaluation of geological conditions.

Length-measuring start km	Length-measuring stop km	Hydro-geological domain	Number of dominant crack-directions	Crack direction vs. tunnel	Cracks assessed endurance	Occurrence of crack-filling	Overall assessment
0+050	0+090	YTNBG					
0+090	0+121	NORBG					
0+121	0+150	SVAGZ					
0+150	0+321	NORBG					
0+321	0+352	SVAGZ					
0+352	0+365	NORBG					
Anslutning: N-spår, S-spår, Plattformsrum, Servicetunnel		NORBG					

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