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Condition Assessment for Wastewater Pipes

Method for Assessing Cracking and Surface Damage of Concrete Pipes

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Submission date: June 2013
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Title: Condition Assessment for Wastewater Pipes – Method for Assessing Cracking and Surface Damage of Concrete Pipes	Date: 05.06.2013 Number of pages (incl. Appendix): 114 Master Thesis
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Extract:
This is a Master of Science conducted at the Department of Hydraulic and Environmental Engineering at the Norwegian University of Science and Technology in spring 2013.

The objective of the Master Thesis was to review the method for condition assessment used in Norway today *NorVar Report 150/2007* (Bernhus, et al. 2007) and propose a methodology to overcome the limitations found in the Norwegian guideline. The focus is to improve the condition assessment of cracking and surface damage in concrete pipes.

The Thesis is based on the findings in my Specialization Project (Hauge 2012) and a literature study. Additionally a set of theoretical calculations is performed to develop a revised condition assessment method.

This Master Thesis will be a contribution to the *DiVA project* (DIVA 2011) financed by the Norwegian Research Council (NFR) and future condition assessment of wastewater pipes.

Key words:

1. Condition assessment
2. Concrete pipes
3. Cracking
4. Surface damage

____
(sign.)

Preface and Acknowledgements

This is a Master of Science conducted at the Department of Hydraulic and Environmental Engineering at the Norwegian University of Science and Technology in spring 2013.

I wish to thank my supervisor Professor Rita Maria Ugarelli at the Department of Hydraulic and Environmental Engineering, Norwegian University of Science and Technology for her invaluable assistance.

I would also like to thank Professor Sveinung Sægrov at the Department of Hydraulic and Environmental Engineering, Norwegian University of Science and Technology, and Torbjørn Smaaberg and Geir J. Lindholm at Rosim, for providing me with help and information during this project.

Gratitude is also given to 3D-Radar and Egil Eide, which has provided me with their information about geo penetrating radar.

Without the help of these people the implementation of this Master Thesis would be a lot harder.

Trondheim, June 2013

Petter Hauge

Abstract

The objective of the Master Thesis has been to provide an improved method for condition assessment, which will give a better correlation between *Condition class* and *actual Condition* of concrete pipes with cracking and/or surface damages. Additionally improvement of the characterization of cracking (SR) and surface (KO) damages was a sub goal.

Based on the findings described in my Thesis and my Specialization Project (Hauge 2012)*, I recommend that the Norwegian condition assessment method based on NV150, is revised.*

The revised condition assessment should focus on the severity of the damages based on measurable damage thresholds. According to NV150 cracks are not characterized and graded based on the measured crack width (see ***Table 2.2*** and ***Table 2.3***). There is no evaluation of the loads and forces influencing the pipe deterioration. My Specialization Project concluded that The Norwegian method gives too little weight to severe single damages when the pipe length exceeds 40 meters, and gives too high weight to distributed damages, as the condition score is very dependent of the damage length (Hauge 2012).

These facts show that adjusting the formula (***Equation 2.2***) for *condition score* calculation or the *damage grade* system (***Table 2.4***) in NV150 is not enough to improve the condition assessment method.

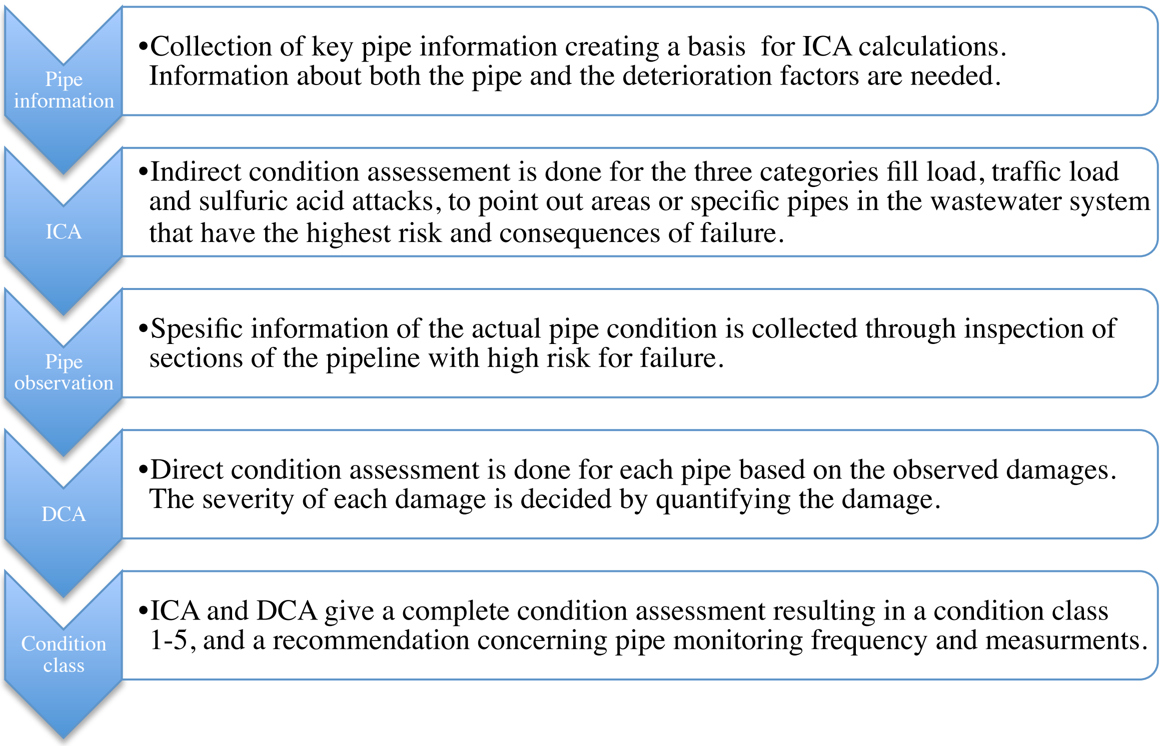
I recommend that a new way of assessing the pipe conditions, founded on research-based damage thresholds in the context of pipe deterioration processes is further developed.

Through this Thesis I have been able to establish damage thresholds for crack width and calculate critical strength in a pipe with decreased wall thickness (chapters 6.1 and 6.3).

I have developed a general method of assessing the loads and forces influencing a pipe without doing inspection, which I have defined as indirect condition assessment (ICA). This method can be used to assess how critical the influencing loads are. It can also be used to correct uncertain condition classes of observed damages defined by NV150.

Also I have developed a method of assessing the observations made through inspection defined as direct condition assessment (DCA). This method is an enhanced assessment of cracking and surface damages which are observed in a pipe.

A total assessment with both these two methods (ICA and DCA) is expressing the actual pipe condition better and more precise than the NV150. The recommended workflow is illustrated below:



To improve both the ICA and DCA further research is needed, and the following research topics can be listed based on the findings in this thesis:

Available tools like Norwegian Road Data Base (NVDB, see chapter 6.2.2) should be used to develop the ICA tool for traffic load assessment. The actual correlation between traffic load influence and YDT-L should be studied more closely. Also critical cover heights for all pipe sizes and dimensions should be calculated for ICA fill load input.

The use and implementation of new technics for trench and pipe bedding inspection, like geophysical methods, should be studied. Information about the bedding and side filling support on a pipe is very important to assess to be able to estimate loading and forces on the pipe.

Equipment for accurate measurement of cracking and internal diameter (wall thickness) should be developed. This information is needed for the estimate of critical fill load as motioned above, especially when the critical cover height decreases due to reduced wall thickness.

The production of sulfide in Norwegian sewer environment should be studied to determine critical retention time in pumping and pressure mains. Inspection robots can be equipped with gas sensors. All possible sources should be registered and their activity and sulfide production evaluated. This will give a basis to understand the correlation between pipes affected by sulfuric acid attacks and the possible reason for this.

Hopefully the method developed and presented in my Master Thesis will be a useful contribution to the *DiVA Project* and future condition assessment of wastewater pipes. The *DiVA Project* is financed by the Norwegian Research Council (NFR).

Sammendrag

Målet med denne masteroppgaven har vært å foreslå en forbedret metode for tilstandsvurdering av betongrør, som vil gi en bedre sammenheng mellom tilstandsklasse og faktisk tilstand med fokus på sprekk og / eller korrosjons skader. Et delmål er å forbedre karakteriseringen av sprekker (SR) og korrosjons (KO) skader.

Basert på funn gjort i mitt fordypningsprosjekt (Hauge 2012) og arbeidet i denne Master oppgaven, anbefaler jeg at den norske metoden for tilstandsvurdering (NV150) gjennomgås og forbedres.

Den nye og reviderte tilstandsvurderingen bør analysere skadenes alvorlighetsgrad basert på målbare skadeterskler. Dagens praksis gjøre ikke dette, dvs. sprekker karakteriseres og graderes ikke basert på målt sprekkbredde (se tabell 2.2 og tabell 2.3). I dagens tilstandsvurdering inngår ikke vurdering av belastninger eller krefter som påvirker rørets nedbrytning. Mitt fordypningsprosjekt konkluderte med at den norske metoden legger for lite vekt på alvorlige punktskader når rørets lengde overstiger 40 meter, og gir for høy vekt til distribuerte skader (Hauge 2012).

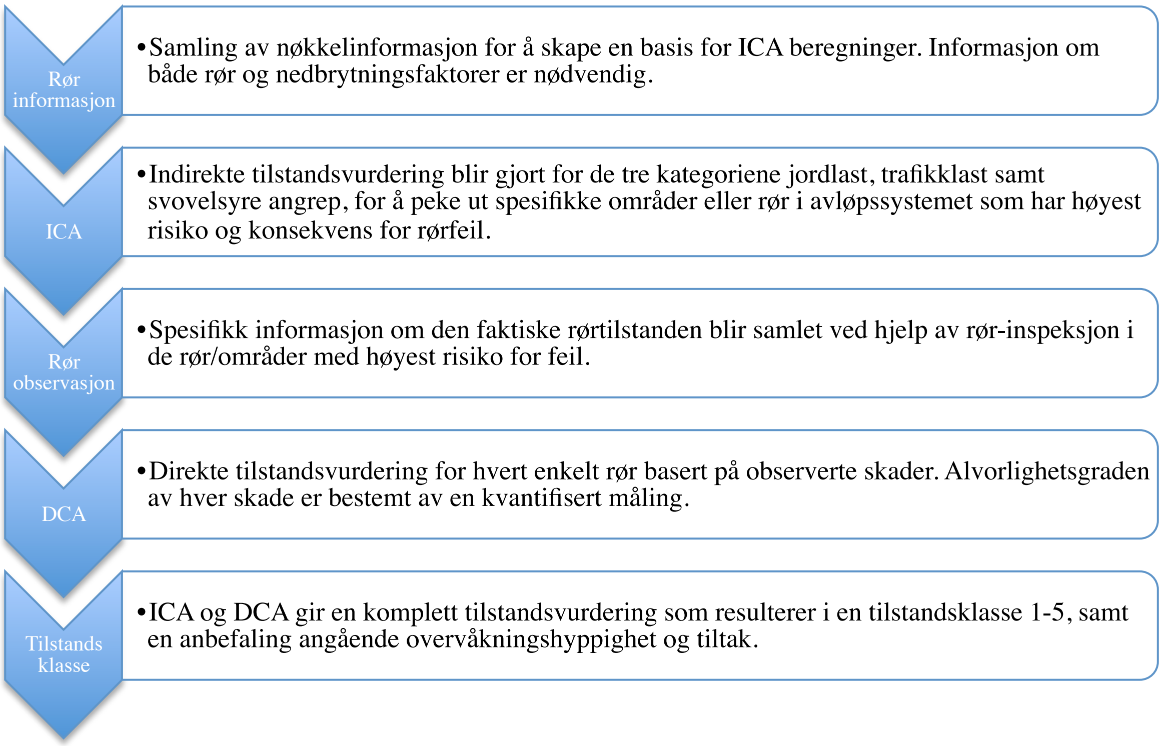
De overnevnte fakta viser at en justering av formelen (Equation 2.2) for *skadepoeng*, eller *skade graderings systemet* (Tabell 2.4) i NV150 ikke er nok til å forbedre tilstandsvurderingen.

Min anbefaling er at det utvikles en ny metode for å vurdere forholdene som påvirker røret, basert på målbar skadegradering, i sammenheng med en vurdering av prosessene som bryter ned røret.

Jeg har studert nedbrytningsprosesser, belastninger og krefter som påvirker avløpsrør av betong (kapittel 3 og 4). Gjennom dette arbeidet har jeg vært i stand til å etablere skadeterskler for sprekkbredde og beregne kritisk last på et rør med redusert veggtykkelse (kapittel 6.1 og 6.3).

I tillegg har jeg utviklet en generell metode for å vurdere de belastninger og krefter som påvirker et rør uten å inspisere det. Dette har jeg definert som en indirekte tilstandsvurdering (ICA). Denne metoden kan brukes til å vurdere hvor kritiske lastene og de observerte skadene er. ICA kan også brukes til å korrigere tilstandsklasser definert med NV150 hvis det er tvil om dette resultatets korrelasjon med faktisk tilstand.

Jeg har også utviklet en forbedret metode for å vurdere sprekker og korrosjonsskader observert ved inspeksjon, definert som direkte tilstandsvurdering (DCA). Observasjonene og vurderingen av disse skadene støtter opp om ICA. En samlet vurdering med begge disse to metodene (ICA og DCA) uttrykker rørets tilstand bedre og mer korrekt enn NV150. Den anbefalte arbeidsprosessen er vist i tabellen under.



Videre forskning for å forbedre både ICA og DCA er nødvendig, og følgende forskningstemaer fremkommet under arbeidet med denne avhandlingen.

Implementere verktøy som Norsk vei databank (NVDB) for å utvikle ICA for trafikkmengdebelastning. Sammenhengen mellom trafikkbelastning og årsdøgntrafikk for lange kjøretøy (ÅDT-L) må utredes nærmere. Også kritisk jordlast på grunn av overdekning må beregnes for alle rørdimensjoner for input til ICA.

Bruk av nye teknikker for grøft og rør inspeksjon, som geofysiske metoder, bør studeres. Informasjon om overdekningshøyde og rørets sidestøtte er svært viktig for å kunne estimere belasting og krefter på røret.

Nøyaktig utstyr for å måle sprekkbredde og innvendig diameter (veggtykkelse) bør utvikles. Denne informasjonen er nødvendig for å kunne gjøre en vurdering av kritisk jordlast, spesielt siden kritisk overdekning minker når veggtykkelsen reduseres.

Produksjonen av sulfid under norske forhold bør studeres nærmere for å kunne si noe om kritisk oppholdstid i pumpe-og trykkledninger. Inspeksjonsroboter kan være utstyrt med gasssensorer. Alle mulige sulfidkilder bør registreres, samt en vurdering av produksjons aktivitet og stabilitet. Dette vil gi et grunnlag for å forstå sammenhengen mellom de observerte angrepene av svovelsyre i rørene og den mulige årsaken.

Jeg tror også at metoden som er presentert i denne masteroppgaven vil være et nyttig bidrag til *DiVA-prosjektet*, finansiert av Norsk Forskningsråd (NFR) samt fremtidige tilstandsvurdering av avløpsrør.

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List of Abbreviations

CCTV	…………………………………………………..	*Closed-Circuit Television*
DCA	…………………………………………………..	*Direct Condition Assessment*
GPR	…………………………………………………..	*Ground Penetrating Radar*
ICA	…………………………………………………..	*Indirect Condition Assessment*
NV150	…………………………………………………..	*NorVar Report 150/2007*
NVDB	…………………………………………………..	*Norwegian Road Data Base*
SS	…………………………………………………..	*Side Support*
YDT	…………………………………………………..	*Year Day Traffic*
YDT-L	…………………………………………………..	*Year Day Traffic – Long vehicles*

1 Introduction

Basis for the Master Thesis

This Master Thesis is based on the findings in my Specialization Project (Hauge 2012), which was addressing the methods for condition classification of wastewater pipelines in use in Norway, the "Norwegian method", shown in the *NorVar Report 150/2007* (Bernhus, et al. 2007). The objective of my Specialization Project was to point out potential limitations with the Norwegian method compared to another method. The German method was chosen for comparison. Analysis and calculations of a set of 11 pipes from the city of Oslo were performed to exemplify and compare the Norwegian method to the German method. The condition classification methods were compared through a five-step process:

The main conclusions of the Specialization Project were:

The characterization step showed that the German method had a more detailed damage description than the Norwegian method. The Norwegian method is easy to apply, but describes just a minimum of the actual situation in the pipe. The German method is more complicated and comprehensive, but gives a more accurate description of the situation in the pipe.

The condition assessment and condition scoring steps showed that the Norwegian method for calculation of damage points was highly influenced by the damage length. The method gives too little weight to severe single damages when the pipe length exceeds 40 meters, and gives too high weight to distributed damages, as the condition score is dependent of the damage length. The German method determined the damage point regardless of the damage length and added points regarding the pipe surroundings, and thereafter corrected the condition score with an additional length factor.

Background

The background for this Master Thesis is the fact that the lag of maintenance of wastewater pipes in Norway is large, and a better tool how to plan for and rehabilitate the Norwegian sewer network is urgently needed. In 2011, 2400 sewage overflows were reported, which is the same as 67 overflows every thousand kilometer of pipeline. Although there were registered a reduction of 11% of overflows in 2010-2011 it still resulted in 400 cases of basement flooding's in Norway that year and as the house-owners usually hold the municipality responsible, there is a need for improvements.

The average age of the wastewater pipes in Norway is estimated to be about 30 years if we don't take into account the part of the pipes that we don't know the age of. The lack of information about the sewage network in Norway is a big concern; wastewater pipes are often built, buried and forgotten until a structural problem or defect appears, and for instance a basement flooding happens.

In 2011 there where a renewal of 156 of total 35 700 km of wastewater pipes which means 0.44 % of the total network. In the same period there were built 407 km of new pipelines. With that rate, it will take over 200 years to renew the whole sewage network, (KOSTRA 2012).

In addition to the actual situation described above in number and statistics, a combination of climate change, population growth and urbanization will further accelerate the need for renewal and investments to ensure a safe development. Accordingly, new methods for resource utilization and rehabilitation planning are required.

The *DiVA project (Digital water and wastewater management)* has a vision of providing a long-term planning tool, with the objective of making the wastewater network in Norway more sustainable. *DiVA* has a high focus on efficiency, use of new technology and right prioritizations to cope with the increasing demand and the lack of expertise in several minor Norwegian municipalities. The project is providing a tool to guide the users in information management and analysis, and helping to choose the right action for a given problem. *DiVA* performs a multi-criteria analysis. In addition to a condition assessment, it also considers risk, economic, social and environmental aspects, (DIVA 2011).

In Europe there are similar projects like *DiVA,* for instance the *Care-S project*, which is an alliance between 11 countries and 15 resource partners, including *NTNU* and *Sintef.*

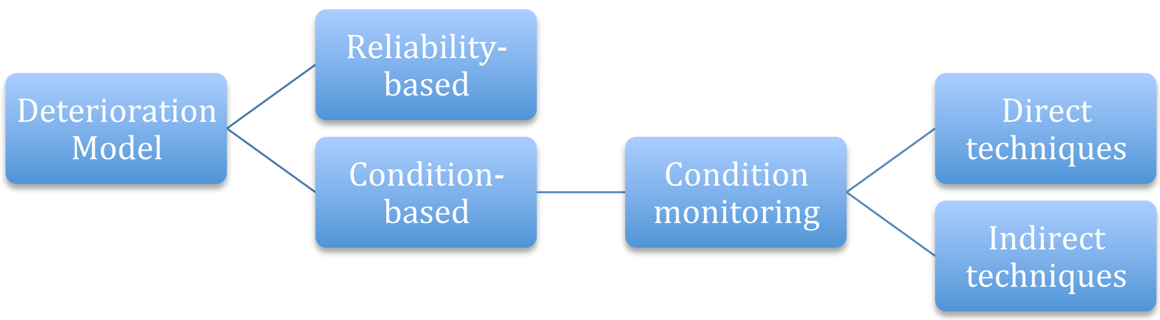
Care-S deals with both wastewater and storm-water pipes and considers problems like structural failure, infiltration, leakage and insufficient capacity in wastewater pipes. All these elements are important factors that can cause flooding, pollution of soil and water, and increased maintenance costs.

Care-S is like the *DiVA project* providing a tool for a cost-efficient system for rehabilitation, renewal and maintenance of wastewater pipes in a social, healthy, economic and environmental friendly way (CARE-S 2007).

The management of urban water systems maintenance is done through deterioration models. A good deterioration model of the wastewater system means that it gives us the possibility to rehabilitate and maintain the system at a cost efficient level, minimize the system downtime and provide the service required. The deterioration model can either be reliability-based or condition-based, where the last one is applied for the Norwegian wastewater systems.

The condition-based model evaluates the system's ability to provide the required service over time and uses the system condition monitoring as input to the model. As for today this is restricted to a condition assessment based on *closed-circuit television* (CCTV) inspection, which is a direct technic for condition monitoring. Using the Norwegian method for condition assessment, the result of the deterioration model does not give a good enough correlation to the real situation, as also shown in my specialization project (Hauge 2012).

To improve the condition monitoring indirect techniques, assessments as fill and traffic load, soil and bedding situation, age and tensile strength capacity should be applied. These are all factors that can cause damage to or create an unstable situation for a pipe. Also the vulnerability of critical areas in the sewer network can be assessed to enhance the prioritization of the monitoring (Ugarelli 2012).



Objectives

Condition assessment supports decision making in rehabilitation planning of wastewater networks. The selection of critical pipes to be prioritized depends on pipes performances. Pipes performances are influenced by structural conditions that impact both the structural reliability (probability to break) and hydraulic reliability (probability to not convey water as requested). There is a need to better understand the relationship between conditions and reliability.

The main objective of my Master Thesis is to provide a method to enhance the condition assessment of Norwegian wastewater systems, focusing on structural aspects, in particular cracks and surface damages of concrete pipes. Impact of pipes defects on hydraulic performance is not into the cope of this work. The ambition of this method is to contribute to more efficient wastewater maintenance programs, in terms of supporting prioritization decision of pipe maintenance. The method proposed in this thesis provides a better correlation between *condition classification* (ref. 2.4.1) and *actual condition*.

2 Description of the Norwegian Method for Condition Classification

In this chapter I will give an overview of the Norwegian NorVar *report 150/2007,* here by referred to as NV150 (Bernhus, et al. 2007), which is the current guidance for condition classification of wastewater pipelines. The report was first developed by Rørinspeksjon-Norge (RIN) or Pipe inspection Norway, an interest group for municipalities and private firms in the wastewater community, and the Norwegian Water and Wastewater BA (Norsk Vann). The report was issued in June 2007 and describes the condition classification method, the importance of pipe inspection and how to use the information provided in the pipe inspection.

I will further in my Thesis build on this guidance to develop my proposal for enhanced condition assessment. Accordingly, I will here focus on the main and most relevant elements of NV150 applicable for my Thesis.

2.1 NV150 Overview

The pipe inspections are basis for the network status and the condition classification. Here I briefly present the objectives of the pipe inspection and how the report is constructed. One of the purposes of NV150 is to suggest a better structure and date flow in the wastewater pipe rehabilitation work.

2.1.1 Information provided by pipe inspection

Pipe inspection is done to provide information about the sewer system and the current condition of the pipes. Specific information is collected concerning cracks, corrosion, hydraulic conditions caused by roots grown into the pipe and connection pipes incorrect installed. The actual methods for pipe inspection described in the report are all in-pipe direct observations.

The intended information provided:

- Location of operational disruptions in the wastewater pipes
- Systematic mapping of the condition situation of the wastewater network
- Systematic rehabilitation planning
- Control of new pipe installations
- Supervision of important and/or critical pipes

2.1.2 The Goal of the report (*NV150*)

The goal of the report is to standardize the pipe inspections, then categorize the information flow, and finally in the analytical part come up with a condition classification of wastewater pipes.

The report is divided in 4 main parts:

1. *Inspection methods:*
   The report emphasizes that awareness from both the buyer/client and the inspection operator should be increased.
2. *Data requirement from pipe inspection:*
   The result of the inspection should summarize the standardized observations, the defects and damages observed and give advice on how to handle damages as well as required datasets for pipe inspection.
3. *Planning modeling:*
   This part of the report describes the condition assessment and classification of the wastewater pipes. In short the condition classification is based on a condition score, which is a product of the damage points calculated from all the damages observed in a wastewater pipe.
4. *Registration, storage and use:*
   The amount of data is quite extensive and you have to make sure that the data is processed for systematical storage, with a possibility of a GIS based overview. For further details of the registration element it is referred to the *NV150,* as it will not be explained or discussed further in this report.

2.2 Inspection Methods

Pipe inspection is an expensive and time consuming process, and therefore it is important to have a clear defined goal of the outcome of a given inspection. The methods or reasons for an inspection are mainly divided in to 5 categories respectively described below: Locating damages, Systematic surveying of pipe conditions, Systematic detailed rehabilitation planning, Control of new installations and Survey of important/critical pipes. This chapter also presents the requirements with regards to agreements, deviations and data registration.

2.2.1 Locating damages

The goal is to locate damages and find reasons for operational disruptions in a wastewater pipe. The main focus is to inform the inspection operator of any suspicions so that he or she has the best possible understanding of the situation in the wastewater pipe.

Actual problems searched for are for instance clogging's, big cracks and leakages. It is important that the equipment has the possibility of looking into connected pipes that leads to the main wastewater pipeline. Requirement of *NV150* is that an inspection unit must have a *Satellite camera* with a reach of 20 – 30 meters.

2.2.2 Systematic surveying of pipe conditions

The main goal here is to have systematic surveys of the pipe conditions and keep track of the maintenance and rehabilitation needs in a city area. This survey requires high performance equipment and good quality reporting. *NV150* recommends cameras that enhance detailed observations, for instance a *wide-angle* or a *rotatable camera lens*. *Satellite camera* is also recommended.

Survey of wastewater pipelines in operation requires high attention by the operator and the quality of the equipment to achieve the expected results. Particularly important in this regard is the inspection of pipelines with rising water level, depressions, and/or areas with sediment settling.

2.2.3 Systematic detailed inspection for rehabilitation planning

This type of inspection gives the basis for choosing the most suitable rehabilitation method. Observations may exist but can be old and verification is needed. The main focus is on the pipeline capacity rather than the condition. The hydraulic potential and in that case the diameter is assessed to evaluate the need for higher hydraulic capacity and potential increase in pipe diameter.

NV150 recommends a high focus on details and a high quality report to be able to give the exact description of measures and chose the right method of rehabilitation. The following list of elements are recommended to be described in detail:

- Pipe diameter
- Change in pipe diameter
- Connection of private drains
- Offset of pipe connection
- Cracks
- Deformations
- Bends and curves (change in angle)
- Exact length form manhole (base) to observation

NV150 requires photos of all defects with grade 3 and 4 (grading is presented in chapter 2.3.1).

2.2.4 Control of new installations

NV150 recommends a high quality inspection and reporting from new installations. The reason for this is that the city or municipality, which has the ownership to the wastewater pipelines, is responsible for controlling the new installation and ensure that the quality of the construction work is sufficient. This survey should be done right after the installation is finished.

The accuracy of the inspection is important, especially when investigating deformations and pipeline slopes, and *NV150* here requires calibrated measuring equipment. A control report like this is the only evidence that holds in court, in case of a dispute between contractor and municipality.

2.2.5 Survey of important or critical pipes

Regular surveys of pipes are important to reduce the risk and prevent critical cases of collapse and/or leakage of sewage or polluted water to a sensitive recipient area.

NV150 requires a high quality level of the reporting with the intention to measure changes in the pipe over time. Particularly it is important to have an accurate measure of the length from manhole (base point) to critical point in the pipe.

2.2.6 Inspection agreement

NV150 specifies that two conditions have to be clarified when client and inspector agrees on terms for a pipe inspection:

1. The purpose of the pipe inspection (specified in the 5 previous chapters)
2. Basis for the pipe inspection (Pipeline network map and registered pipeline data)

2.2.7 Deviation and registration

The information database of the wastewater network may often have deviations from the real situation, and this must be discovered by the inspection and corrected. Registration of deviations is done in 2 sequences: The deviation is registered during inspection; this is the control of the manhole outlines and pipe diameter. Secondly the deviation is registered during report reviewing, which may be information about connection of drains, pipe material, change in material, change in pipe diameter and bends.

NV150 recommends that it is written a deviation statement when a deviation is discovered. This has to be controlled by the responsible part (municipality) before any corrections in the information database are executed.

2.2.8 Software for inspection and information databases

Two types of software are used to handle data collected from pipe inspections, and they are normally categorized as software used during pipe inspection and software used for pipe information databases.

The first type of software, used during pipe inspection is designed to store information collected on a digital platform. A standardized coding system expresses the different observations (described in chapter 2.3.1). *NV150* recommends that the software is equipped to register and save the following information:

- Length from a base point to an observation
- Length or number of observations
- Coding system of standardized observations
- Possibility to store other not standardized observations
- Photos that elaborate the observations

- Video footage (with sound) from the inspection

The second type of software is used to store and handle information about all pipes in the whole wastewater network. This is intended to keep track over the overall condition in the wastewater system. *NV150* recommends that the software can handle data like:

- Pipeline data; position, diameter and material
- Registrations and notifications; operational disturbances and/or maintenance.
- Analytical results; pipe condition classification
- Other information, for instance video of pipe inspection

2.3 Data Registration from Pipe Inspection

This chapter defines the requirements for the damage observation registration, the grading description of cracks/fissures and surface damages, description of distributed and point damages and finally reasons for rising pipe water level.

2.3.1 Observation registration

During a pipe inspection observations are made and this information is used in software that calculates condition classification. Damages are categorized as material/structural damages or operational defects. Other observations are categorized as constructional/inventory observations or other observations. The different observations, defects and damages are listed in ***Table 2.1*** below.

Category:	Damage:	Code:
Material/structural damages:	Deformation	DF
	Cracks	SR
	Corrosion	KO
	Product defects	PF
	Intruding connection	IR
	Connection defect	TF
	Defect reopening of connection (after rehab.)	DG
	Defect hattprofile	DH
	Visible/loose gasket	SP
	Displaced joint	FS
	Defect transition part	DO
Operational defects:	Roots	RØ
	Deposits/coating	UB
	Sediments	SM
	Obstructions	HI
	Infiltration	IS

Constructional/Inventory observations:	Pipe connection Clogged connection Rehabilitated section Direction change Cross section change Material change Decrease or increase of diameter Drop man hole	TK PT PR RE TE ME DE FK
Other observations:	Cancelled inspection Water level in pipe Flow from connected pipe Decreased visibility	IA VN VS DS

Table 2.1: Categorical overview of damages according to NV150

All observations are evaluated by the operator and given a 2-letter code, according to the coding system in ***Table 2.1***. The actual point of start and end of each observation are also noted in the software.

NV150 states that some cases need a more accurate reporting and measuring. These cases are:

- Deformation on newly installed flexible pipes
- Accurate cross section measurements on rehabilitated concrete pipes
- Inspection of structural damages on concrete pipes without reinforcement or plastic pipes

NV150 also recommends that a review of CCTV inspection is done where water flow and rising water level are assessed.

2.3.2 Grading description of cracks/fissures and surface damages

As mentioned above, the inspection manual *NorVar project report 145/2005* describes how to grade the different observations. In this master thesis, cracks/fissures (SR) and surface damage (KO) are the focus area and therefor described here in detail. (See Appendix 1).

2.3.2.1 Cracked pipe, SR

Definition	Crack: The crack is visible on the pipe wall, but the pipe is not deformed and all parts are still in its place Burst: Parts of the pipe is loose, but not missing Collapse: A total structural breakdown
Grading	1. Fissures. Some shelling of the concrete and bricks are partly loose 2. Open crack and bricks are loose 3. Parts of the pipe wall are loose or missing. Bricks are missing from initial position 4. Collapse
Characterization	• Longitudinal cracks • Circumferential cracks • Complex cracks • Shelling (loss of concrete)
Inspection requirements	Position of the crack on the pipe wall must be noted (clock position)
Measured values	Grade 1 and 2: The value must be noted, if crack width is measured Grade 3 and 4: The value must be noted, if the length of the burst/collapse is measured
Requirements	Measure the cracks width in millimeters

Table 2.2: Description of cracks grading. See Appendix 1 for examples of grade cases

2.3.2.2 Surface damage/Corrosion, KO

Definition	The pipe wall surface is damage by chemical substances/corrosion or mechanical abrasion.
Grading	1. Increased roughness (pipe wall is affected) 2. Visible aggregates (pipe wall is significantly affected) • Concrete pipes, aggregates clearly exposed • Clay pipes, missing coating • Cast iron pipes, some corrosion damage • Brick pipes, missing mortar 3. Missing aggregate or visible reinforcements (pipe wall is strongly affected) • Concrete pipes, aggregate is missing or visible reinforcement • Cast iron pipes, strong corrosion damage • Brick pipes, porous surface and missing mortar 4. Pipe wall is missing
Characterization	Not specified
Inspection requirements	Position of the crack on the pipe wall must be noted (clock position)
Measured values	Not specified
Requirements	Not specified

Table 2.3: Description of surface damage grading. See Appendix 1 for examples of grade cases

2.3.3 Distributed and point damages

NV150 distinguishes between distributed and point damages. The inspector registers lengths of all damages, but if the damage is not distributed, the numbers of such damages are registered. These damages are characterized as *point damages*. Other damages, characterized by its length (in meters), are called *distributed damages*.

NV150 states when a point damage is observed, it should be marked with a characterization code, for instance *point deformation (DF),* and given a grade 1-4. When a distributed damage is observed it is also marked with a characterization code. Both the start point and the end point of the distributed damage are marked with a length from the base point, and the damage is given a grade 1-4, or a percentage of the water level (see chapter 2.4.1.3). The reason for these specifications is the need for a basis to calculate the condition score for a pipe; this will be further elaborated in chapter 2.4.1 (*Condition classification*).

2.3.4 Pipe water level

NV150 specifies that there are three main reasons for water level or a change in water level in a pipe:

1. Depressions in the pipe may prevent the water from flowing freely and retain the water
2. Surcharge from a downstream pipe
3. Surcharge because of sedimentation and gravel

All these three cases can lead to a rising water level in the pipe, which again may cause deposits of organic material (like fat), and sedimentation of other inorganic substances (like suspended solids and particles). A sufficient layer of deposits may lead to a decrease of the pipeline's effective area and enhance the possibility of clogging. Other problems related to this, are increased hydraulic friction/roughness, which leads to decreased flow capacity.

The water level in the pipeline depends on the water flow, which is continually changing, and is therefor not a very precise measurement. The method of calculating the grade of water level will be described in chapter 2.4.1.3 (*Additional scoring*). Although the method is not very precise, the water level is an indication that a pipe may have hydraulic weaknesses, related to the reasons explained above.

2.4 The Norwegian Condition Classification Method

This chapter describes how the Norwegian condition classification method is defined. Condition classification is the first step in developing a plan for rehabilitation and maintenance. The level of a pipeline's condition is determined through the wastewater pipeline condition classification. 3 "ambition levels" are of importance and described below:

Condition classification:

- Observations and damages given a certain grade multiplied with the length or number, gives the damage points
- Pipe condition is expressed as a condition scoring, based on the sum of the damage points multiplied with a constant and divided by the total pipe length

Function classification:

- Based on the condition classification
- Operational reliability
- Capacity
- Impermeability

Condition measurement:

- Conditions like the pipes criticality and importance in the network system

- Critical possible flooding situations downstream
- Environmental considerations; soil and recipient sensitivity
- Operational conditions

As we can see, the 3 ambition levels are covering different areas. The condition classification is calculated based on the damages observed, which will be explained in the next chapter 2.4.1. The function classification describes the total situation in the pipe and not only the structural stability, but depended on the condition classification. The condition measurement takes into account the surroundings of the pipeline and the consequence of a leakage; the probability of a failure is based on the function classification.

2.4.1 Condition classification

This chapter describes the method for calculating the condition classification, which is a central method applied in my Thesis. The process to come up with the condition classification can be described as a flow sheet starting with the observations followed by the characterizations, further condition assessment, following condition scoring and finally the condition classification. ***Figure 2.1*** displays the workflow of condition classification. The observation and characterization steps are presented in the former chapter and the next 3 steps are focused here.

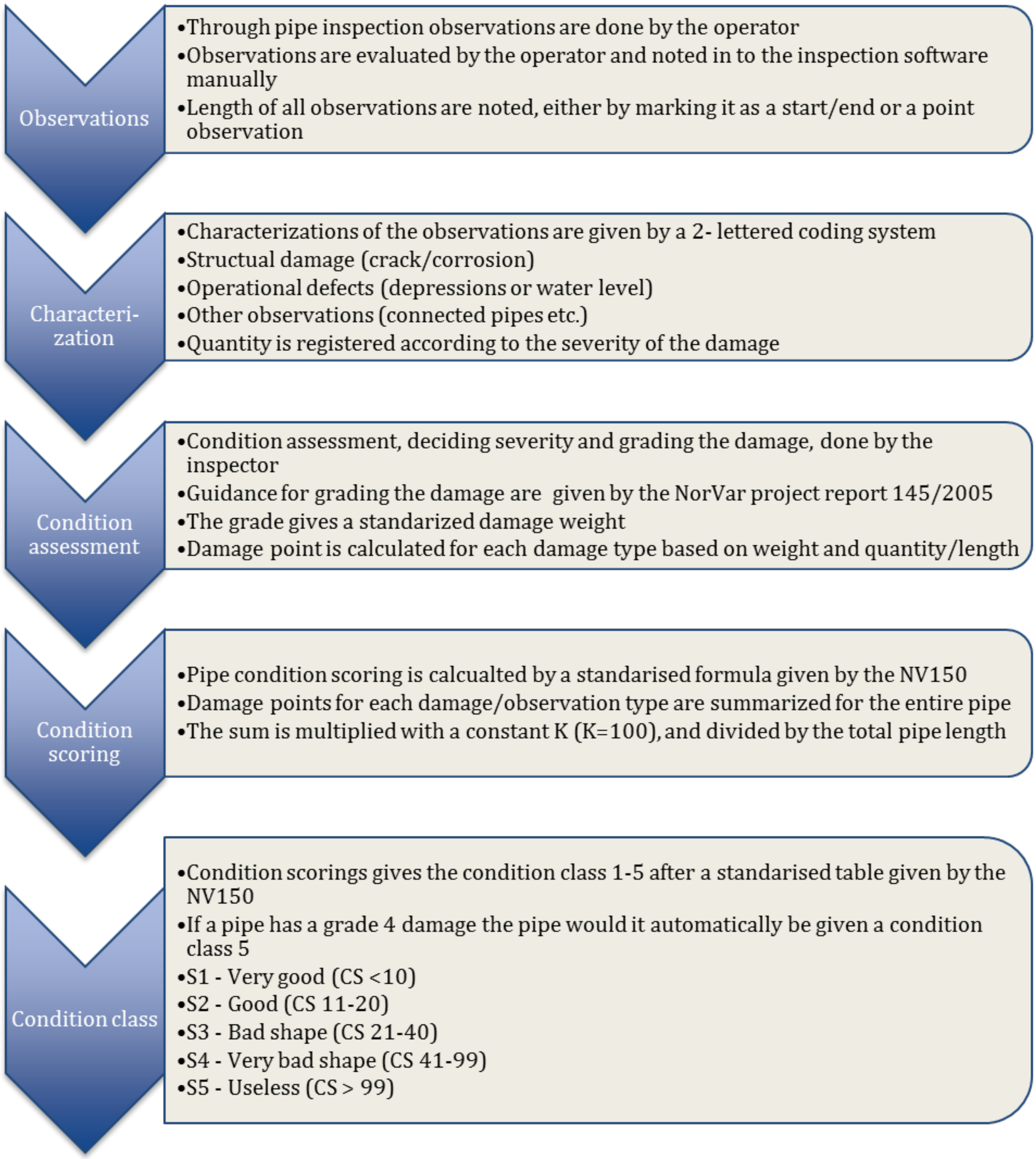


Figure 2.1: The Norwegian condition classification method **(Hauge 2012)**

2.4.1.1 Condition assessment

The result of the condition assessment is the calculated *damage point* for each damage type. The parameters needed for this calculation are the *damage grade*, and the *length* or *number* of observations of the given damage type.

Guidance to determine the damage grade is described in NorVar project report 145/2005 (Haugen, et al. 2005), which is based on NS-EN 13508-2, (CEN 2011) it is referred to these reports for further details beyond the description given here.

The damage grade describes the severity of the damage, i.e. the size of a crack, or percentage of pipe area reduction caused by a deformation or blockage. Damage grade for a distributed damage is, likewise given by the severity.

The damage grade for each damage type is given a certain weight (P_x), which is shown in ***Table 2.4***. The other parameter is the length or number/count (L_x) for respectively distributed or point damages, which are registered through the pipe inspection. The formula for calculating the damage point for each damage type is given below:

$$DP_X = \Sigma(P_X \cdot L_X)$$

Equation 2.1: Damage points

Observation		Grade / Weight (P_x) / Quantity (L_x)								Damage points
Type	Code	Grade 1		Grade 2		Grade 3		Grade 4		
		P_x	L_x	P_x	L_x	P_x	L_x	P_x	L_x	
Cracks	SR	0		2		8		12		$\Sigma(P_{SRx} x L_{SR})$
Corrosion	KO	0		1		3		9		$\Sigma(P_{KOx} x L_{KO})$
Product defects	PF	0		2		4		6		$\Sigma(P_{PFx} x L_{PF})$
Deformation	DF	0		1		2		4		...
Connection offset	FS	0		0		5		9		...
Visible/loose gasket	SP	0		1		2		3		...
Roots	RØ	0		1		5		9		...
Infiltration	IS	0		1		6		9		...
Sediments	SM	0		0		1		2		...
Desposits/coating	UB	0		1		2		3		...
Obstructions	HI	0		1		3		9		...
Connection pipe visible	IR	0		1		3		9		...
Connection defect	TF	0		1		3		9		...
Defect reopening of connection (after rehab.)	DG	0		2		3		12		...
Defect hattprofil	DH	0		1		6		9		...
Defect transition part	DO	0		1		3		9		$\Sigma(P_{DOx} x L_x)$
										$\Sigma[DP(x_i)]$

Table 2.4: Overview of damage types and corresponding damage grades, for calculation damage points **(Bernhus, et al. 2007)**

2.4.1.2 Condition scoring

The next step after calculating *damage points* for each damage type is the calculation of the *condition score* for the entire pipe. The condition score is calculated by summarizing the damage points for all the different damage types registered in a given pipeline, multiplied with a constant *(K),* and dividing the product by the total pipeline length. The formula used to calculate the *condition score* is given under:

$$CS = \frac{K \cdot [(\Sigma P_{SRx} \cdot L_{SR}) + (\Sigma P_{KOx} \cdot L_{KO}) + \cdots + (\Sigma P_{DOx} \cdot L_{DO})]}{L_{tot}}$$

Equation 2.2: Condition score

The constant (K) is always equal to 100.

2.4.1.3 Additional scoring

An additional score is given if there is a standing water level in parts of the pipeline. During the pipeline inspection the water level is registered as a distributed damage, and the length registered. The grade of the water level is determined by the percent coverage of the pipeline area. The reasons for the water level or surcharge are given exemplified above in chapter 2.3.3. Measurement of the water level is also a method to measure the depth of a depression. By estimating the water level in a depression, after sediments and deposits are removed, it can be compared to the downstream water level and a ΔVN is found, as shown in ***Figure 2.2***.

$$\Delta VN = VN2 - VN3$$

Equation 2.3: Water level

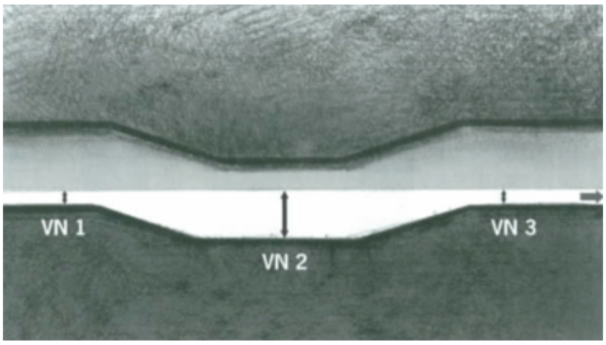


Figure 2.2: Measuring water level in a depression **(Bernhus, et al. 2007)**

The value of ΔVN gives the grade of the water level, which again gives the weight of the damage and the additional damage point can be calculated by adding the length. In ***Table 2.5*** the grade, ΔVN boundaries and weight are given.

Grade	ΔVN [%]	Weight
0	0	0
1	5 – 15	1
2	20 – 35	4
3	40 – 60	9
4	65 – 100	12

Table 2.5: Grades and weight of water level

The additional scoring is calculated in the same way as the condition scoring. All the additional damage point is summarized, multiplied with a constant (K=100) and divided by the total length of the pipe, as given in the formula under.

$$AS = \frac{K \ \cdot (\Sigma P_{VNx} \cdot L_{SR})}{L_{tot}}$$

The additional score is added to the condition score and gives the total condition score. If there is no standing water level in the pipe, the additional score is zero.

2.4.1.4 Condition class

The condition class for each pipe is given by the total condition score, which is described above. ***Table 2.6*** shows the condition score within a given range corresponding to the condition class 1-5, where 1 is *very good shape* and 5 is *useless*.

Condition class	Condition score	Definition
S1	0 – 10	Very good shape
S2	11 – 20	Good shape
S3	21 – 40	Bad shape
S4	41 – 99	Very bad shape
S5	> 99	Useless

Table 2.6: Condition classes, score boundaries and definitions

As shown, the condition score determines the condition class; however, there is one exception to this. If one or more observed damages, of a certain type, in a given pipe is evaluated and given grade 4, that automatically will give a condition class of 5, regardless of the total condition score value. Even if the condition score is lower than 99 points. This is because grade 4 damage is so severe for the pipe stability or operational reliability that immediate rehabilitation measurements are needed. These types of damages are given in ***Table 2.7***.

Category:	Damage type:	Code:
Material/structural damages:	Cracks Corrosion Connection pipe visible Defect reopening of connection (after rehab.) Visible/loose gasket	SR KO IR DG SP
Operational defects:	Roots Obstructions Infiltration	RØ HI IS

Table 2.7: Grade 4 damage types which gives CC=5

Even though there are 5 condition classes, *NV150* point on 3 main actions to be performed based upon those 5 classes:

- < 20, does not need any rehabilitation (S1 and S2)
- 20 – 40, may need rehabilitation (S3)
- > 40, needs rehabilitation (S4 and S5)

NV150 also recommends that the regular condition classes are not used on short pipes (5-20 meters), because this will generally give too high scores as you divide on the total pipe length.

3 Cracking in Concrete Pipes

The development of cracks in buried concrete pipes is a result of the static and dynamic load on the pipe. Normally a concrete pipe is designed to withstand the fill load and traffic load, which are the dominant loads. However, in situations where the installation work of the pipe trenches is of poor quality, the actual load on the pipe can exceed the design load and cracking or in some cases pipe failure may occur.

If the back filling masses in the trenches are redistributed because of erosion depressions will occur, which in turn may cause cracking of the pipe because of concentrated loading on isolated areas on the pipe wall.

To categorize the type of cracks occurring in concrete pipes, usually it is distinguished between longitudinal and circumferential cracks. That is because the reason for crack development is different in these two categories.

The actual age of the pipeline is also relevant information for condition assessment.

3.1 Longitudinal Cracking

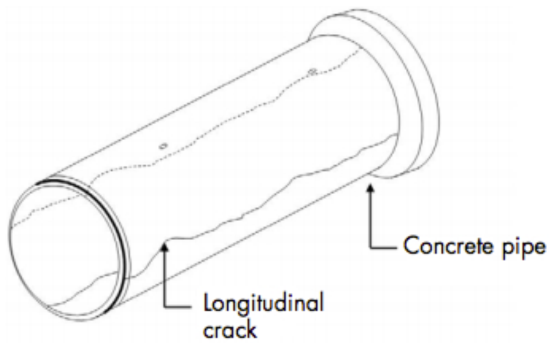


Figure 3.1: Example of longitudinal cracking **(CCPA 2011)**

Longitudinal cracking describes the cracks that run along the pipe axis as exemplified in the ***Figure 3.1*** above. These types of cracks usually occur when the pipe is overloaded and the vertical pressure exceeds the pipe tensile strength. The cracks and fractures can be detected at the top and bottom, which are usually referred to as 12 and 6 o'clock on the inside wall of the pipe. The same situation may also cause longitudinal cracks at 3 and 9 o'clock on the outside wall and those cannot be detected using conventional CCTV inspection techniques from the inside of the pipe.

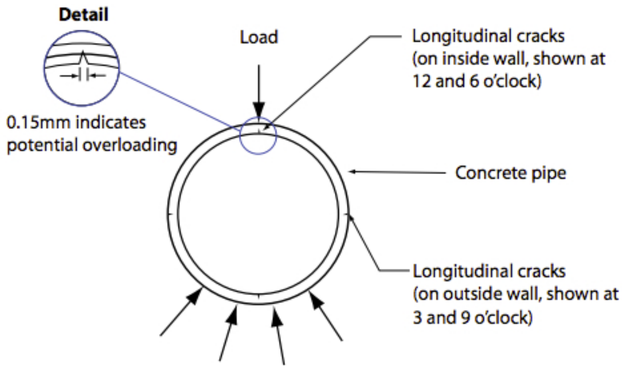


Figure 3.2: Examples of areas where longitudinal cracks appear **(CCPA 2011)**

Figure 3.2 above shows the typical areas of longitudinal cracking. *Concrete Pipe Association of Australasia* (CCPA) states that a 0.15mm crack, at top and bottom is an indication of potential overloading. CCPA also states that all cracks up to 0.5mm are not a risk if the crack is stable and not growing. If the crack is wider than 0.5mm, long term effects of these needs to be assessed (CCPA 2011).

CCPA has made a guideline for acceptance and assessment of longitudinal cracks. ***Table 3.1*** shows the recommended actions for different crack sizes, taken from the engineering-guideline (CCPA 2011).

For Pipes with Longitudinal Cracks – Top and Bottom	
Size of Crack	Action Recommended
< 0.15 mm	No action required – crack unlikely to extend through the wall and equivalent to the design serviceability load crack defined by AS/NZS4058.
0.15-0.50 mm	Monitor for stability of crack. Cracks up to 0.5 mm are not considered to be a durability risk. If crack is stable, no further action required.
> 0.50 mm	Engineering assessment required to consider effects of long term loads.
For Pipes with Longitudinal cracks - Sides	
< 0.15 mm	No action required. If pipe is in final loaded condition, the imposed loads will close longitudinal cracks in the spring zone.
> 0.50 mm	Engineering assessment required. Repair may be deferred to allow autogenous healing to occur. If after agreed monitoring period autogenous healing has not occurred refer to recommended repair.

Table 3.1: Acceptance and Assessment Chart, taken from **(CCPA 2011)**

Figure 3.3 under shows a 1mm crack in a DN375 concrete pipe, which was probably caused by overload. To assess this crack it is advised to consider the long-term load effects corresponding to ***Table 3.1***.

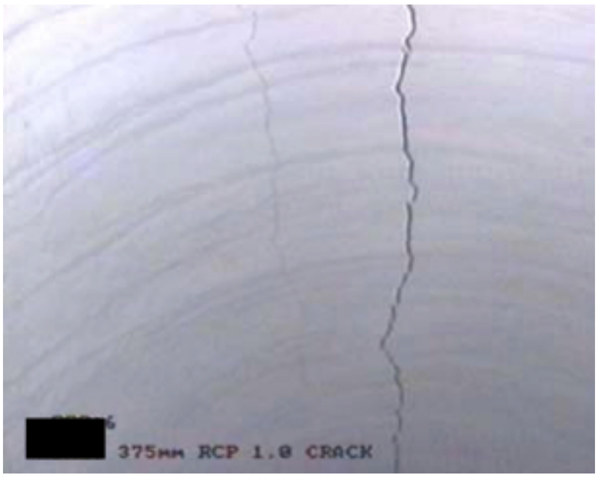


Figure 3.3: 1mm crack at 12 o'clock of a DN375 pipe, possible cause is overloading **(CCPA 2011)**

3.2 Circumferential Cracking

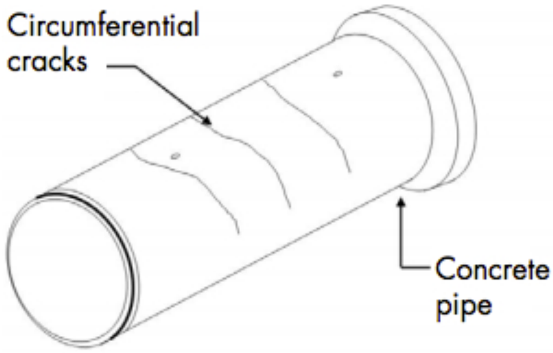


Figure 3.4: Examples of circumferential cracking **(CCPA 2011)**

Circumferential cracks describe the cracks that run perpendicular on the pipe axis as exemplified in ***Figure 3.4*** above. These cracks are a result of uneven bedding under the pipe resulting that the pipe segment is loaded like a beam. The circumferential cracks appear both on the top and bottom of the pipe depending on where the concentrated bedding pressure is situated on the pipe segment. In some cases the cracks can be seen as a full circumference of the pipe.

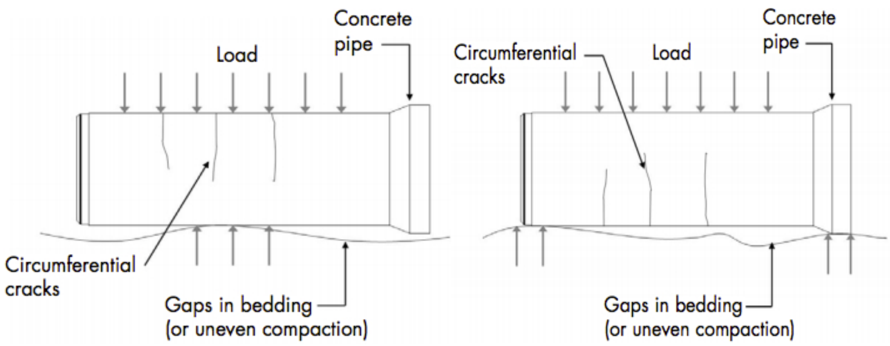


Figure 3.5: Different load situations and corresponding location of the circumferential cracks **(CCPA 2011)**

Figure 3.5 above shows how different load situations result in different areas with circumferential cracks. If the bedding pressure is concentrated under the middle of the pipe segment, the cracks appear between 9 and 3 o'clock (top half of the pipe). If the bedding pressure is concentrated under the two ends of the pipe segment the cracks will appear between 3 and 9 o'clock (bottom half of the pipe).

The most common cause of these types of cracks is gaps or uneven compacting of the bedding under the pipe, which often is a result of poor construction work or erosion caused by a water flow through the trenches (CCPA 2011).

CCPA has made a guideline for acceptance and assessment of circumferential cracks. ***Table 3.2*** shows the recommended actions for different crack sizes, taken from the engineering-guideline (CCPA 2011).

For Pipes with Circumferential Cracks	
Size of Crack	Action Recommended
< 0.15 mm	No action required
0.15-0.5 mm	No action required, allow autogenous healing to take place
0.5-1.0 mm	Monitor and allow autogenous healing and review after 12 months
1.0-2.0 mm	Assess potential for fines to migrate through crack (which may degrade the bedding)
> 2.0 mm	Crack likely to be all way around and may require repair or replacement of pipe

Table 3.2: Acceptance and Assessment Chart, taken from **(CCPA 2011)**

3.3 Mechanical Loads on Concrete Pipes

The total mechanical loads on a concrete pipe consist of four different types of loads. It is a sum of cover load, fill load, wheel load and water pressure inside the pipe. The longitudinal and wheel loads are usually the most significant and the remaining two are often neglected. In addition the total load is multiplied by a factor, which takes into account the long-term effects of compression of the filling on both sides of the concrete pipe. This means that the total mechanical load on the pipe over time is larger than the weight of the vertical column right above the concrete pipe.

Fill load is in this thesis expressed as the effective weight of the surrounding soil, applied to the pipe. The wheel load is expressed as such loads produced by ground-transported traffic.

The calculation methods of the loads mentioned above will be explained in the next chapters.

3.3.1 Fill load theory

There are different methods for calculating fill load on buried concrete pipes, but the most common method, and the one I will explain the *Marston load theory*, which is based on a section of soil that imposes a load on a buried pipe (Moser 2001).

The Marston theory assumes that the fill above the pipe is equally or more compressible than the fill on both sides of the pipe. Since the concrete pipe is very rigid the side fills will compress over time and because of friction in the soil above, the pipe will carry the weight of a prism wider than the pipe diameter.

The Marston theory distinguishes between *trench conditions* and *embankment conditions*. Originally trench conditions correspond to quite narrow trenches not that relevant and I will focus on the embankment conditions to explain the load coefficient. The most usual calculation methods used in Norway are also based on embankment conditions with positive projection. This method will be explained in chapter 3.3.1.2.

3.3.1.1 Embankment condition with positive projections

When a trench is sufficiently wide there will be compressions in the column above the pipe (here called the interior prism), and in the column on bot sides of the pipe (here called the exterior prism). Then the embankment condition for calculating longitudinal load needs to be applied.

When a pipe is located at the bottom of the embankment, the exterior prism will have a downward motion relatively to the interior prism, which generates a vertical shear plain between the two prisms. If the total settlement in the exterior prism is greater than the total settlement in the interior prism there will be downward shear strength in the exterior prism boundary layer to the interior prism. This is what Marston defined as a *positive projection* and is also the phenomenon observed in a wide trench where a rigid pipe is buried. This is because the rigid pipe is a part of the interior prism height, which gives it a smaller potential for settlement compared to the exterior prism.

If the embankment is deep enough a *plane of equal settlement* will appear. Above this plane the interior and the exterior prism will settle equally and the effect of the projection will not be noticeable. This makes two distinguished cases, one with an *incomplete projection condition*, meaning the embankment is deeper than the given plane depth (See ***Figure 3.6***, left), and another with *complete projection conditions,* meaning the embankment is not deeper than the given plain depth (See ***Figure 3.6*** right) (Trott and Young 1984).

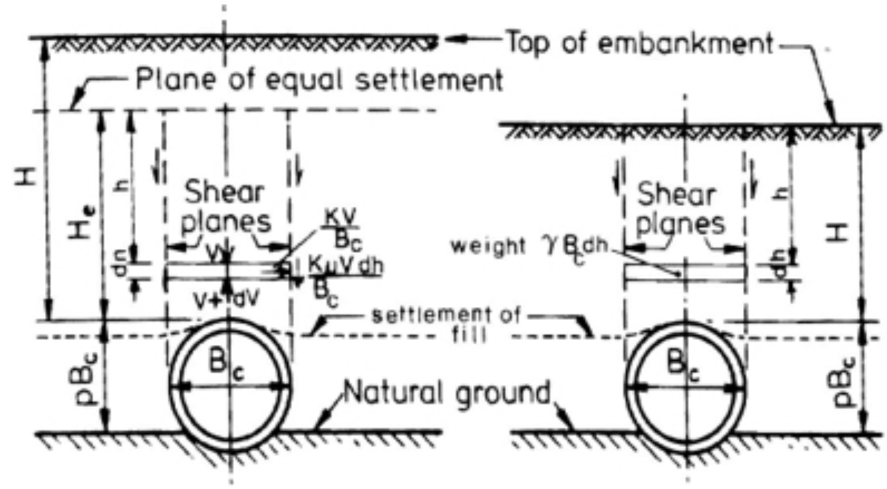


Figure 3.6: A pipe in an embankment with positive projection. Situation to the left is an incomplete projection and the one to the right is a complete projection **(Trott and Young 1984)**

The load of the soil on a pipe in an embankment (or a wide trench) with positive projection (i.e. rigid pipes) can be calculated as follows:

$$W_c = C_c \gamma B_c^2$$

Equation 3.1: Fill load

Where:
$W_c = Fill\ load\ on\ the\ pipe\ [kN/m]$
$C_c = Loading\ coefficient$
$\gamma = Desity\ of\ the\ soil\ [kN/m^3]$
$B_c = Pipe\ diameter\ [m]$

The value of C_c is calculated differently for situations with incomplete and complete projection

Complete projection condition:

$$C_c = \frac{e^{\pm 2K\mu(H/B_c)} - 1}{\pm 2K\mu}$$

Equation 3.2: Loading coefficient for complete projection

Incomplete projection condition:

$$C_c = \frac{e^{\pm 2K\mu(H_e/B_c)} - 1}{\pm 2K\mu} + (\frac{H}{B_c} - \frac{H_e}{B_c})e^{\pm 2K\mu(H_e/B_c)}$$

Equation 3.3: Loading coefficient for complete projection

$$H_e = r_{sd} p B_c$$

Equation 3.4: Height to equal settlement

Where:
$H = Height\ to\ cover\ [m]$
$H_e = Height\ to\ equal\ settlement\ [m]$
$r_{sd} = Settlement\ ratio$
$p = Projection\ ratio$
$K = Rankine's\ constant$
$\mu = Coefficient\ of\ friction$

K and μ are determined by the friction of the soil and the product is usually treated as one coefficient. The ± signs in front of the $K\mu$ value describe the settling relation between the interior and the exterior prism. If there are complete projection conditions the sign is positive, exterior prism settles more than interior prism, which is the case of a rigid pipe. If the opposite happens there are complete trench conditions and the sign is negative (Trott and Young 1984), which occurs if the pipe is elastic. The value of $K\mu$ is 0.19 for complete projection condition and 0.13 for complete trench conditions. ***Table 3.3*** below gives the value of C_c for various values of H/B_c and $r_{sd}p$, and experience shows that the values of C_c from this table can be used with good results (Moser 2001).

Incomplete projection condition $K\mu$ = 0.19		**Incomplete trench condition $K\mu$ = 0.13**	
$r_{sd}p$	**Equation**	$r_{sd}p$	**Equation**
0	$C_c = H/B_c$	-0.1	$C_c=0.82(H/B_c)+0.05$
+0.1	$C_c=1.23(H/B_c)-0.02$	-0.3	$C_c=0.69(H/B_c)+0.11$
+0.3	$C_c=1.39(H/B_c)-0.05$	-0.5	$C_c=0.61(H/B_c)+0.20$
+0.5	$C_c=1.50(H/B_c)-0.07$	-0.7	$C_c=0.55(H/B_c)+0.25$
+0.7	$C_c=1.59\ (H/B_c)-0.09$	-1.0	$C_c=0.47(H/B_c)+0.40$
+1.0	$C_c=1.69(H/B_c)-0.12$	-2.0	$C_c=0.30(H/B_c)+0.91$

Complete projection condition $K\mu$ = 0.19	**Incomplete trench condition $K\mu$ = 0.13**
$C_c = \dfrac{e^{\pm 2K\mu(H/B_c)} - 1}{2K\mu}$	$C_c = \dfrac{1 - e^{-2K\mu(H/B_c)}}{2K\mu}$

Table 3.3: Values of positive projection fill load coefficient C_c (Trott 1984)

The determination of $K\mu$ *is* difficult for different types of soils and represents a problem by applying the method explained above (Sægrov 1992).

3.3.1.2 Norwegian guideline

As mentioned above the Norwegian guideline for calculating fill load is based on the Marston/Embankment condition method. The guideline calculates the load with or without contributing support from the filling on both sides of the pipe.

If installation manuals are followed and the trench installation work is done properly, the difference in settlement in the exterior and the interior prism is small and the effect of the positive projection is minimized. The load on the pipe is in this case smaller than if the installation work is poor and the settlement in the sidefilling is significant. Additionally is the moment in the pipe wall significantly higher if the support from the side filling is poor (see ***Figure 3.9***). The Norwegian guideline distinguishes between these two situations and calculates the two different load situations as follows (Sægrov 1992).

With side support, compressed sidefillings:

$$W_c = \gamma B_c^2 (1.131 \frac{H}{B_c} - 0.508)$$

Equation 3.5: Fill load with side support

Without side support, poor or not compacted sidefillings:

$$W_c = \gamma B_c^2 (1.616 \frac{H}{B_c} - 0.266)$$

Equation 3.6: Fill load with out side support

The formulas above are based on the Marston/Sprangler theory, but parameters for Norwegian conditions have been found through experimenting.

Parameters:		**Value:**
Relativ height to equal settlement:	H_e/B_c	0.8
Friction:	$K\mu$	0.5
Side support factor:	K	0.485

Table 3.4: Values of parameters for calculating fill load by Aadnesen (1973)

As mentioned in chapter 3.3.1.1 the friction parameters K and μ are difficult to determine, but in 1990 Vaslestad developed a model, which solved this problem (Sægrov 1992). The parameter S_{vn} replaced Kμ. S_{vn} is determined based on general frictional properties of the soil. His adjustment of the two formulas above is as follows.

With side support, compressed sidefillings:

$$W_c = \gamma B_c^2 (1.157 \frac{H}{B_c} - 0.477)$$

Equation 3.7: Fill load with side support

Without side support, poor or not compressed sidefillings:

$$W_c = \gamma B_c^2 (1.642 \frac{H}{B_c} - 0.690)$$

Equation 3.8: Fill load with out side support

Parameters:		**Value:**
Relative height to equal settlement:	H_e / B_c	2.0
Friction:	S_{vn}	0.124
Side support factor:	K	0.485

Table 3.5: Values of parameters for calculating fill load by Vaslestad (1992)

3.3.2 Traffic load theory

Traffic and wheel loads from highways and railways are based on *the Boussinesq solution* (Moser 2001). Boussinesq calculated stress distribution in a semi-infinite elastic medium due to a concentrated load at its surface. Originally the solution assumes a homogenous, elastic and isotropic medium, which is not directly applicable for soil, but if it is properly applied experiments shows that the Boussinesq solution gives fairly good results.

Figure 3.7 below shows a combination of a wheel load, H-20 highway load and the fill load of the soil. H-20 loading is simulating the weight from a 20-ton truck rolling over a buried pipe and as we can see the load from the truck is decreasing with increasing cover height. When passing a cover height of 5 ft. (~1.5 m) the fill load is dominating the total load on the pipe. This means that pipes affected by wheel load are those with 5 ft. of cover or less (Moser 2001).

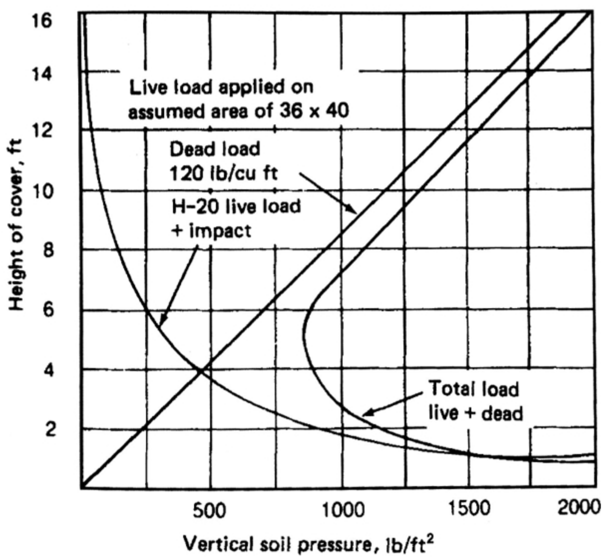


Figure 3.7: Combined effect of H-20 wheel load and fill load **(Moser 2001)**

The Norwegian traffic load calculation method is based on a set of wheel loads with a combined force of 260 kN (two concentrated loads, 2 meters apart). The wheel load has a dynamic addition and the net concentrated load for each wheel is 75 kN. Generally the dynamic addition is greater when the soil consists of silt, sand or gravel, rather when it is grained (clay) or coarse (rocks) (Sægrov 1992).

The traffic load on a pipe is calculated as follows:

$$W_t = 75C_t\psi(1 + k)$$

Equation 3.9: Wheel load

Where:
$W_t = Traffic\ load\ on\ the\ pipe\ [kN/m]$
$C_t = Loading\ coefficient$
$H = Height\ to\ cover\ \ [m]$
$B_{\square} = Pipe\ diameter\ [m]$
$k = concentration\ coefficient$
$\psi = Impact\ coefficient$

One important difference from the Boussinesq solution to the calculation of traffic loads on pipes is that a rigid pipe disrupts the soil stress equilibrium. This has an effect of increasing the traffic load on the pipe and is taken into account by applying the concentration coefficient, k.

$$k = \frac{0.7}{4.5}\frac{H}{B_c}\ ;\ for\ \frac{H}{B_c} < 4.5$$

Equation 3.10: Concentration coefficient (<4.5)

$$k = 0.7\ ;\ for\ \frac{H}{B_c} \geq 4.5$$

Equation 3.11: Concentration coefficient (≥4.5)

The impact coefficient Ψ is applied because an uneven cover surface produces impact forces on the pipe that is significantly greater than a static traffic load.

$$\psi = 1.3 - \frac{H}{7}$$

Equation 3.12: Impact coefficient

The loading coefficient C_t is a function of the pipe length (L) and diameter (B_c), and the cover height (H). ***Figure 3.8*** below shows C_t values with a standard pipe length of 1 meter (Sægrov 1992).

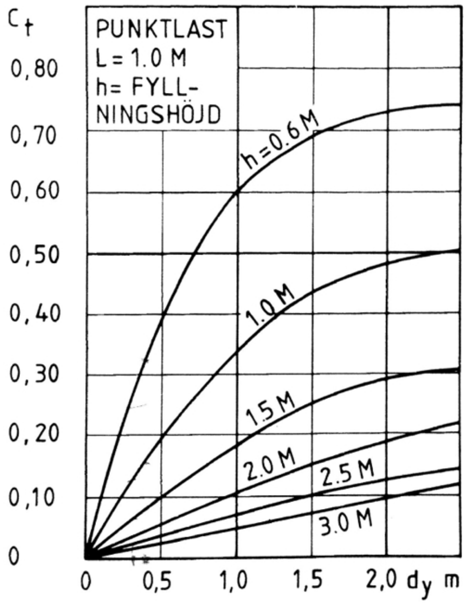


Figure 3.8: Load coefficients for 1 meter pipe length **(Ekbäck and Hedman 1983)**

3.3.3 Other loads

In addition to the two most important loads on pipes; fill and wheel loads, in some cases loads such as cover load and the water pressure from inside of the pipe also need to be considered.

Cover load:

Cover load is the load of the material covering the soil and trench fill material, for instance concrete or tarmac road cover. When the cover height is decreasing the load from the cover influence a buried pipe and needs to be considered. The cover load is calculated as a part of the embankment theory, and the cover load is transferred to a cover height equivalent:

$$h_e = {}^{q}/_{\gamma}$$

Equation 3.13: Cover load

The cover load (kN/m^2) is divided by the density of the cover material (kN/m^3) (Sægrov 1992).

Water pressure from inside of the pipe:

Industrial wastewater can be hot and high temperatures generate stress as the pipe expansion is limited by the surrounding soil. For straight pipelines the longitudinal stress is:

$$S_L = E\alpha(T_2 - T_1) - \nu S_h$$

Equation 3.14: Water pressure

The modulus of elasticity (N/mm^2), coefficient of thermal expansion (1/ºC) in the concrete and the temperature increase (ºC) are the source of the stress generation. Also a hoop stress due to fluid pressure inside the pipe is deducted (Moser 2001).

Pressure from the fluid inside a partial or completely filled pipe is also generating stress in the pipe wall. The formula for calculating load by the fluid is as follows:

$$W_f = 5\, B_c^2$$

Equation 3.15: Load by fluid pressure

For pipe diameters $B_c < 1500$ mm the water level inside the pipe has no significance (Ekbäck and Hedman 1983).

3.3.4 Calculation stress and necessary wall thickness

The total load on a buried concrete pipe generates stress in the pipe wall. The correct formula to calculate stress in the pipe wall is very complex because of the variation along the pipe length, but a simplified formula is developed. The formula assumes that the total load is evenly distributed respectively along the upper and the lower quadrant of the pipe and results in equal design moment at both top and bottom:

$$M = 0.0844\, WB_c$$

Equation 3.16: Moment at pipe top and bottom

The load generates stress, but because of the wall curvature the strain and stress are different in the inner and outer edge of the pipe wall. At the top and bottom the strain on the inner edge (σ_i) is higher than the stress on the outer edge (σ_o). The strain and stress are respectively calculated as follows:

$$\sigma_i = \frac{6M}{s^2}\frac{1 - s/3d}{1 - s/d}$$

Equation 3.17: Strain in the inner edge

$$\sigma_o = \frac{6M}{s^2}\frac{1 + s/3d}{1 + s/d}$$

Equation 3.18: Stress in the outer edge

As a simplification the difference between the two formulas above is neglected and the stress in the pipe wall is calculated as a beam with an evenly distributed uniform load:

$$\sigma = \frac{6M}{s^2}$$

Equation 3.19: Mean pipe wall stress/strain

Where:
$\sigma = Pipe\ wall\ stress/strain$
M = Moment
s = Wall thickness
d = Pipe diameter
(Ekbäck and Hedman 1983)

When testing concrete pipes a concentrated longitudinal point load on top of the pipe is used. To convert the test load to a evenly distributed load the following formula is used:

$$W = \frac{0.159 - 0.125\theta}{0.0844} W_{test}$$

Equation 3.20: Test load calculations

Where:
$W_{test} = Test\ load$
$2\theta = Angle\ of\ the\ test\ load\ distribution$
(Ekbäck and Hedman 1983)

The load situation from one trench to another is different and this is often due to the quality of the installation work (experience with Norwegian installation work is described in chapter 3.6). If the work is of high quality and the pipe has good support from bedding and side filling, the pipe stress is a lot smaller than if the construction work is poorly done.
Sprangler and Handy (Sægrov 1992) developed three scenarios of load situations:

1. Good side and bedding support (***Figure 3.9 a***)
2. Good bedding support, but low side support (***Figure 3.9 b***)
3. No significant support (***Figure 3.9 c***)

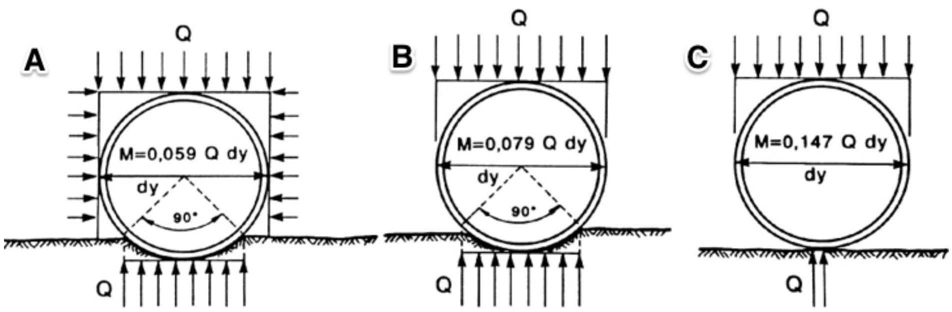


Figure 3.9: Three load situations according to the degree of support **(Sægrov 1992)**

Figure 3.9 shows three different load situations and corresponding formula for calculating maximal moment in top and bottom of the pipe generated by the load applying the pipe. Assuming the same size of load in all three cases and the smallest moment will be generated in case a. This is because the filling on both sides of the pipe is supporting it and you have the most optimal force distribution in the pipe wall. Case b illustrates a typical situation of poor installation work and the support of the filling on both sides of the pipe is neglected and the force distribution is not optimal. If we assume the same load size and outer pipe diameter will the moment in case b be 25% higher than in case a. Case c is illustrating a situation where the pipe is embedded on a rigid surface, for instance on bedrock (Sægrov 1992).

$$M = 0.059\, WB_c$$

Equation 3.21: Case a moment at pipe top and bottom

$$M = 0.079\, WB_c$$

Equation 3.22: Case b moment at pipe top and bottom

$$M = 0.147\, WB_c$$

Equation 3.23: Case c moment at pipe top and bottom

3.4 Factors Influencing the Mechanical Forces on Concrete Pipes

Only under ideal conditions, the distributed load is of the same magnitude along the whole length of a sewer pipeline. Nonuniform bedding, erosion, differential settlement, ground expansion like frost heave and dynamic forces from traffic and even earthquake may influence pipe and trench conditions creating complex load situations. Earthquakes are not very relevant for Norwegian conditions, but the other ones are causing several pipe failures every year. Typical of these problems, opposed to the mechanical loads (3.3), is that they are localized forces, dynamic and highly variable, and therefore very difficult to calculate. ***Figure 3.10*** shows that nonuniform bedding is one of the causes to axial bending stress generation and beam action in a pipe. This is further discussed below together with other mechanical forces.

3.4.1 Nonuniform bedding and erosion

Nonuniform bedding can be caused by uneven compaction of the bed fill material. It may also be a result of erosion of the soil caused by a leaking sewer pipe or flowing water in the trench. Unstable foundation material such as soft clay can also cause nonuniform bedding. All these effects may result in uneven settlements.

Figure 3.10 a shows that this backfilling method will cause depressions and settlements if we have unstable clay bedding. By backfilling in layers as shown in ***Figure 3.10 b*** the bedding will settle more even.

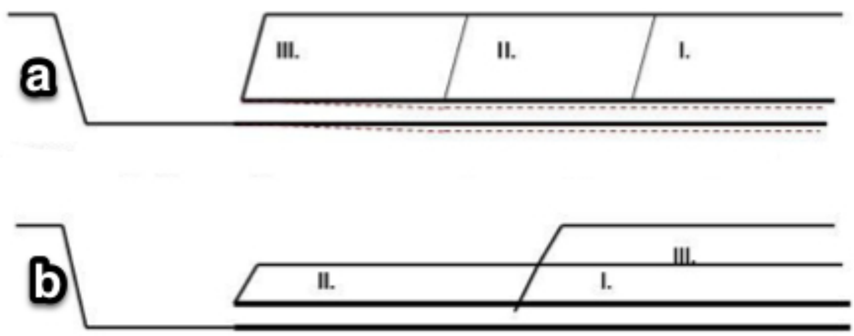


Figure 3.10: Installation of pipes in unstable foundation materials **(Sægrov 2012)**

The bending moment induced by the above mentioned processes would cause a ring deflection in a long horizontal pipeline. High concentrated stress and strain forces can affect local areas on top or bottom of the pipe, and a result can be circumferential cracks or even fractures (see ***Figure 3.5***).

To reduce the risk of rupture and cracking, flexible joints can enhance the pipes ability to withstand bending moments and forces generated. Proper installation work and good engineering can reduce and even eliminate axial bending forces, and reduce the risk of failure. ***Figure 3.11*** below shows bending moments generated by three different scenarios in pipelines with flexible joints.

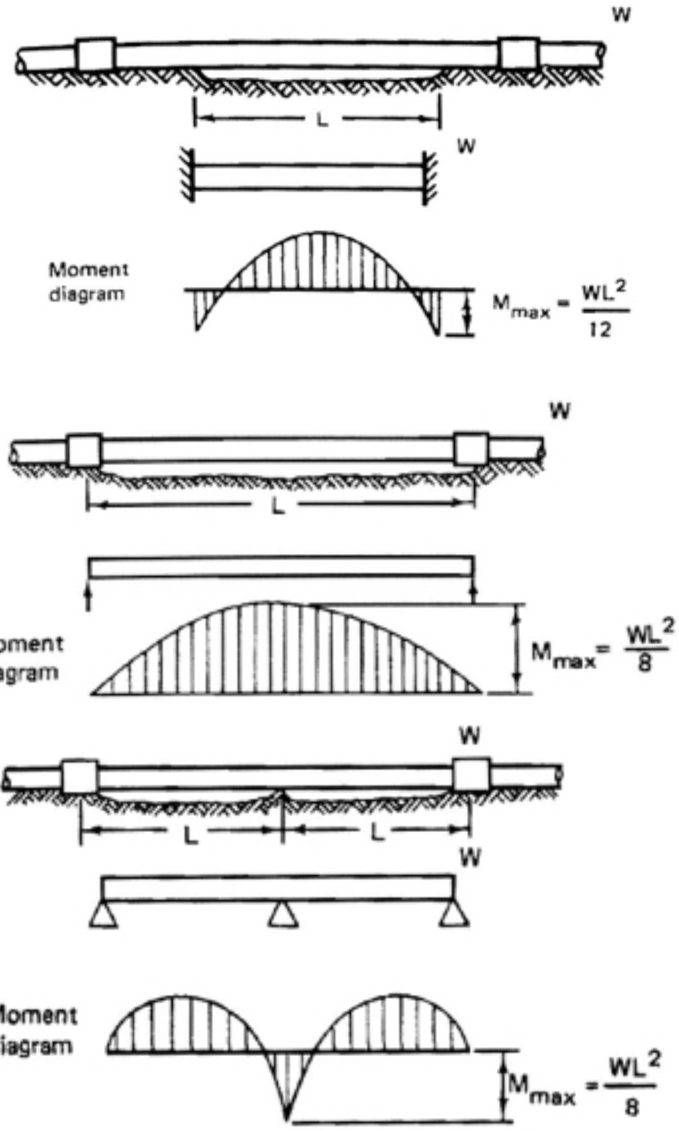


Figure 3.11: Bending moment in pipelines caused by nonunifrom bedding **(Moser 2001)**

3.4.2 Differential settlements

Shear forces and high bending moments can be induced when structures like manholes, which is a rigid connection to the pipe, move vertical or horizontal with respect to each other. These problems can occur due to the larger weight of the manhole, or when the ground water table changes due to dry seasons etc. This causes differential settlements between manholes and pips and inducing bending moments in the pipes as shown above in ***Figure 3.11***. There will also be shear forces in the connection between pipe and manhole or other rigid connected structures.

The way to minimize these effects is to secure good engineering design, flexible connections, proper installation work and use of backfill material of proper quality. In general the quantity of bending moment and shear forces are very hard to measure and evaluate.

3.4.3 Frost heave and ground expansion

The seasonal change in soil moisture and ground water variations may result in upward expansion or settlement of the soils, which will transfer forces on a buried pipe. A typical phenomenon in cold climate is that the soil moisture freezes to a certain depth during the winter season. As the frost penetrates the ground the small volume of water freezes, and has a drying effect on the soil. This leads to a capillary suction of ground water from the saturated zone below and the water freezes as it reaches the frost zone. This effect continues until capillary forces reach equilibrium. Because ice has a higher displacement than water the soil will expand (Moser 2001).

The pressure on a buried pipe will increase because of the expanding soil. It is shown that the load can be nearly doubled on a rigid pipe. This result was presented in a paper by W.H. Smith (AWWA Journal, Dec. 1975) (Moser 2001). In their test they used a rigid pipe cut horizontally in half and the deflection was measured. In the test the pipe side supports were little to nothing, which leads to a great magnifying of the vertical load.

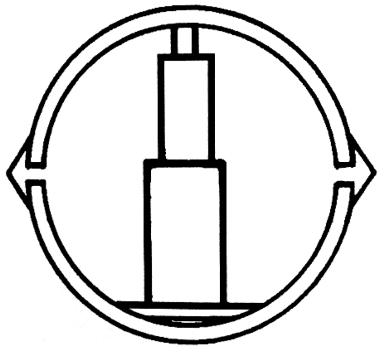


Figure 3.12: Test setup for increased load due to frost **(Moser 2001)**

Two different problems with pipes have been observed related to frost heave in Norway.
The first problem appears as explained above when the soil above and around the pipe expands and creates an increased load pushing down on the pipe. Longitudinal cracks evolve along the inside top of the pipe or circumferential cracks appear around the bottom of the pipe if the side and sole support are not sufficient. At points where the pipe is rigid attached, such as manhole connections, increased shear-forces occur and the pipe can break during the winter season.

When soil thaws, during spring, pipe breakage may occur. This can happen if the trench backfill is not frozen but the soil surrounding the trench freezes. The frozen soil around the trench expands pushing the pipe up, and during thawing the soil retracts and the pipe settles and rigid connections may break. To avoid these problems flexible connections in combination with high drainable backfill material are good measures.

3.5 Geophysical Methods for Trench Inspection

Geophysical technology, for instance shallow seismic and ground penetrating radar (GPR) has for some time been used to monitor and investigate the maintenance level of road and highway foundations in urban areas. In this chapter I will briefly explain the theory and use of these technics, and look into the possibilities of adapting this into pipe and trench inspection procedures.

The pipe inspection methods used today focus on monitoring the situation inside a buried pipeline, and opposed to this geophysical technology is offering the possibility of analyzing the situation in the trench, i.e. outside the pipe. In chapter 0 it is shown that pore bedding support can increase the internal stresses and strains in a rigid pipeline. With geophysical methods is it possible to investigate the bedding support in terms of the type of backfill material, compaction level and detect voids. An example of this use could be additional investigation of the effects causing circumferential cracks detected by pipe inspection. If a geophysical investigation of the pipe trench reviles voids and poor bedding support, these cracks may possibly grow and lead to a collapse prior to the end-of-life prediction of the pipeline. Maintenance or replacement of this pipe should then be prioritized in comparison to a situation where no voids are reviled.

3.5.1 Ground penetrating radar (GPR)

GPR is a radar system and using electromagnetic waves to determine the direction and range of objects.

The GPR`s ability to distinguish between obstacles is dependent on the range resolution, which is determined by the signal bandwidth (BW). Grater BW gives a better resolution, and the shortest time interval that a GPR can resolve is the invers BW ($\tau = 1/BW$). This gives the range resolution ($c\tau/2 = c/2BW$, c is the signal velocity: 30 cm/ns in air). A method for generating a high BW is to use short pulses of high power or sweep through a wide frequency range over time.

GPR is a technology used for mapping soil layering, structures or cracks in road cover and concrete in bridges. This requires a very high resolution, down to millimeter accuracy. The BW of a GPR is in the range of 200 MHz to several GHz, which gives a time resolution of 5 ns to under 0.5 ns. In comparison a typical navigation radar has a max BW of approximately 15 MHz, which gives a time resolution of 70 ns.

A problem with such high frequency is that the penetration of waves decreases rapidly. This means that the center frequency needs to be sufficiently low to be able to reach the required depth.

Moving the GPR along the surface while it transmits waves down through the ground will detect soil layers or objects in the ground. Objects will generate echo from the transmitted waves, which is received by the GPR antenna. As the GPR approaches the object the echo is getting smaller until the GPR is on top of the object and the range is at it shortest. The range increases again as it moves away. This results in a 2D image where the objects echo path is marked as a hyperbola. If we know the signal wave speed it is possible to focus the picture and data are summed along the hyperbola path. This is called migration. ***Figure 3.13*** shows that the hyperbolas are removed and the image is sharpened.

The GPR 2D image slices through the ground. To tell the shape of the objects the x, y and z planes must be compared (3D-Radar 2009).

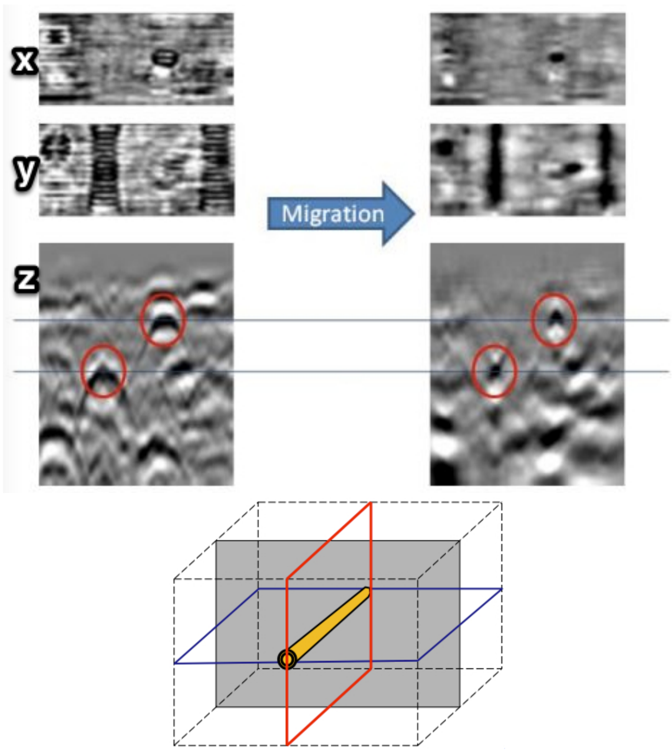


Figure 3.13: Migration of receiving echo. Identifying objects trough 3D imaging **(3D-Radar 2009)**

3.5.2 Detection of voids

Identifying voids in the backfill material around the pipe is important as this may be the main source of cracks evolving in the pipe wall. This chapter illustrates the use of GPR and ultrasonic seismic to detect voids. Survey done by Nigel Cassidy at Keele University in 2011 (Cassidy 2011).

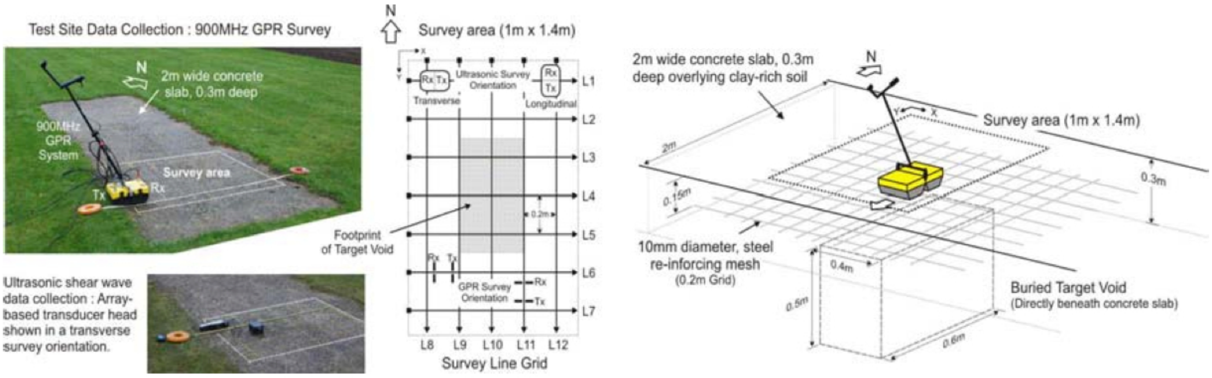


Figure 3.14: Survey area layout and method for data collection **(Cassidy 2011)**

Figure 3.14 shows how the survey was done. The goal was to detect a target void buried under a reinforced concrete slab. The slab geometry was 2 m x 6 m and 0.3 m thick, which is similar to a road surface. A air filled target void; 0.4 m wide, 0.6 m long and 0.5 m deep is buried in a uniform dense, low permeability boulder clay soil with a conductivity of 10-50 mS/m.

Because of the known position of the target void under the slab, a 1 m x 1.4 m survey area was laid out equidistance from the mid-point. The distance between the data collection lines was 0.2 m and the survey were collected in an orthogonal grid aligned to the edges of the survey area. The survey was done with a 900 MHz GPR system and an ultrasonic shear wave data collection.

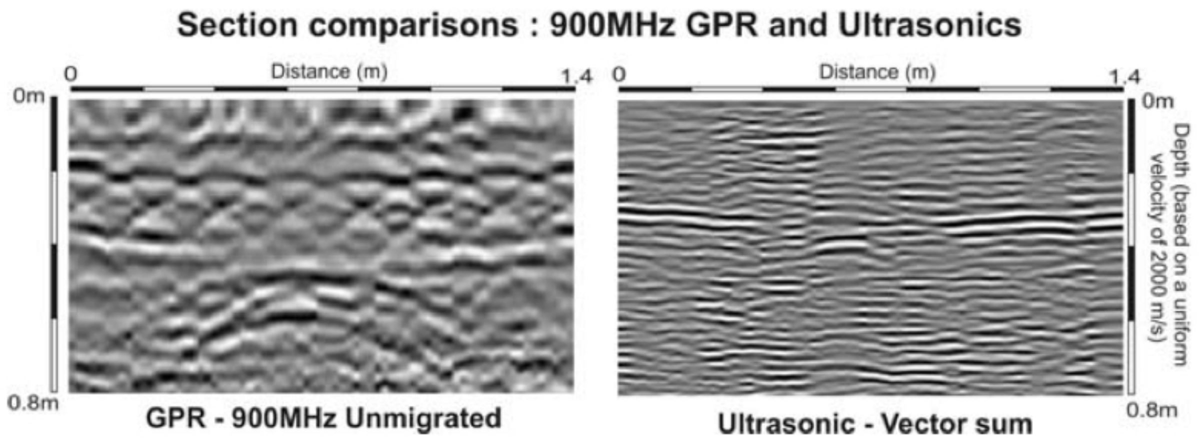


Figure 3.15: 2D Section. Comparison of GPR and ultrasonic data collection along the L10 survey line **(Cassidy 2011)**

Figure 3.15 shows the 2D image along the L10, center survey line produced by the GPR (left) and the Ultrasonic method (right).

The GPR image shows a strong coherent uniform diffraction hyperbola, which is the echo of the steel reinforcing meshes. A strong and broad coherent hyperbolic-shaped reflection shows the top and sides of the target void. The Ultrasonic data has a shorter wavelength and tighter pulse width, which gives an image with higher resolution compared to the GPR. The top of the target void is appearing where the reflection of the concrete slab is lost.

The GPR image has a lower resolution but tells us also more about the situation under the concrete slab in this shallow survey.

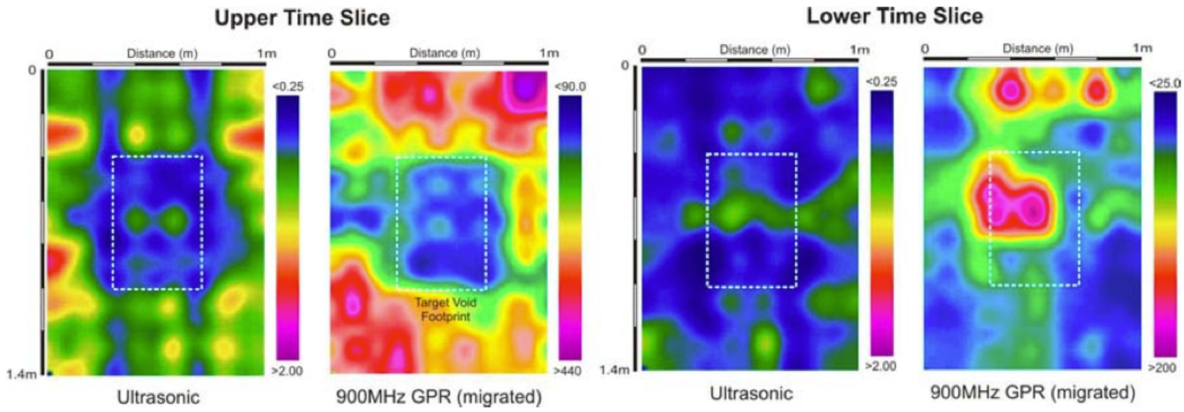


Figure 3.16: 3D upper and lower time slices. Comparison of GPR and Ultrasonic time slices at 0.3-0.36 m and 0.36-0.42 m **(Cassidy 2011)**

Figure 3.16 shows two comparable situations. The upper time slice is supposed to highlight the reflection from the base of the slab and the top of the target void (0.3-0.36 m). The lower time slice is supposed to highlight the corners of the void (0.36-0.42 m). The colors indicate the amplitude signal of the reflection waves. The blue color indicates smaller absolute amplitude than purple.

For the upper time slice both data sets are showing a quite clear reflection of top target void defined by a rectangular low-amplitude zone, which also correlates well with the target void position. Especially for the GPR there is a sharp contrast between the low-amplitude zone and the high-amplitude zone close to the void boundaries. The reason for this is the return of strong coherent energy from the base of the slab and loss of energy over the void.

For the lower time slice there is no strong contrast of amplitude zones for the ultrasonic measurements, but the for the GPR is it possible to see a high amplitude zone in the center of the survey area. This is the energy reflected from the base of the void and is most likely interfered by late time signals from the sides and edges of the void (Cassidy 2011).

3.6 Quality of the Norwegian Wastewater System

The general experience of the Norwegian storm and wastewater system is that the quality of the concrete pipes installation work varies with the age.

Research on quality and residual strength of concrete wastewater pipelines in Norway (Sægrov 1992) show that the measured mean value of tensile crush load is decreasing with age, see ***Table 3.6***.

Crush load tensile strength (σ_{max})				
	1920-44	1945-59	1960-69	1970-79
Number of tested pipes	9	10	10	7
Mean value [MPa]	7.32	9.35	11.78	12.90
Standard deviation	3.25	2.52	2.43	2.70

Table 3.6: Mean brake load tensile strength measured by testing of used concrete pipes **(Sægrov 1992)**

Later statements (Sægrov 2012) elaborates how forces and loads influence the pipes due to the age.

- Installation work of wastewater pipes older than the 1940s where dug and built by hand. Typical for these are narrow and shallow trenches and backfill masses of the same kind as the soil in the surrounding area.
- During the period of 1950-1970, installation work were more commonly done by machines and excavators. Typical for these are deeper trenches and often poor installation work. This results in not beneficial load situation; high fill loads and low side support (see ***Figure 3.9 b***).
- The period after 1970 is characterised by gradually better installation work and consequently a more beneficial load situation (see ***Figure 3.9 a***).

Demands and requirements to the production of concrete pipes have increased gradually, and the quality of the pipes to day is very good. ***Figure 3.17*** shows the increase of wall thickness and crushing load demands from 1909 – 2003, for a DN300 concrete pipe (Standard Norge 2003) (Sægrov 1992).

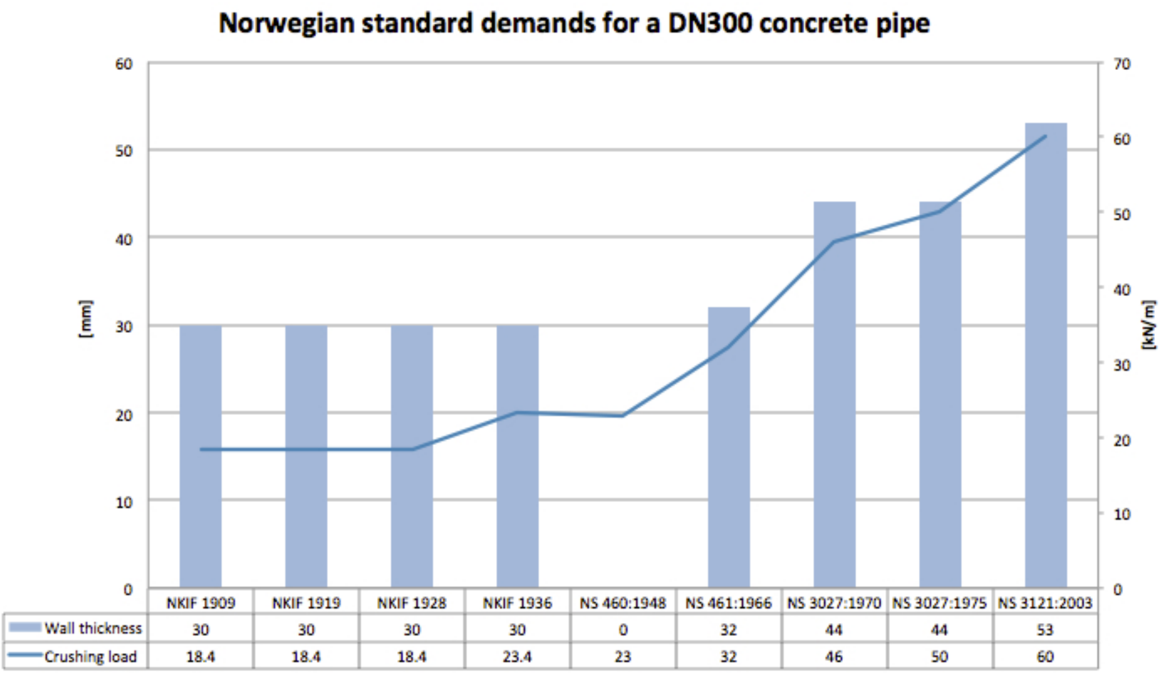


Figure 3.17: Norwegian standard demands for a DN300 concrete pipe, from 1909 – 2003.

The demands for production of concrete pipes in 2013 is described in NS 3121:2003. It consist of general demands for the production materials (cement, aggregates, reinforcement steel etc.), the concrete quality, water-cement ratio, chlorine content, max water absorption. There are also defined demands for the design and strength of the pipe elements, such as surface quality, design of bends and connections, maximum cross sections deviation, water leakage and minimum crushing load (Standard Norge 2003).

Table 3.7 below is taken from NS 3121:2003 and displays the minimum crushing load, wall thickness and maximum cover height for all standardized pipe sizes (Standard Norge 2003).

DN	Wall thickness bell pipe [mm]	Wall thickness rebated pipe [mm]	Min. crushing load [kN/m]	Max. cover height [m]
100	29		60	15
125	30		56	12
150	33		56	10
200	37		60	8
250	45		60	7
300	53		60	6
300		90	135	12
400	63		65	5
400		85	95	7
500		90	90	6
600		94	75	4

Table 3.7: Standard demands for design of concrete pipes NS 3121:2003 **(Standard Norge 2003)**

4 Surface Damages on Concrete Pipes

This chapter is focusing the most common processes that cause surface damages on concrete storm and wastewater pipes. Stormwater and wastewater feeding into the system may start mechanical and/or biochemical and chemical processes resulting in corrosion and erosion inside the pipe.

Generally storm and wastewater from housing areas and roads/highways have normal pH levels and corrosion on concrete pipes does not appear. If we have biological digestion in the wastewater, chemical processes may start and cause concrete deterioration and surface damages due to very low pH-values as a result of acid production.

The alkalinity of Norwegian wastewater is normally about 1-5 mmol/l (Sægrov 1992). From the level of alkalinity the concentration of dissolved CO_2 can be calculated by using the carbon equilibrium of water. By knowing the CO_2 concentration the pH-level can be determined, which can tell us how acidic the wastewater is. However, this will not tell anything about how much of the CO_2 is available for chemical digestion of the concrete. ***Table 4.1*** below shows the correlation between CO_2 concentration and pH-level.

pH	**CO_2-concentration (mg/l)**
< 6.3	132
6.3	66
7	26
8	2.6

Table 4.1: Correlation between CO_2 concentration and pH-level **(Sægrov 1992)**

Focus in Norway has been to reduce or prohibit untreated industrial and agricultural wastewater to come into the wastewater system, and this has reduced the risk of surface damages in the concrete wastewater pipes.

Table 4.2 below shows the parameters affecting concrete corrosion and at which concentrations it occurs.

Parameter	Measurements of Norwegian wastewater		Corrosion threshold level/concentration
	Value	Std. deviation	
pH	7.13	± 0.41	5.5
Hardness			24 ^{0}dH
CO_2			20 mg/l
NH_4^+	50.4 mg/l	± 11.8	30 mg/l
COD	470 mg/l	± 8.8	100 mg/l
SO_4^-	25.6 mg/l		1 mg/l

Table 4.2: Parameters affecting concrete corrosion **(Sægrov 1992)**

4.1 Aggressive Water and Chemical Corrosion

Normally concrete pipes are not supposed to be affected by aggressive water at those levels that occurs in Norwegian wastewaters. However, due to poor concrete quality in old pipes, acidic wastewater can cause serious surface damage when it has the right combination (Sægrov 1992).

4.1.1 Acid water

Calciumhydroxide, $Ca(OH)_2$ is the most easily dissolved concrete substance, and in combination with aggressive water corrosion or wash out of $Ca(OH)_2$ this is a common problem. The dissolved calciumhydroxide in the pore water is in steady state equilibrium with the calciumhydroxide in the concrete during normal circumstances.

$$Ca(OH)_2 \leftrightarrow Ca^{2+} + 2OH^-$$

Equation 4.1: $Ca(OH)_2$ dissolution

If the wastewater has low hardness, low values of calcium, or is acidic (low pH), the equilibrium in the wastewater is driven to the right in the equation above. This may result in more dissolution of calciumhydroxide from the concrete. If the pore water is in contact with the water on the surface of the concrete pipe wall, this process will continue and the calcium is washed out.

Porous concrete, which has a high level of surface area will increase this reaction and result in a higher deterioration speed of the concrete pipe (Sægrov 1992).

4.1.2 Carbondioxide

Excess amount of dissolved carbondioxide, CO_2 in the wastewater leads to corrosion by carbonic acid.

This type of corrosion is a two-step reaction.

1. First calciumhydroxide in the concrete will react with dissolved carbonhydroxide and water (see the first equation below). The product of the reactants is calciumcarbonate, $CaCO_3$ and water. $CaCO_3$, which is a low soluble salt, will precipitate if the circumstances allow it. The reaction decreases the pH in the water closed to the concrete wall from around 12 down to 9 or 8 and this leads to corrosion on the steel

reinforcement in the concrete pipe. This reaction is not influencing concrete pipes without reinforcement steel.

2. The second step will occur if excess carbondioxide is dissolved in the wastewater. Calciumcarbonate reacts with dissolved carbondioxide and forms calciumbicarbonate, $Ca(HCO_3)_2$. The amount of dissolved calcium, Ca^{2+} is the driving force in this reaction, and an increasing amount of dissolved Ca^{2+} implies an increasing amount of carbonic acid, which is in steady state equilibrium with calciumcarbonate (Sægrov 1992).

Below the two steps described above are shown as chemical reactions:

i. $$Ca(OH)_2 + CO_2 + H_2O \leftrightarrow CaCO_3 + H_2O$$

ii. $$CaCO_3 + CO_2 + H_2O \leftrightarrow Ca(HCO_3)_2 + H_2O$$

Equation 4.2: Formation of $Ca(HCO_3)_2$

4.1.3 Ion exchange

Corrosion by ion exchange on concrete pipes may occur in certain environments. Anions from substances in the wastewater are exchanged by the Hydroxide ions, OH^- from calciumhydroxide; so that calcium becomes easy soluble and the concrete will be erodible.

Three examples of chemical compounds that dissolve calciumhydroxide are:

- Sulfuric acid, H_2SO_4
- Ammonium chloride, NH_4Cl
- Sodium nitrate, $NaNO_3$

Sulfuric acid reacts with calciumhydroxide and OH^- is exchanged with SO_4^{2-} to form the easy soluble salt, calciumsulfate, $CaSO_4$. Calcium sulfate is carried away with the water in the pipe and the reaction continues, the equilibrium in the reaction below is transferred to the right. The production of sulfuric acid in wastewater will be described in chapter 4.2.2

$$Ca(OH)_2 + H_2SO_4 \leftrightarrow CaSO_4 + H_2O$$

Equation 4.3: Dissolution of $Ca(OH)_2$ by H_2SO_4

Ammonium chloride is an insoluble salt, but when reacting with calciumhydroxide the reaction will produce soluble compounds. This reaction can be hazardous because it produces ammoniumhydroxide, NH_4OH, which is a toxic gas. Calciumchloride, $CaCl_2$ is also a product of the reaction and is washed out with the waterflow in the pipe. The chemical reaction below shows the process:

$$Ca(OH)_2 + NH_4Cl \leftrightarrow CaCl_2 + 2NH_2OH$$

Equation 4.4: Dissolution of $Ca(OH)_2$ by NH_4Cl

Nitrates and nitrites are the third example of anion groups that can exchange the hydroxide and make calcium dissolve easily. Calciumhydroxide reacts with sodiumnitrate and produces sodiumhydroxide and calciumnitrate, which is an easy soluble salt (Sægrov 1992).

$$Ca(OH)_2 + 2NaNO_3 \leftrightarrow Ca(NO_3)_2 + 2NaOH$$

Equation 4.5: Dissolution of $Ca(OH)_2$ by $NaNO_3$

4.1.4 Calcium products

When calciumhydroxide reacts with certain chemical compounds, the volume of the product is larger than the total volume of the reactants. This will lead to expansion of the substances in the concrete pore space and cracking and erosion of the pipe occurs.

One chemical with this effect is aluminumsulfate, $Al_2(SO_4)_3$. When reacting with calciumhydroxide it forms calciumsulfate, $CaSO_4$, which is either easy soluble or it can form gypsum. Gypsum has a high volume and will lead to cracking of the concrete.

$$3Ca(OH)_2 + Al_2(SO_4)_3 \leftrightarrow 3CaSO_4 + 2Al(OH)_3$$

Equation 4.6: $CaSO_4$production with $Al_2(SO_4)_3$

Calciumsulfate can react with calciumaluminate, C_3A, which is a substance in concrete, and form Ettringite. This leads to more cracking of the concrete. Ettringite is observed in concrete pipes of poor quality and high porosity (Sægrov 1992).

$$3CaSO_4 + C_3A\ x\ nH_2O \leftrightarrow C_3A\ x\ 3CaSO_4 x\ 32H_2O$$

Equation 4.7: $CaSO_4$production with C_3A

4.1.5 Corrosion on the outside of the pipe

The corrosion mechanisms on the outside of a concrete pipeline are exactly the same as the inside of the pipe. If the soil is acidic and contains a high concentration of available carbondioxide and low pH, the concrete will corrode in the same way as explained in the chapters above.

In a low pH soil the grade of corrosion will also depend on the level and gradient of the saturated groundwater zone and the soil permeability.

	Permeability (Darcy coeff, m/s)	**Expected Lifetime [years]**	
		Gradient of groundwater zone	
		I = 1/10	I = 1/20
Sand	10^{-4}	3.8	4.0
	10^{-5}	5.6	8.3
	10^{-6}	42	83
	10^{-7}	420	830
Silt	10^{-8}	4200	8300

Table 4.3: Lifetime (years) of concrete pipe by carbonic acid corrosion. Diameter = 500mm, Weight = 150kg/m **(Sægrov 1992)**

Table 4.3 gives a lifetime calculation for a concrete pipe in acidic soils, 100mg/l concentration of available carbondioxide. This shows that a more coarse soil and steeper gradients decrease the expected lifetime of the pipe a lot. A pipe must be protected from corrosion if the groundwater is this aggressive (Sægrov 1992).

4.2 Biochemical Corrosion and Sulfuric Acid Attack

Biochemical corrosion is a result of microbiological activity in the sewer pipe. The most common situations occur when bacteria's decompose nutrition's. Sulfate, nitrate and strong acid are the resulting products of the metabolism. Sulfate and nitrate are oxidized and organic carbon in the wastewater is used as a carbon source. Together with consumption of carbondioxide and absence of oxygen, sulfuric and nitric acids are produced. Organic acids can also be produced under these circumstances, but this is a much weaker acid.

Field investigations of concrete pipelines with biochemical corrosion shows that sulfuric acid attacks are the most important. Attacks by nitric or organic acid are rarely observed.

1. Biological process of Sulfuric acid

Sulfate Protein	⇒ (anaerobic)	Volatile sulfur compounds	⇒ (aerobic)	Elemental sulfur	⇒ (aerobic)	Sulfuric acid

2. Biological process of Nitric acid

Nitrate Protein Nutrients	⇒ (anaerobic)	Ammonia	⇒ (aerobic)	Nitric acid

3. Biological process of Organic acid

Cellulose Glucose Carbon dioxide Fat Protein	⇒ (anaerobic)	Organic acid

Table 4.4: Process of biological acid production **(Sægrov 1992)**

4.2.1 Nitric acid attacks

Nitric acid production in wastewater is a nitrification process, which means that ammonia, NH_4^+ is concerted to nitrate, NO_3^- by bacterial metabolism. The nitrate ion is then able to dissolve calciumhydroxide trough ion exchange, similar to the process described in chapter 4.1.3. The nitrification process is an aerobic digestion process, which take place under correct environmental conditions, pH and temperature, for the bacteria culture to live and grow.

Field investigations done in Norwegian sewer pipelines show a low concentration of nitrate compared to the measured concentration of ammonia (Sægrov 1992).

The chemical reaction below shows the dissolution of calciumhydroxide by nitric acid:

$$Ca(OH)_2 + 2HNO_3 \leftrightarrow Ca(NO_3)_2 + H_2O$$

Equation 4.8: Dissolution of $Ca(OH)_2$ by HNO_3

4.2.2 Sulfuric acid attacks

4.2.2.1 Principles of Sulfide and Sulfuric acid formation

For Sulfuric acid attacks in wastewater pipes to happen, the presents of Sulfide is required. Occasionally Sulfide is present in industrial wastewater or trough infiltration by Sulfide containing groundwater. However, the most common source of Sulfide is microbiological activity in the wastewater as mentioned above. Both Sulfur containing organic matter and inorganic sulfur compounds are reduced by bacterial metabolism. When sulfate ion, SO_4^{2-} and organic matter are present and oxygen is absent, sulfate will be reduced to sulfide and organic matter oxidized by the bacteria species: *Desulfovibrio desulfuricans* (U.S. Envitonmental Protection Agency 1974).

The chemical reaction can be written as follows:

$$SO_4^{2-} + organic\ matter \xrightarrow{bacteria} S^{2-} + alcohol + organic\ acid$$

Equation 4.9: Bacterial reduction of sulfate

In a partly filled pipe the wastewater is exposed to the air volume above the water level and the rate of gas transmission from air to water creates an aerobic environment. This means that Sulfate reduction is not happening in a sewer pipe of regular gravity. However, on the submerged and slime covered pipewall the oxygen concentration can decrease to an anaerobic level. This slime cover is hereby referred to as biofilm.

Figure 4.1 shows the biofilm covering the pipewall. As we can see the wall is divided in three zones:

1. Aerobic zone, where aerobic bacteria consume oxygen from the stream, which defuses into the zone. This leads to a limited outer aerobic zone.
2. Anaerobic Sulfide producing zone, where anaerobic bacteria reduce sulfate to sulfide. Because of the limited diffusion capacity of sulfate and organic nutrients this zone is also limited.
3. Inert anaerobic zone, which is largely inactive because of the lack of nutrient supply.

The oxygen supply to the layer determines the extent of the aerobic zone and again the production of sulfide. The oxygen supply and depth of the penetration into the biofilm is determined by the oxygen concentration, temperature and concentration of organic digestible matter in the wastewater. Aerobic bacteria are present in the anaerobic zone and if the oxygen penetrates deeper will these bacteria become active and oxidize the organic nutrients, which leads to a decrease in sulfide production. The anaerobic bacteria resume the activity when the oxygen disappears and sulfide production increases. Since the concentration of organic nutrients, oxygen and temperature in wastewater are varying the production of sulfide is also varying.

The sulfide produced in the anaerobic zone diffuses out of the zone and into the aerobic zone. If the oxygen concentration is sufficient sulfide will oxidize immediately and there is no noticeable sulfide accumulation in the water stream (U.S. Envitonmental Protection Agency 1974).

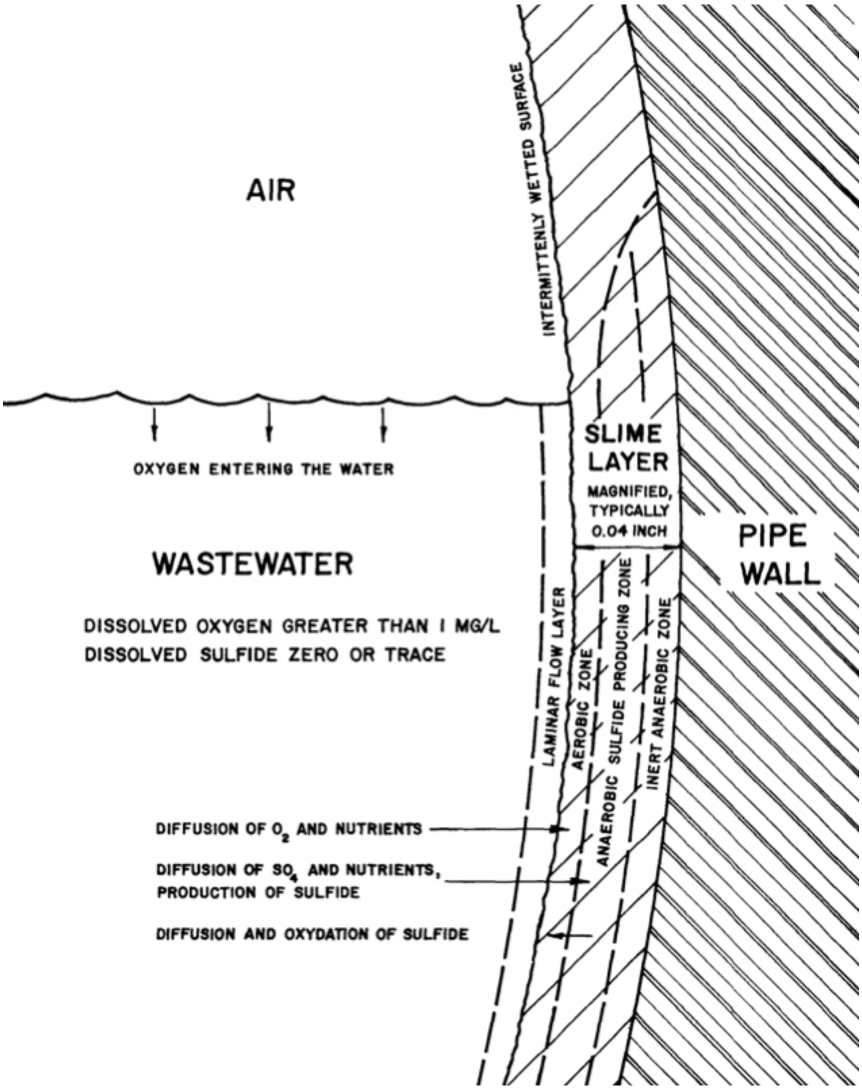


Figure 4.1: Sulfide production on submerged pipe wall at high oxygen concentrations **(U.S. Envitonmental Protection Agency 1974)**

If the oxygen concentration in the wastewater reaches a low level of around 0.1 mg/l sulfide will be able to escape the biofilm and will be released into the stream. Also if the wastewater is stationary or very slow moving, the laminar flow along the pipe wall may be depleted of oxygen and sulfide is released into the water even at higher oxygen concentration than 0.1 mg/l (up to 1 mg/l). This situation leads to a change in the biofilm zones, the aerobic zone ceases and anaerobic sulfide producing zone dominates (see ***Figure 4.2***) (U.S. Envitonmental Protection Agency 1974).

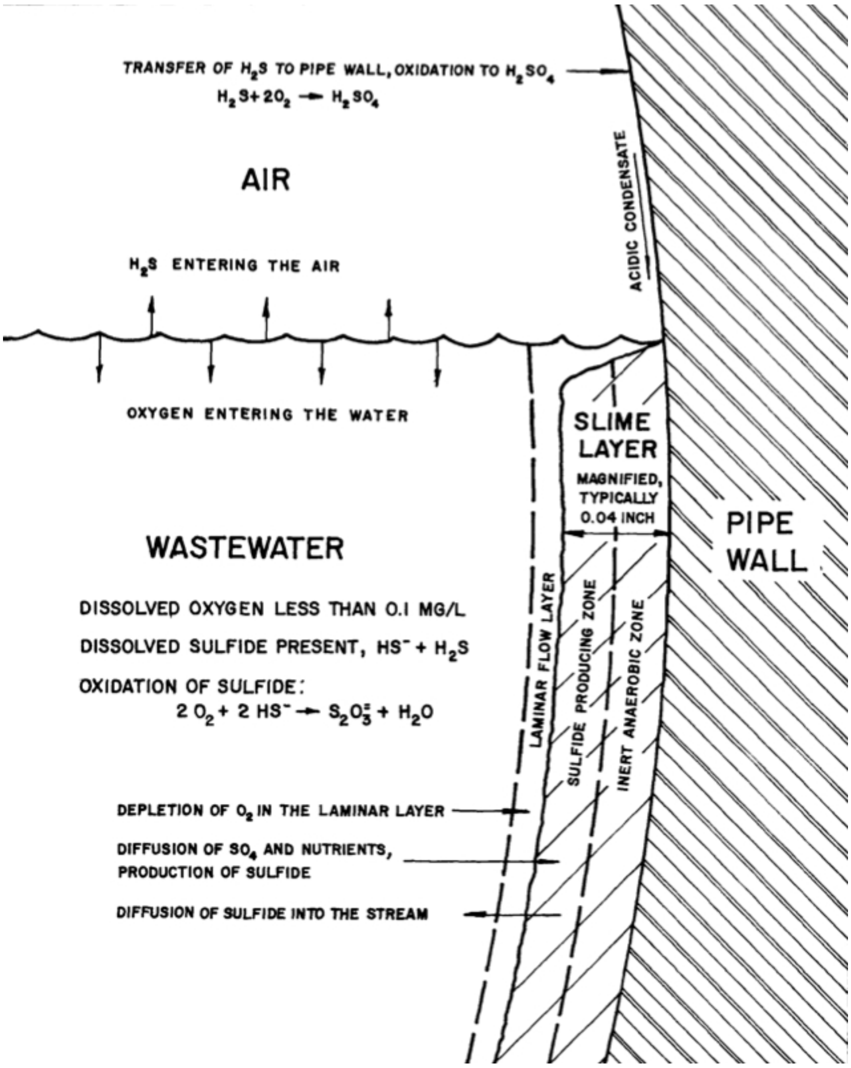


Figure 4.2: Sulfide production on submerged pipe wall at low oxygen concentrations **(U.S. Envitonmental Protection Agency 1974)**

The sulfate concentration in wastewater is normally high and usually not a limiting factor for sulfide production. The concentration of organic nutrients is also usually high, but if the concentration decreases, the sulfide production will decrease.

High temperature in the wastewater increases the biological activity and speeds up the diffusion capacity and reaction speed. The sulfide production increases by 7% by a temperature increase of 1^0C (U.S. Envitonmental Protection Agency 1974).

In a typical Norwegian wastewater network the gradients and stream velocity are relatively high. The high concentration of dissolved oxygen, low temperature and low rate of organic matter leads to low production of sulfide. The situation is similar to the one illustrated in ***Figure 4.1***. The oxygen concentration is often more than 1 mg/l, which means an instant oxidation of the sulfide as it reaches the aerobic zone. For most of the year temperatures are too low to have any production at all (Sægrov 1992).

The sulfide production in Norwegian wastewater is most common in wet wells of pumping stations, sludge separators and pressure mains, and occasionally in gravity sewer lines with small gradients or depressions. In these areas the stream velocity is sufficiently low and also the supply of oxygen (Sægrov 1992). The situation is similar to ***Figure 4.2***.

The actual sulfuric acid attacks on concrete pipes are located above the water surface on the damped pipe wall. Most common is the reaction between hydrogen sulfide, H_2S and drops of water on the pipe wall surface to form sulfuric acid. The formation of sulfuric acid by oxidation of hydrogen sulfide is as follows:

$$H_2S + 2O_2 \xrightarrow{bacteria} H_2SO_4$$

Equation 4.10: Formation of sulfuric acid

This redox reaction is a result of bacterial metabolism by the *Thiobacillus*, which is normal species in the wastewater bacterial flora. The bacteria use the sulfide as a source of energy and are capable to remain active at high concentrations of sulfuric acid, reducing the pH on the pipe wall down to under 0,5. The concrete is attacked, reacting with the acid producing pasty gypsum mass (calciumsulfate) and trace amounts of inert materials. If the pipe crown is dry and not moist, there is no production of sulfuric acid (U.S. Envitonmental Protection Agency 1974).

The production of sulfuric acid varies a lot with temperature and is up to 3 times higher during summer than winter. Surveys have shown a concentration of sulfuric acid up to 6% and a pH under 0,2 on a pipe wall with water temperature at 18ºC (Sægrov 1992).

4.2.2.2 Corrosion distribution

The corrosion of wastewater pipes above the water level is not uniform. This is because the corrosion depends on the air currents in the sewer atmosphere. Usually the air flows down the pipeline along with the stream, but because of temperature difference, a transvers air current can occur. Biological activity heats the wastewater and is usually warmer than the surrounding pipe wall. This results in a rising air current in the warmer air above the water. When the air reaches the pipe crown it cools down and flows down again along the pipe wall, see ***Figure 4.3***. This air circulation is more intense during summertime because of generally higher water temperature, and for the same reason the sulfide production and concentration are higher. The hydrogen sulfide gas rises from the water with the maximal rate of transfer at the pipe crown. The acid containing condensate runs down the pipe wall. Acid deteriorates the concrete in the area above the water surface, with the highest penetration rate observed just above the water surface (see ***Figure 4.3***). This is because the water immediately washes away the decomposition product (gypsum) and new parts of the pipe wall are open for acid attacks (U.S. Envitonmental Protection Agency 1974).

Figure 4.3: Sulfuric acid corrosion distribution **(U.S. Envitonmental Protection Agency 1974)**

In addition to the acid corrosion also small cracks in the concrete are formed. This is because the concrete swells when gypsum is produced and internal pressure in the concrete increases. A result of this process is accelerated corrosion because the cracks allow the acid to penetrate deeper (U.S. Envitonmental Protection Agency 1974).

Results of sulfuric acid attacks studied in Belgium are shown in ***Figure 4.4*** (***a-f***). The corrosion can eat away the concrete leaving aggregate and reinforcement visible (***Figure 4.4 a*** and ***b***). A reduction of the pipe wall thickness and loss of aggregates can result in a severe reduction of the mechanical strength of the pipe. Reinforcement can be weakened due to corrosion, which makes the situation even worse. The pipes ability to withstand gravity and traffic forces are reduced and may lead to cracks (***Figure 4.4 d***) or collapse (***Figure 4.4 f***). Exfiltration of wastewater polluting the surroundings or infiltration of groundwater are also resulting effects (***Figure 4.4 c***) (Donckels, et al. 2010).

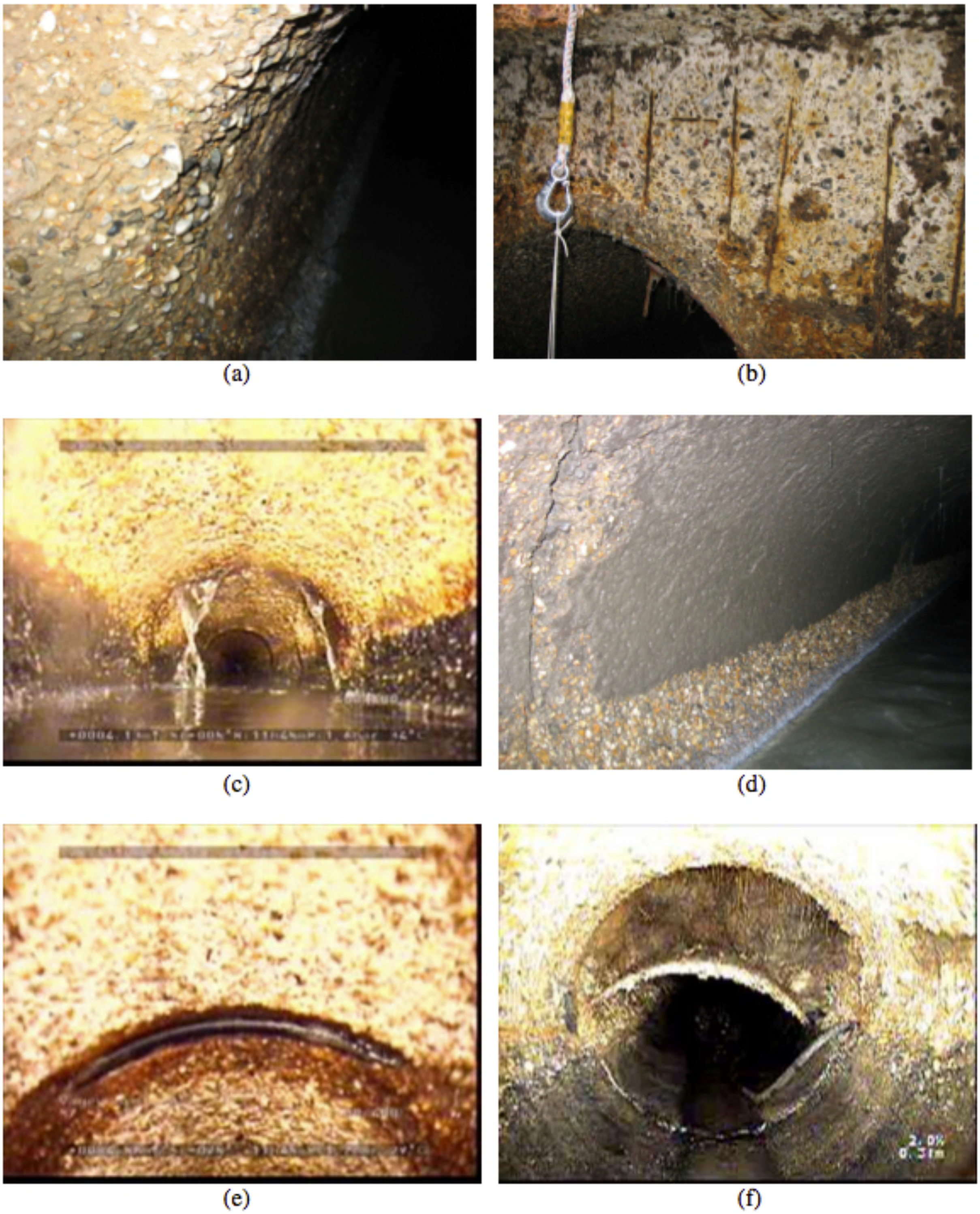


Figure 4.4: Examples of sulfuric acid attacks **(Donckels, et al. 2010)**

4.2.3 Empirical model of sulfide production in pressure mains

The Danish Professor T. Hvitved-Jacobsen at Aalborg University published in 1988 an empirical model for predicating sulfide production in pressure main. In 1998 this model was evaluated and modified by P.H. Nielsen, based on results from two pressure mains at Northern Jutland in Denmark (Nielsen, Raunkjær and Hvitved-Jacobsen 1998).

Important parameters to estimate the production rate of sulfide are the sulfate and organic matter concentrations (BOD or COD) and temperature. The sulfate concentration is usually not limiting for the sulfate reduction, but the concentration of organic matter may influence the reaction.

The objective of the study done in 1998 was to improve the empirical model described by this equation:

$$r_S = a(COD_{sol} - 50)^{0.5} \cdot \theta^{t-20}$$

Equation 4.11: Rate of sulfide production

$r_S = Rate\ of\ sulfide\ production\ [gS/m^2 \cdot h]$
$COD_{sol} = Chemical\ oxygen\ demand\ for\ a\ filtered\ sample\ [gO_2/m^3]$
$t = Temperature\ [\ ^oC]$
$\theta = temperature\ correction\ constant\ [-]$
$a = wastewater\ quality\ parameter\ [-]$

		Hvitved-Jacobsen, 1988	Nielsen, 1998
Temperature correction constant	θ	1.07	1.03
Regular domestic wastewater	a	0.0015	0.0010-0.0020
Mixed domestic and industrial wastewater	a	0.0030	0.0030-0.0060
Mainly industrial wastewater	a	0.0060	0.0070-0.0100

Table 4.5: Constants used in the sulfide production model **(Nielsen, Raunkjær and Hvitved-Jacobsen 1998)**

Table 4.5 shows the values of the 1988 study and adjusted values of the 1998 study. Nielsen suggested two things: To lower the temperature correction constant (θ) and raise the wastewater quality parameter (a), especially if food-processing industry is contributing to the wastewater (Nielsen, Raunkjær and Hvitved-Jacobsen 1998).

4.3 Abrasion Erosion

Abrasion erosion of the concrete material inside pipelines has been observed in many Norwegian cities, and is mainly caused by sand transported in the wastewater. The main factors influencing the erosion are stream velocity, amount of sand transported and concrete quality.

During the 1970's a Norwegian research was conducted and concluded that sand erosion effected pipe bends to some extent and strait pipelines was not very effected. The worst erosion was found at the bottom of the pipe just after connections (Schei and Tekle 1976) most likely because of the turbulence occurring at this local point.

The survey also showed that the worst damage correlated very well with the lowest concrete quality (Søpler 1976).

The survey resulted in a method for calculating the abrasion erosion caused by sand:

$$S_0 = k_0 M_S \frac{D_0}{D}$$

Equation 4.12: Mean depth of erosion

$$S_{max} = k_S S_0$$

Equation 4.13: Max depth of erosion

$S_{max} = Max\ depth\ of\ erosion\ [mm]$
$S_0 = Mean\ depth\ of\ erosion\ [mm]$
$M_S = Transportated\ sand\ [10^3 \cdot tons]$
$D_0 = Experimental\ pipe\ diameter\ [200\ mm]$
$D = Mean\ depth\ of\ erosion\ [mm]$
$k_0 = Pipe\ material\ coefficient\ [mm/10^3 \cdot tons]$
$k_S = Abrasion\ coefficient$

The maximal erosion at connections is calculated by applying the erosion coefficient, k_S. This factor expresses how well the two pipes are connected. At ideal conditions with almost no gap k_S is measured to be 2.0. At small stream velocities (< 3 m/s) and a non-ideal connection situation was k_S measured to ~4. At 6 m/s it was measured to ~8 (Søpler 1976).

A study was done in Taiwan focusing on abrasion erosion of concrete by water-borne sand. They found how the erosion varied with water-cement ratios (w/cm), permeability and the concretes splitting tensile strength capacity (Liu, Yen and Hsu 2006).

The study concluded:

1. Increasing w/cm ratio is decreasing the abrasion resistance (See ***Figure 4.5***)
2. A concrete with increasing permeability results in a weaker abrasion resistance (See ***Figure 4.6***)
3. Increasing splitting tensile strength is increasing the abrasion resistance (See ***Figure 4.7***)
4. Weaker concrete with a coarser aggregate gives greater abrasion resistance

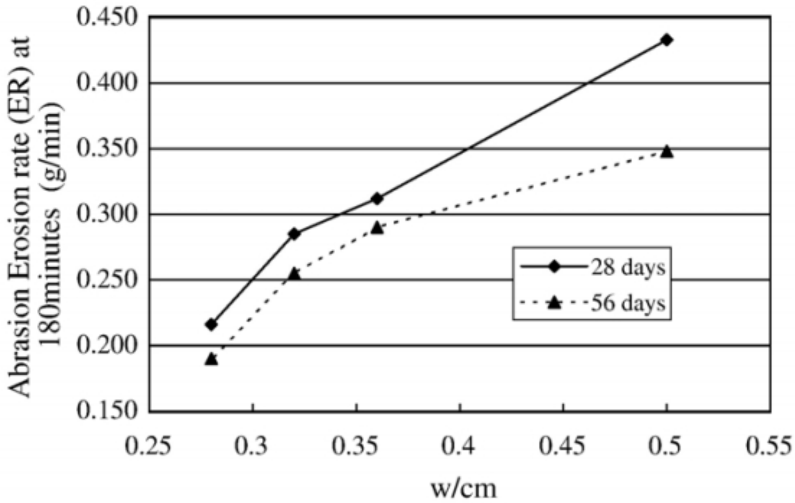


Figure 4.5: The abrasion erosion rate as a function of w/cm ratio **(Liu, Yen and Hsu 2006)**

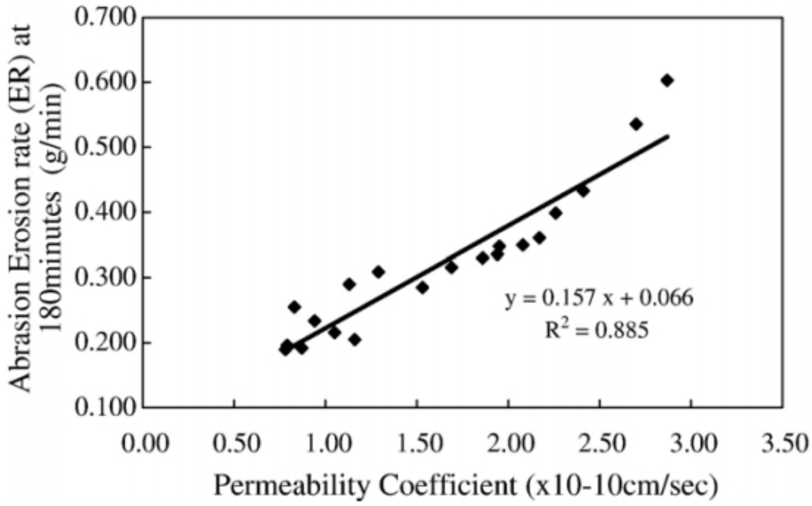


Figure 4.6: The abrasion erosion rate as a function of permeability **(Liu, Yen and Hsu 2006)**

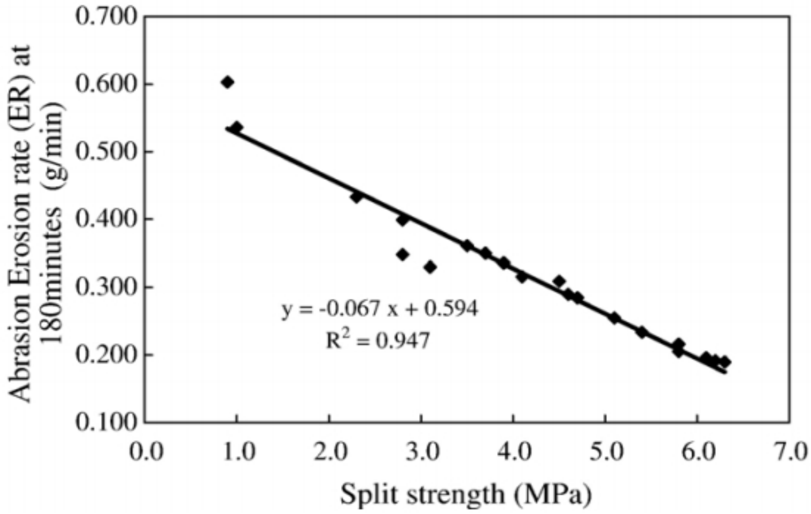


Figure 4.7: The abrasion erosion rate as a function of Split strength **(Liu, Yen and Hsu 2006)**

5 Modeling and Calculations by use of Examples

This chapter consists of theoretical calculations based on mechanical load theory, and two example cases focusing on two specific objectives:

1. Demonstrate how theoretical calculations can support the revised condition assessment method
2. To exemplify one possible usage of the revised condition assessment method

The revised condition assessment method, which is explained in chapter 6, is based on an indirect and a direct condition assessment (referred to as ICA and DCA). ICA is an assessment of the three most important influencing forces on concrete pipe deterioration: fill load, traffic load and surface damage by sulphuric acid. DCA is an enhanced assessment of cracking and surface damages, which is observed in a pipe. The observations and assessment of the inside of the pipe are supporting the ICA. A total assessment with both methods (ICA and DCA) is expressing the actual pipe condition better and more correctly than the NV150.

Chapter 5.1 consists of theoretical calculations of fill/traffic load, moment and critical wall thickness as a function of cover height. These calculations are made to give basis for the evaluation of fill and traffic loads in the ICA, and the severity of cracks and surface damage in the DCA.

Chapter 5.2 consists of two examples, where ICA is used to adjust and correct the performed condition assessment by using NV150.

5.1 Calculation of Mechanical Loads and Forces on Buried Concrete Pipes

In the following a theoretical calculation of mechanical loads, such as fill and traffic loads as a function of cover height is done. Also calculations of moment generated in the pipe wall because of the loads and the critical wall thickness to withstand the moment as a function of cover height are estimated.

All the calculations are based on the theory explained in chapter 3.3, and express the forces applied to a DN300 rigid pipe, buried under 0-9 meters of cover height defined as an embankment with positive projections. Appendix 3 shows the formulas and calculation results.

5.1.1 Fill and traffic load calculations

Figure 5.1 shows how fill and traffic loads vary with the increasing cover height. The theory behind this calculation is explained in chapter 3.3.1 and 3.3.2.

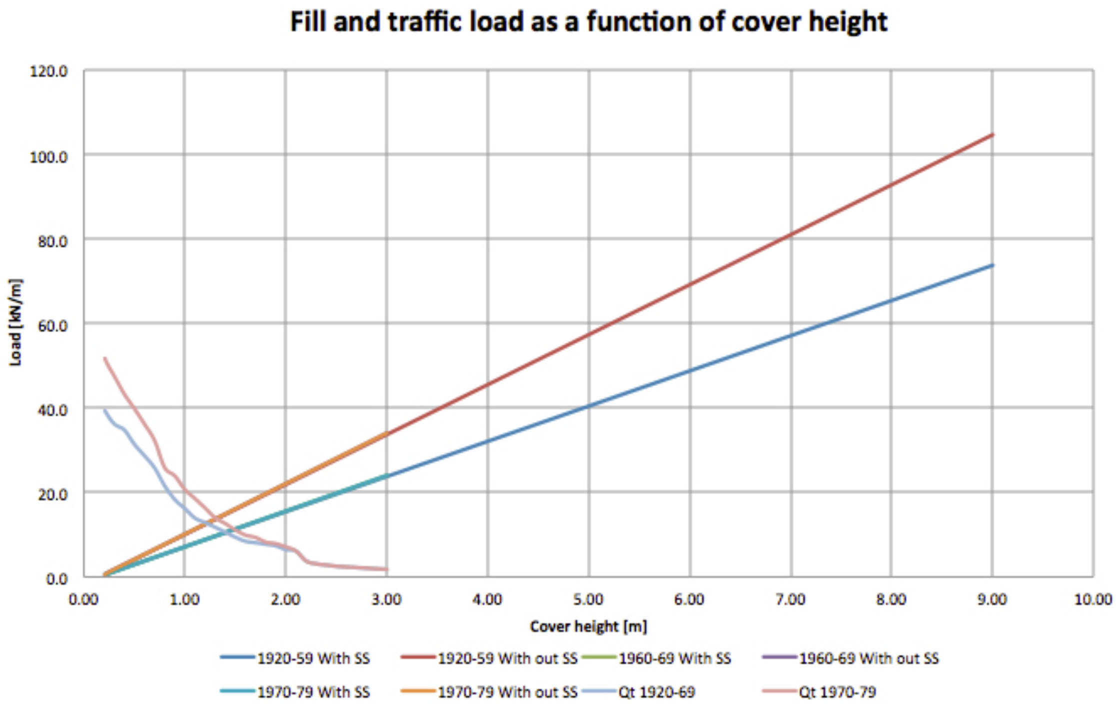


Figure 5.1: Fill and traffic load as a function of cover height

The equations used for the fill load calculation are ***Equation 3.7*** and ***Equation 3.8***, which needs the following input:

- Cover height (H), which varies from 0.2 – 9.0 meters
- The fill mass density (γ), assuming gravel at 2000 kg/m^3
- The outside pipe diameter (B_c), this value varies with the construction period (see ***Figure 3.17***)

DN300	Wall thickness [mm]	Outside diameter [m]
1920-59	30	0.360
1960-69	32	0.364
1970-79	44	0.388

The fill load is calculated for two separate cases; good or poor support from the side filling (referred to with side support (SS); ***Equation 3.7*** and without side support (SS); ***Equation 3.8***).

The fill load on pipes with support from the side filling is increasing linearly by approximately 8 kN for every meter of cover height. The fill load on pipes with poor side filling support is increasing linearly by approximately 12 kN for every meter of cover height.

That is a 50% higher increase of the load by every meter of cover height, for the situation without SS compared to the situation with SS.

The equation used for the traffic load calculation is ***Equation 3.9***, which needs the following input:

- Loading coefficient (C_t), a function of H, B_c and pipe length, see ***Figure 3.8***
- Impact coefficient (Ψ), a function of H, see ***Equation 3.12***
- Concentration coefficient (k), a function of H and B_c, see ***Equation 3.10*** and ***Equation 3.11***

The traffic load is calculated for two periods 1920-69 and 1970-79 because of the increase in outer diameter. It is only calculated for these to periods because a difference by four mm (1920-59 vs. 1960-69) did not give any significant result difference.

The traffic load is decreasing exponentially with the increasing cover height. If the cover height is larger then 2.5-3.0 meters the total load applied to the buried pipe is dominated by the fill load and the traffic load can be neglected. The traffic load applied to the pipes built after 1970 is higher than for the older pipes and the reason for this is the size of the pipe outside diameter (B_c). The inside diameter is the same for all the pipes in the calculation, but the standard wall thickness varies with the age of the pipe (see Appendix 2). Higher outside diameter corresponds to a higher traffic load.

5.1.2 Moment calculation

Figure 5.2 shows how the moment in the pipe wall varies with the increasing cover height. The theory behind this calculation is explained in chapter 3.3.4 (and ***Figure 3.9***).

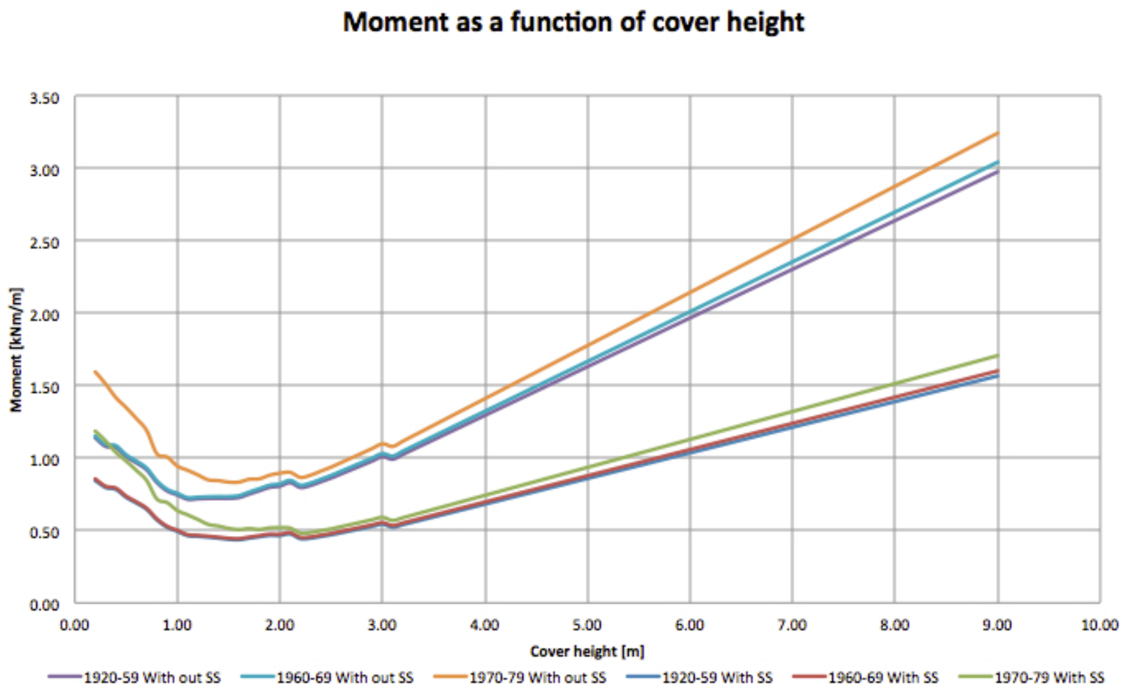


Figure 5.2: Pipewall moment as a function of cover height

The equations used for calculating the moment are ***Equation 3.21*** and ***Equation 3.22***, which need the input of:

- Fill load, values from ***Figure 5.1***
- Traffic load, values from ***Figure 5.1***
- The outside pipe diameter (B_c), this value varies with the construction period (see ***Figure 3.17***)

DN300	Wall thickness [mm]	Outside diameter [m]
1920-59	30	0.360
1960-69	32	0.364
1970-79	44	0.388

The moment calculations are done for three different time periods, because of the increase in wall thickness and for fill/traffic load with SS (***Equation 3.21***) and with out SS (***Equation 3.22***).

The graph expressing the moment function can be divided in two with a limit at 2.5 meters of cover height.

1. 0-2.5 meters where the moment decreases exponentially, due to the domination of traffic load

2. 2.5-3.0 meters where the moment increases linearly, due to the domination of fill load

At the transition area 1.5-2.5 meters of cover height, the lowest calculated value of the moment is found.

The moment is calculated for the same two cases as above (***Figure 5.1***). The first is with support from the side filling, similar to ***Figure 3.9 a,*** with the lowest calculated value of 0.5 kNm/m. After the low point the moment is increasing linearly by 0.33 kNm/m for every meter of increased cover height. The second case with no side filling support has the lowest calculated value 0.8 kNm/m. After the low point, the moment is increasing linearly by 0.67 kNm/m for every meter of increased cover height. After the low point (1.5-2.5m) the moment increases twice as much per meter of increased cover height, for the situation without SS compared to the situation with SS.

5.1.3 Calculation of critical wall thickness

Figure 5.3, ***Figure 5.4***, ***Figure 5.5*** and ***Figure 5.6*** show calculation of minimum required wall thickness to withstand the moment forces caused by traffic and fill load at a certain cover height. The theory of these calculations is explained in chapter 3.3.4.

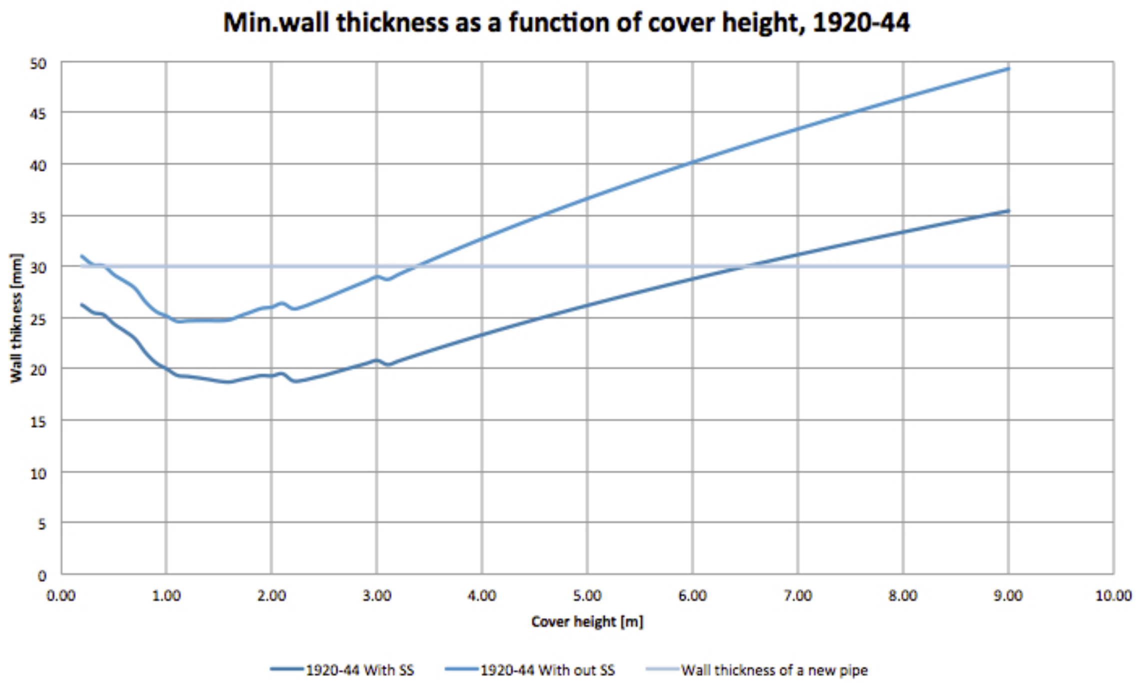


Figure 5.3: Required minimum wall thickness of pipes from 1920-44, with and with out Side Support

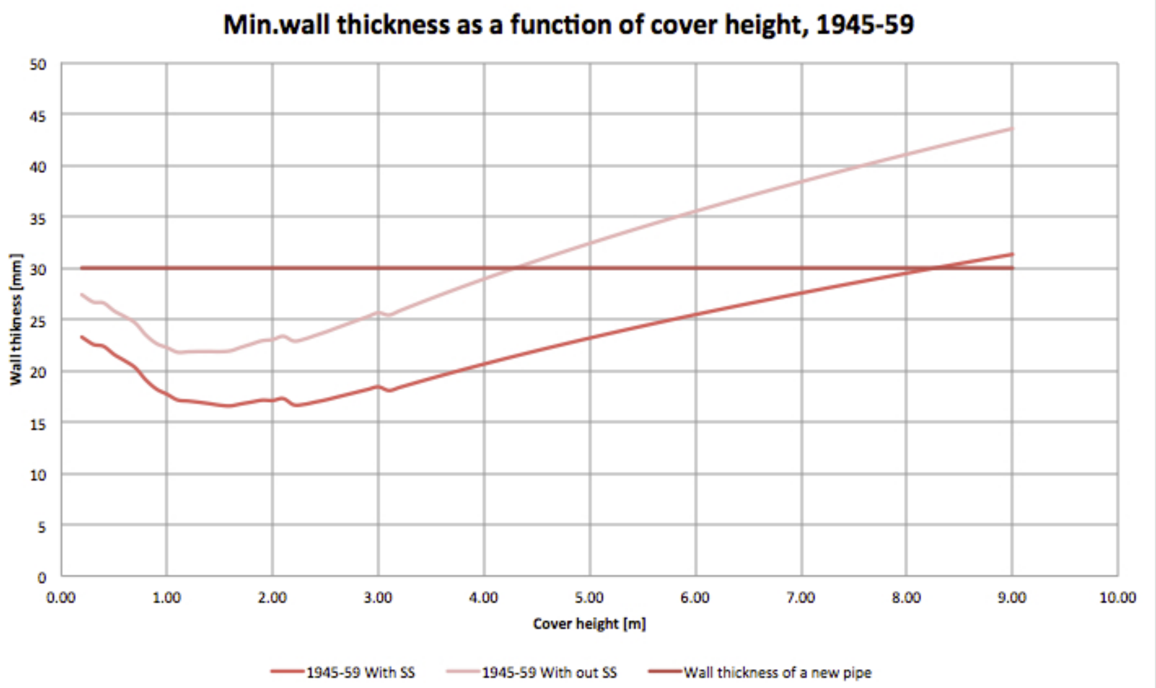


Figure 5.4: Required minimum wall thickness of pipes from 1945-59, with and without Side Support

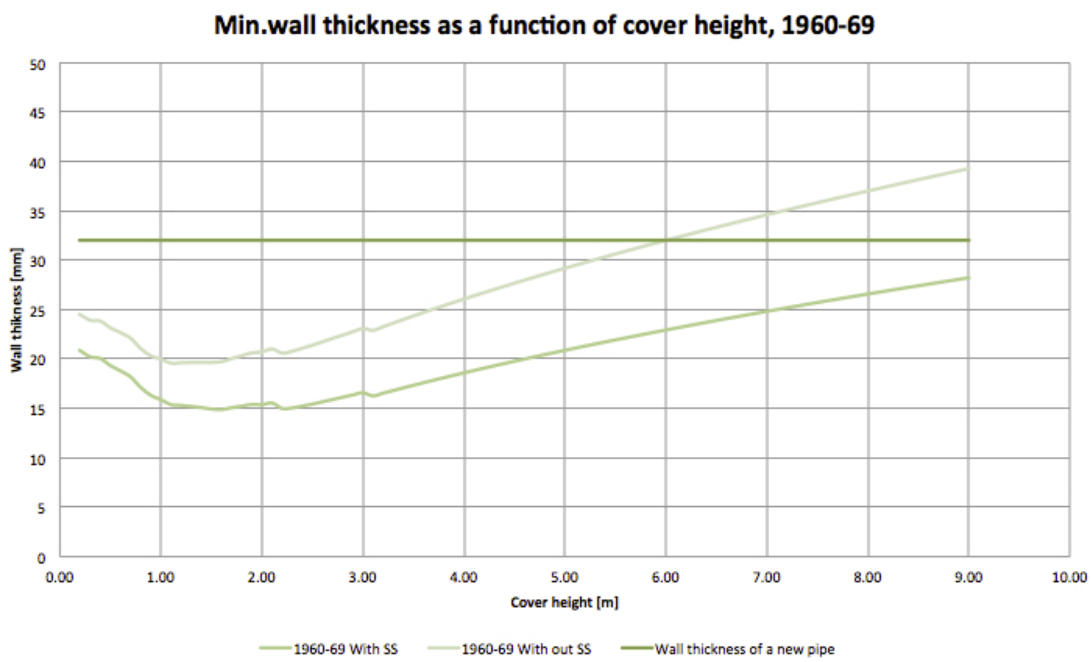


Figure 5.5: Required minimum wall thickness of pipes from 1960-69, with and with out Side Support

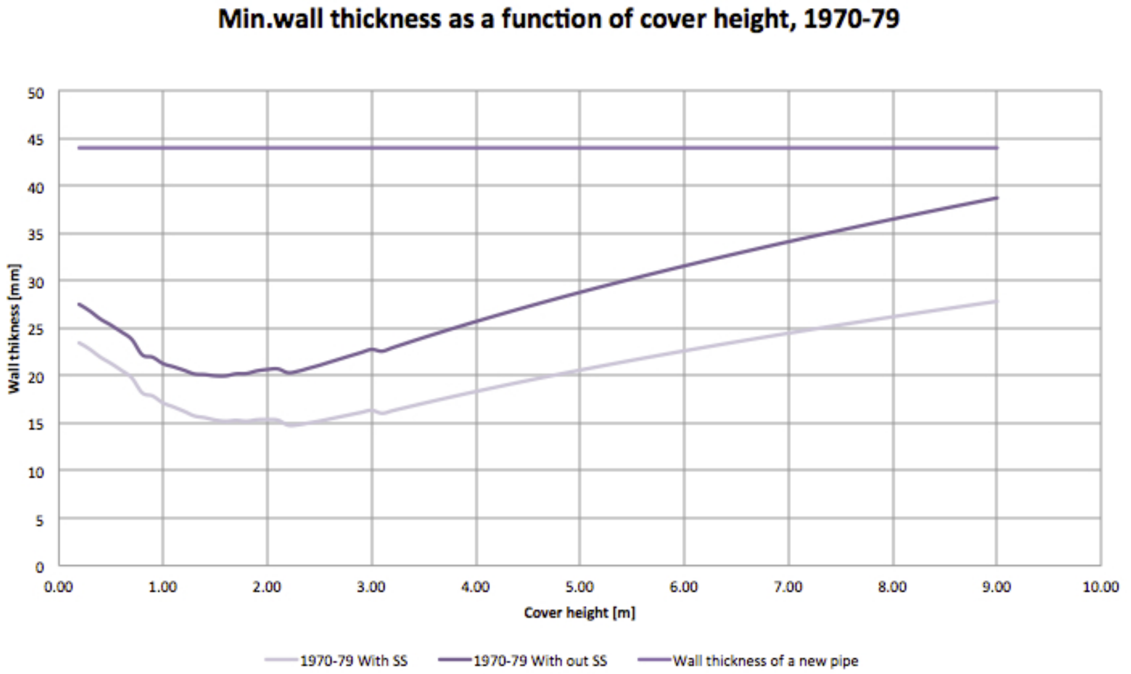


Figure 5.6: Required minimum wall thickness of pipes from 1970-79, with and with out Side Support

Equation 3.19 is used to calculate the critical wall thickness.

The input needed to calculate the wall thickness is:

- Pipe wall moment, values from ***Figure 5.2***
- Crush load tensile strength (σ_{max}), this value varies with the construction period (ref: ***Table 3.6***)

	Wall thickness [mm]	σ_{max} [kPa]
1920-44	30	7320
1945-59	30	9350
1960-69	32	11780
1970-79	44	12900

Because of the difference in σ_{max} the minimum wall thickness is calculated for four different ages of the pipes:

- 1920-44 (***Figure 5.3***)
- 1945-59 (***Figure 5.4***)
- 1960-69 (***Figure 5.5***)
- 1970-79 (***Figure 5.6***)

All the figures are displaying the standard wall thickness of a new pipe from the period (Standard wall thickness is taken from Appendix 2).

Table 5.1 shows key information from the wall thickness calculation.

	Low point, with SS	Low point, without SS	Intersection with Standard wall thickness. with SS	Intersection with Standard wall thickness. without SS
1920-44	1.6 m, 18 mm	1.1 m, 24 mm	6.4 m, 30 mm	0.4 m, 30 mm and 3.4 m, 30 mm
1945-59	1.6 m, 17 mm	1.1 m, 22 mm	8.2 m, 30 mm	4.3 m, 30 mm
1960-69	1.6 m, 15 mm	1.1 m, 19 mm	-	6.0 m, 32 mm
1970-79	2.2 m, 14 mm	1.6 m, 20 mm	-	-

Table 5.1: Key information from the calculation of wall thickness vs. cover height

5.2 Example Cases Illustrating the Adjustment of the NV150 Condition Assessment

Two examples where ICA is used to adjust and correct the condition assessment found by the Norwegian method NV150 are presented. Both the examples describe two pipes in a random Norwegian city with random observations.

The first example case is a pipe with longitudinal cracking and a low condition class of 2 that may be too low for this type of cracking. The ICA is used to assess the deterioration processes influencing this pipe and correcting the condition class if the influencing loads are significantly high.

The second example case is a pipe with surface damages and a high condition class of 4 that may be too high for this kind of damage. The ICA is used to assess the deterioration processes influencing this pipe and correcting the condition class if the deterioration of the pipe surface is significantly high.

5.2.1 Example 1, Longitudinal cracking

Example 1 is a DN300 for combined storm and wastewater, installed under a road in a Norwegian city, all assumptions are conceptual.

Table 5.2 includes key information about the pipe needed for the Norwegian condition assessment (NV150).

Sewer type:	Comb. Storm and Wastewater
Dimensions:	300 mm
Material:	Concrete
Length:	47m
Construction year:	1949
Inspection year:	2010

Table 5.2: Pipe information

Table 5.3 consists of the observations done in example pipe 1, where all are given a two-lettered code (explained in ***Table 2.1***) and a grade number (see ***Table 2.4***) Grading of cracks are described in ***Table 2.2***. The observations marked with red are structural damages. ***Table 5.3*** gives the information of the observed location, length, position on the pipe wall and the calculated damage points.

Observa-tions	**Observa-tion start**	**Observa-tion end**	**Clock-position 1**	**Clock-position 2**	**Damage points**
SI	0,0				-
FS1	7	46	12	12	0
DS	37				0
SR2	38		12		2
SR2	44	47	12		6
IF	47				-
				∑ DP:	**8**
Condition score:					17
Condition class:					**2**

Table 5.3: Condition assessment with NV150

$$CS = \frac{100 \cdot 8}{47} = 17$$

The equation above calculates the condition score of the pipe, according to ***Equation 2.2*** in chapter 2.4.1. This gives us the following pipe condition class (See ***Table 2.6***)

Condition class = **S2**

In this example the pipe is given the condition class of 2, which is a low class. ***Table 2.6*** defines the pipe as in *good shape*. The most severe observation in the pipe is a 3 meter long crack in the crown of the pipe (see ***Figure 5.7***). As mentioned in chapter 3.1 longitudinal cracking is a result of overloading of the pipe and then condition class 2 is too low. An adjustment is needed to correct the condition assessment for this pipe.

Figure 5.7: 3m longitudinal cracking

It is assumed that the cover height and fill load are not critical for the pipe strength capacity, i.e. the cover height over the pipe is not grater than the critical cover height (h_{max}). This will be explained in more detail in chapter 6.2.1.

The pipe is installed beneath a road, and the part with the observed cracking can be located in three different zones of the road. According to ***Figure 5.8*** these three zones have different grades of traffic, which can be categorized as:

- Red zone is a very buzzy part of the road and the design traffic load is applied to the pipe several times every day.
- Yellow zone is a buzzy part of the road and the design traffic load is applied a couple of times every week.
- Green zone is a calm part of the road and the design traffic load is applied less than one time every week.

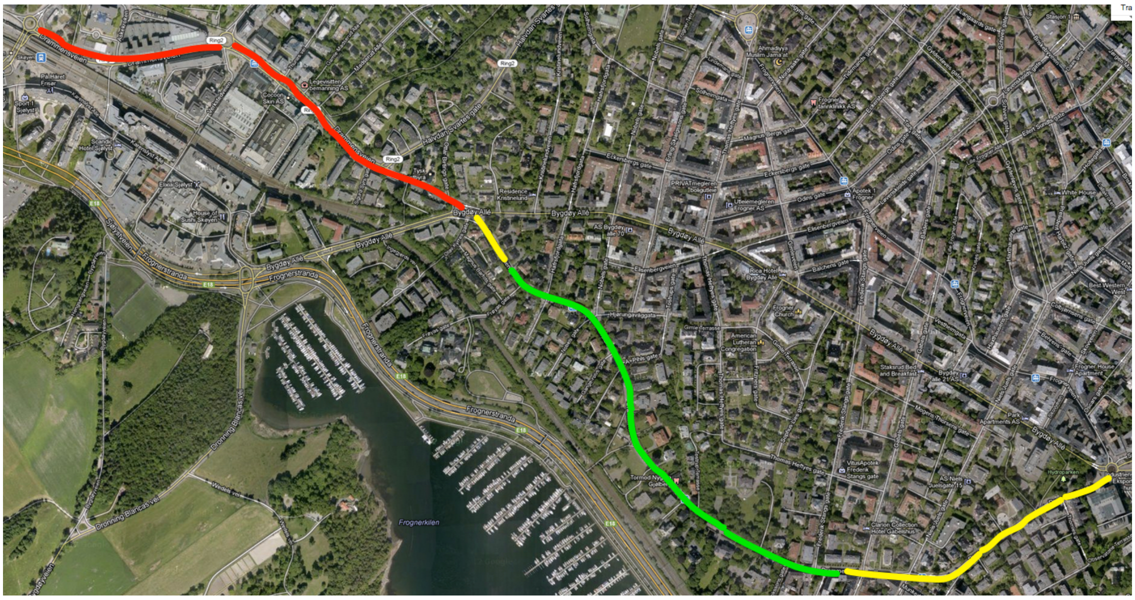


Figure 5.8: Map over the road where the pipe is located

Based on which of the zones the cracked part is located, I will give advice on how to adjust the condition class of the pipe by using the ICA traffic load explained in 6.2.2.

1. The cracked part is located in the RED zone.
   The design traffic load is applied on top of the pipe several times every day, which means there is a high possibility that the crack will grow. This makes the situation unstable and there is a possibility of a pipe failure. I suggest that the pipe condition class is adjusted from 2 to 4. I would also advise that this crack is monitored every 6-12 months.

2. The cracked part is located in the YELLOW zone.
   The design traffic load is applied a couple of times every week, which gives a possibility for crack growth. The situation is more stable than case 1, but it is still a possibility that the pipe will have a shorter service life than expected. I suggest that the pipe condition class is adjusted from 2 to 3. I would also advise to monitor the crack every 3-5 year.

3. The cracked part is located in the GREEN zone.

The design traffic load is not very often applied and the situation is fairly stable. If there are no other severe damages observed in the pipe, there is a high possibility that it will reach the expected lifetime. There is no need to adjust the condition class, and the crack might need monitoring every 10 years.

5.2.2 Example 2, Surface damage and decreased wall thickness

In this example a DN200 wastewater is installed in a calm suburban city street in Norway, and all assumptions made in this example are conceptual.

Table 5.4 includes key information about the pipe needed for the Norwegian condition assessment (NV150).

Sewer type:	Wastewater
Dimensions:	200 mm
Material:	Concrete
Length:	48,47 m
Construction year:	1969
Inspection year:	2009

Table 5.4: Pipe information

Table 5.5 consists of the observations done in example of pipe 2, given a two-lettered code (explained in ***Table 2.1***) and a grade number (see ***Table 2.4***). Grading of cracks are described in ***Table 2.2***. The observations marked with red are structural damages. Other information in ***Table 5.5*** is the observations of location, length, position on the pipe wall and the calculated damage points.

Observa-tions	**Observa-tion start**	**Observa-tion end**	**Clock-position 1**	**Clock-position 2**	**Damage points**
SI	0,0				-
KO2	2	48	12	12	46
IF	48				-
				∑ DP:	**46**
Condition score:					97
Condition class:					**4**

Table 5.5: Condition assessment with NV150

$$CS = \frac{100 \cdot 46}{48} = 96$$

The equation above calculates the condition score of the pipe, according to ***Equation 2.2*** in chapter 2.4.1. This gives us the following pipe condition class (See ***Table 2.6***)

Condition class = **S4** (See ***Table 2.6***)

In this example the pipe is given a calculated condition class of 4, which is high. ***Table 2.6*** defines the pipe as in *very bad shape*. The most severe damage is a grade 2 surface damage that means visible aggregates (see ***Table 2.3***).

Figure 5.9 shows the surface damage, which can be evaluated as a severe damage if the pipe has a cover height close to the critical fill load on the pipe. In this case it is assumed that the pipe is in the green category of ICA fill loads, that means the cover height is smaller than the critical cover height (h_{max}). This pipe is located in a suburb residence area in a street with little traffic and by taking this in to account it might be assumed that the design traffic load of this pipe is not very often applied to it. This situation equals the green ICA traffic load category.

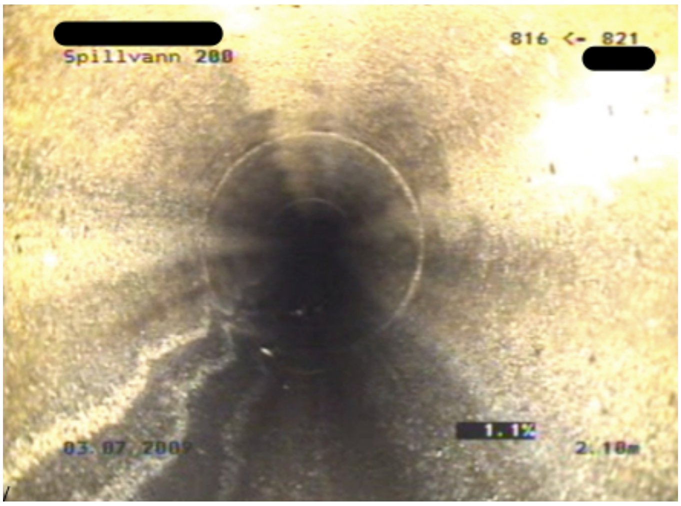


Figure 5.9: Surface damage

Based on the assumptions above a condition class 4 is not a suitable condition assessment, and an adjustment of the class by using the ICA surface damage (explained in 6.2.3) is advised.

The ICA is evaluating both fill and traffic load to be of the GREEN category, which means that the pipe condition class can be moved to a lower class. However, the surface damage deterioration speed needs to be evaluated. If there are any traces of sulfuric acid attacks or any possible areas of sulfuric acid production upstream, this situation may become unstable. This gives us three possible scenarios:

1. GREEN, No traces of Sulfuric acid and there is no possible sources of production upstream.
   This case is evaluated to be a stable situation, the deterioration speed is not considered to be any higher than in other regular sewers. The pipe condition class can be adjusted from 4 to 2. I would advice to monitor as regular pipes.
2. YELLOW, There are traces of Sulfuric acid, but there is no possible source of production upstream.

There have been Sulfuric acid attacks and it can happen again. The deterioration speed could be higher than in regular sewers, but the situation is still not critical so the condition class can be adjusted from 4 to 3. I would advise to monitor every 5 years.

3. RED, There are traces of Sulfuric acid and there is possible production upstream.
   This is a critical scenario and the deterioration speed could be high. No adjustment of the condition class is advised. The levels of Sulfuric acid from the source needs to be monitored and actions to prevent further damages needs to be done. I would advise to monitor the pipe situation every 6-12 months.

6 Discussions and Recommendations

Based on the descriptions given in the preceding chapters I will discuss possible options to enhance the condition assessment of cracks and surface damage in rigid concrete pipes, and give advice to include additional important information for a pipe condition assessment. Description and examples of new methods of assessing the pipe condition are also presented.

A condition based deterioration model (referred to as the model) is used for maintenance planning in Norway, applying the condition assessment as input. The model is supposed to give an indication of the time when the pipe is not able to perform the service required, and the intention is to keep downtime and maintenance expenses at a minimum level. For the model to be successful, good input parameters are needed, which means in our case the condition assessment result.

As concluded in my specialization project work (Hauge 2012), today's method of condition assessment is not giving a good enough description of the actual condition of the pipe. ***Figure 6.1*** shows that a deterioration model moves from fair to poor at the level of minimal performance, but with the condition assessment method used today this gives an incorrect baseline for the deterioration model. I pointed out in my specialization project that the level of minimal performance level is reached "too soon" in a situation of surface damages and "too late" in a situation of a cracked pipe, as the condition assessment is overestimating and underestimating the severity of the surface- and crack damages respectively. This demonstrates that it is not relevant to adjust the whole scale of the deterioration model. However, it needs to be adjusted "down" in a case of surface damages and "up" in a case of a cracked pipe. Additionally the deterioration speed is dependant of the different deterioration forces and magnitude.

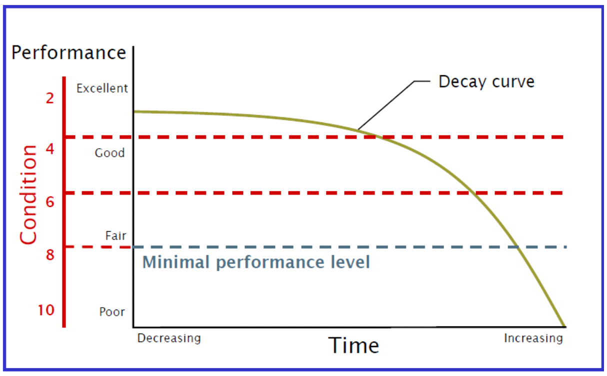


Figure 6.1: Correlation between condition based deterioration model and performance over time **(Ugarelli 2012)**

In general, an adjustment or a calibration could be a successful way to enhance a model, but in some cases it is hard to decide how to calibrate to get the best result. Consequently, I will propose to change some of the fundamental elements of the condition assessment method to enhance the deterioration model and to give a better overview of the situation. The objective is to include some input parameters, which are easier to calibrate, and also to give a better baseline for the deterioration model.

To understand the cause of events leading to damages in a wastewater pipe is challenging due to the complex nature of the sewer system. The driving factors of the damages are many and they can have incidental effects. This is a complex situation and to try to evaluate the result of a damage observed without taking into account what caused the damage, may produce many sources of error. The width of a crack or the residual strength of a wastewater pipe cannot be assessed without taking into account the driving forces of these situations.

In my opinion the science and knowledge of sewer systems, trenches and chemistry in the sewer must be improved significantly. This is important for creating a more complete and comprehensive picture of the process in the sewer system, and make a better condition assessment. The replacement rate of wastewater pipes in Norway is quite low, and we know by experience that problems and downtime of the sewer system will increase. To reduce the effect of this situation, a better condition assessment is requested so more efficient prioritization of maintenance of the pipes can be performed. The method used today is time consuming because of the manual inspection.
A revised assessment system can also be used to adjust/correct earlier condition assessments with NV150, as exemplified in chapter 5.2.

6.1 Indirect Condition Assessment and Driving Factors of Pipe Deterioration

By understanding the factors that influence the deterioration of concrete wastewater pipes, a more reliable picture of which section of the pipes that are in the critical zone of deterioration may be created. The most vulnerable pipes in the system will be located and suitable maintenance or remediation measures can be done before downtime occur.

The most critical factors creating cracks and reduced strength capacity in concrete pipes are filling load, traffic load and sulphuric acid attacks. The factors that influence the magnitudes of these forces are possible to measure. Also properties like tensile strength and wall thickness of the concrete pipe may be estimated according to the time of when the pipeline was built as explained in chapter 3.6 (see also Appendix 2).

Chapter 5.1 showed theoretical calculations expressing how fill load increased with cover height and by knowing the age and dimension of a given concrete pipe we can calculate the maximum cover height by using the wall thickness and the maximum tensile strength (see chapter 5.1.3).

The fill load and moment were increasing, respectively with 50% and 100% more per meter of increased cover height if the support from the side filling is poor, compared to good side support. This tells us that poor installation work will have a significant influence on the pipes ability to withstand the forces applied to it. As we know by experience, we can assume some characteristics about the installation work dependent on the age of the pipe. Chapter 3.6 showed that installation work done in the period from 1950-1970 were often of low quality. From the measured tensile strength we also know that the concrete pipe quality has increased over time (see chapter 3.6). Based on this information it is possible to comment on the findings represented by the calculations done in chapter 5.1 and ***Figure 5.3- Figure 5.6***.

1. The most likely situation for a pipe from 1920-1944 (***Figure 5.3***) is that the installation work is of fair quality, which is referred to *with side support* (lower graph). This means that these pipes are in the critical fill load zone, assuming 30 mm of wall thickness, when the cover height is exceeding 6.5 m. The trenches from this period are not so deep. If there is a reduction of wall thickness of 5mm the max cover height is reduced to 4.5 m. There is also an intersection at 0.5m that means the pipe is vulnerable to traffic load at this depth or less.
2. For a pipe from 1945-59 (***Figure 5.4***) the installation work is most likely of poor quality, which is referred to *without side support* (upper graph). This means that these pipes are in the critical zone, assuming 30 mm of wall thickness, when the cover height is exceeding 4.3 m. If there is a reduction of wall thickness of 5mm is the max cover height is reduced to 2.8 m. There is also an intersection at 0.6 m that means the pipe is vulnerable to traffic load at this depth or less.
3. The most likely situation for a pipe from 1960-69 (***Figure 5.5***) is that the installation work is of poor quality, which is referred to *without side support* (upper graph). This means that these pipes are in the critical zone, assuming 32 mm of wall thickness, when the cover height is exceeding 6 m. If there is a wall thickness reduction of 5mm, the max cover height is reduced to 4.3 m. For this period there is no intersection at low cover heights, which means that traffic load is not interfering.
4. The most likely situation for a pipe from 1970-79 (***Figure 5.6***) is that the installation work is of varying, but increasing quality. As we can see, the critical cover height is over 9 m. If there is a reduction of wall thickness of 5mm is the max cover height around 8.5 m, which means that pipes from this period can take quite a lot of surface damage. For this period there is no interference from traffic load.

The trend from 1920 until today is that the wall thickness has been increased. The quality of concrete and the installation work has also increased, see chapter 3.6 and ***Figure 3.17***.

The traffic load can be described as a dynamic load because a vehicle roles over the pipe. The deterioration speed of a pipe influenced by traffic load must be put in the context of the dynamic load frequency. It is therefore useful to establish a function of the traffic load vs. time, where the amount of heavy vehicles applying the design wheel load on the pipe during a time interval is measured. By using this as a part of the indirect condition assessment it is possible to range the pipes based upon level of traffic load influence.

Surface damage and sulphuric acid attacks are more challenging to measure. We know that the presence of sulphuric acid is varying over the seasons and that oxygen free environments are needed for sulphide production to occur (see chapter 4.2.2). Several factors must work together to develop sulphuric acid attacks, but some elements are more likely than others, as long pumping and pressure mains or large grease separators. Possible sources of sulphuric acid in our environments needs to be investigated more closely and then mapped as a part of the condition assessment. This has been done in Denmark, and they have also developed models to estimate the production of Sulphuric acid (see chapter 4.2.3). This work can be a good model for similar mapping in Norway.

6.2 Development of the Tool for Indirect Condition Assessment (ICA)

The reason for doing an indirect condition assessment, is to point out areas or specific pipes in the wastewater system that have potential to fail before expected or designed lifetime, without doing an inspection. It gives us the ability to increase the monitoring frequency on certain parts of the pipes and better plan the maintenance and rehabilitation than done today.

The most effective way to exploit the ICA is to develop a electronic-database (EDB) tool. Below is the method and the information needed to produce such a tool explained with respect to the three most important ICA factors; fill load, traffic load and surface damage. It is useful to have a pipe-vulnerability evaluation to visualize the importance of the functionality of each pipe in the whole system, to provide relevant maintenance to the most critical part of the pipeline. For instance a collapse in a downstream interceptor may cause problems upstream because of surcharge i.e.

The best benefit from the ICA tool will be if the information and evaluation of the influencing factors are displayed in a GIS based user interface in addition to regular database lists. Then you can localize pipes, which may be affected by sulfuric acid attacks because they are close to a potential source. Then through a DCA it is easier to see which of the pipes, and downstream area that can be defined as not affected by sulfuric acid.

6.2.1 Fill loads

As shown from calculations in chapter 5.1 there is a correlation between the cover height and the required wall thickness to support the load. By using this I propose to develop a tool to estimate which pipes that are above or under the critical cover height for the pipes maximum tensile strength. The result can be presented as a map with the pipes of the different categories: under, over and at critical depth with different colours, see ***Figure 6.2***.

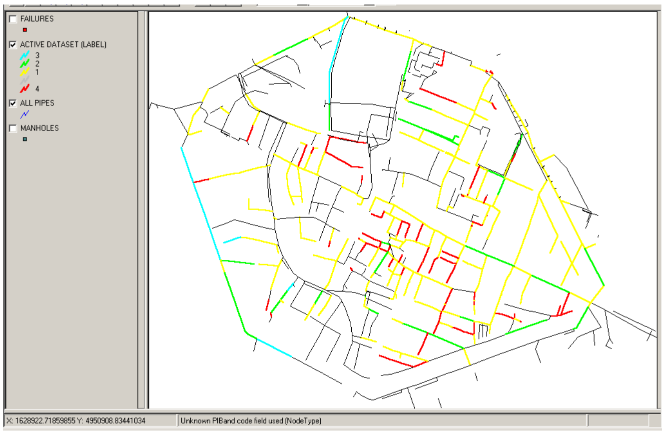


Figure 6.2: Indirect condition assessments of fill loads on pipes **(Ugarelli 2012)**

The information that needs to be provided to produce the ICA fill load map:

- Age of the pipe – assumed wall thickness and tensile strength
- Depth of upstream manhole (h_1)
- Depth of downstream manhole (h_2)

All pipes have a standard wall thickness after the standardized production demands from the period the pipe was produced (see, chapter 3.6 and Appendix 2). No surface damage or decreased wall thickness is assumed at this point. A maximum cover height (h_{max}) for the given pipe is defined, like calculated for a DN300 pipe in chapter 5.1.3 (see ***Figure 5.3** - **Figure 5.6***) for all pipe diameters and ages. With the depths, h_1 and h_2 a mean cover height over the pipe length is calculated (h_{mean}).

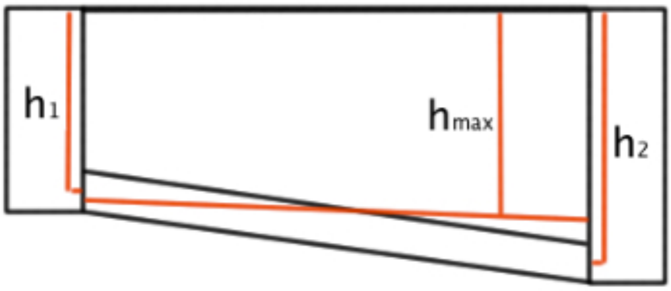


Figure 6.3: h_{max} for ICA fill load

Three categories are then defined:

- $h_{mean} < h_{max}$ (+1m). The pipe is at least one meter above the critical depth and is marked with the color GREEN
- $h_{mean} = h_{max}$ (+/-1m). The pipe is localized at the critical depth +/- one meter and is marked with the color YELLOW
- $h_{mean} > h_{max}$ (-1m). The pipe is at least one meter below the critical depth and marked with the color RED

If the pipe gradient is steep enough and/or the cover height increases along the length it is a possibility that the pipe crosses from one category to another.

Estimating category using ICA on fill load:						
Diameter Age	→	Tensile strength Wall thickness	→	h_{max}	→	RED/ YELLOW/ GREEN
Depth Length	→	h_1 and h_2 L_{Tot}	→	h_{mean}		

Table 6.1: ICA fill load

6.2.2 Traffic loads

As for the fill load the tool can also provide assessment of the traffic load influencing a pipe. From the calculations in chapter 5.1 we notice that the traffic load decreases with the increasing cover height. At 2.0-3.0 meters cover height the fill load is dominating and the traffic load is not influencing the pipe. This means that the traffic load is influencing pipes with low cover height (< 2.5 m), and only the parts of the pipe located under a road. Also for these pipes there are two categories; if the pipe is critically influenced by the traffic load or not, depending on the pipe tensile strength/wall thickness.

The information that needs to be provided to produce the ICA traffic load map:

- Age of the pipe – assumed wall thickness and tensile strength
- Depth of upstream manhole (h_1)
- Depth of downstream manhole (h_2)
- Length of the pipe which is located under the road (L_{T1}, L_{T2})

By using the age of the pipe, a standardized wall thickness and the pipe tensile strength, we are able to calculate the middle cover height over the partial pipe that is located under the road. Then it is possible to calculate the cover height where the pipe is entering the road area (h_{T1} at L_{T1}) and out of the area (h_{T2} at L_{T2}). For the given pipe a critical wall thickness is calculated for the fill load and the traffic load. The actual wall thickness is compared to the required wall thickness. We can say that the pipe is over or under critical influence by traffic load.

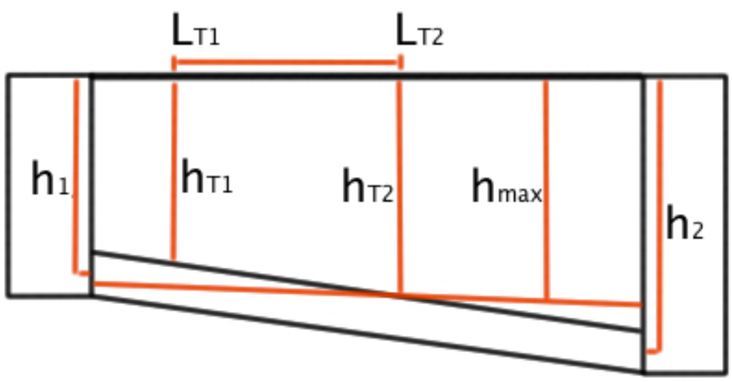


Figure 6.4: h_{max} for ICA traffic and fill load

The traffic load is usually not a constant, but a dynamic load, and the pipes in the critical traffic load zone need an evaluation of the amount of design wheel load affecting it over a time interval. The amount of traffic could for instance be categorized in these three classes:

- Red zone, heavy traffic (>600 YDT-L)
- Yellow zone, intermediate traffic (600-100 YDT-L)
- Green zone, light traffic (<100 YDT-L)

The amount of traffic on a certain road could either be evaluated, categorized with knowledge by the local authority, or the traffic can be measured. The traffic amount unit used in Norway is year-day-traffic or YDT, the number of vehicles during one median day of a year. YDT-L can be explained as the number of "long" vehicles (length equal 5.6 m of longer), which includes the pipe design wheel load category. A tool providing these numbers is NVDB (The National Road Data Bank). ***Figure 6.5*** shows an exemplified map of NVDB where information about traffic can be found (Statens vegvesen 2013).

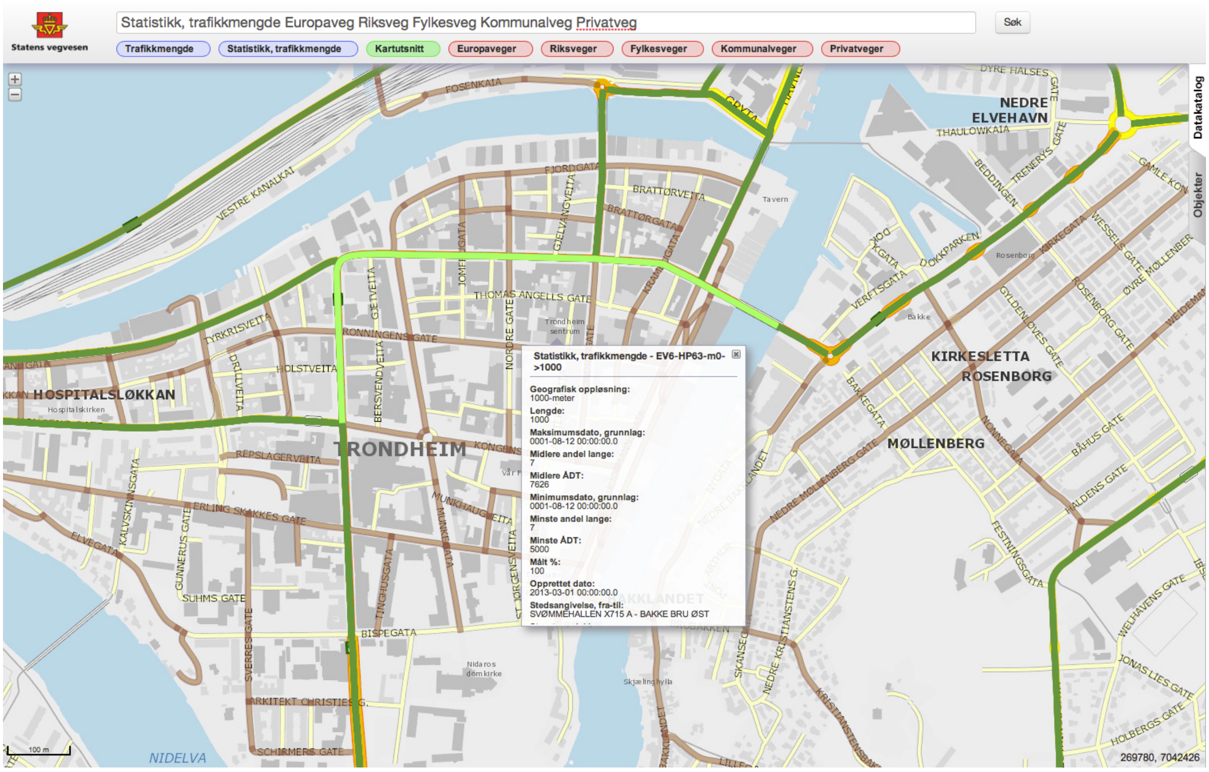


Figure 6.5: Exemplified map of categorized traffic **(Statens vegvesen 2013)**

Estimating category using ICA on traffic load:								
Diameter Age	→	Tensile strength Wall thickness	→	h_{Tmax}	→	YDT-L	→	RED/ YELLOW/ GREEN
Depth Length Road length	→	h_1 and h_2 L_{Tot} L_{T1} and L_{T2}	→	h_{Tmean}				

Table 6.2: ICA traffic load

6.2.3 Possible sources of Sulfuric acid

The last point of the ICA is to evaluate possible sources of sulfuric acid production. As mentioned above there are certain areas in the wastewater network where sulfuric acid attacks are more likely to be observed than others. These places are downstream of large pumping/pressure mains and/or large grease separators. As here the oxygen concentration can decrease to <1 mg/l (see chapter 4.2.2.1). To be able to have such low concentration of oxygen a minimum retention time and organic matter are required. As mentioned in chapter 4.2.2.1 the amount of sulfate and organic matter usually are not limiting for the production of sulfide in Norway. This means that the retention time in a pressure main could be an indirect measured factor for sulfide production and if we have sulfuric acid attacks or not.

The information that need to be provided to produce the ICA surface damage map:

- Register all pressure mains and grease separators
- Evaluate the wastewater source
    1. Normal Norwegian wastewater
    2. Normal wastewater with industrial sewage (high concentration of organic matter)
    3. Industrial sewage (high concentration of organic matter)
- Evaluate the retention time

ICA on sulfuric acid attacks:	
Possible sources	Pumping/pressure mains Grease separators
Wastewater source	Normal wastewater Normal with industrial sewage Industrial sewage
Retention time	T_{max}

Table 6.3: ICA Sulfuric acid attacks

6.3 Further Proposals for Adjusting Condition Assessment with DCA

The direct condition assessment is done with the information collected through inspection of each specific wastewater pipeline. Today the inspection is done using CCTV, but it is possible to develop new and better technics for DCA like GPR and other methods as sample testing of pipes in ground to measure residual wall thickness and tensile strength.

In this thesis it is focused on surface damages and cracking, and it is proposed a more comprehensive method for assessing these damages, by damage quantity thresholds.

Also it is proposed to do an adjustment of the ICA, due to observations of damages to make the condition assessment more complete and correct. Inspection of the wastewater pipes is done to adjust or confirm the anticipation made through the ICA.

If a condition assessment is already done with the NV150 guidelines and the calculated condition class does not match with the indirect condition assessment, is it advised to adjust the condition class due to ICA. Two examples of this were made in chapter 5.2.

6.3.1 DCA due to observation of surface damage

In this situation the wall thickness is of great importance and also the strength of the pipe. The preferred option would be to measure the new inside diameter and find the median wall thickness of the pipe. The wall thickness needs to be measured with a high-resolution device providing accuracy in terms of mm, because the critical cover height can vary in the range of meters with just a small decrease in wall thickness (see ***Figure 5.3***). It is preferred to know the tensile strength of that pipe, through a sample testing, but by knowing the pipe age we can assume a mean tensile strength found through lab tests (see ***Table 3.6***). With the new wall thickness the pipe tensile strength and a new max cover height (h_{max}) can be calculated with the same method as shown in chapter 5.1 (see ***Figure 5.3*** - ***Figure 5.6***). The pipe depth in the upstream and downstream manhole is measured (same way as in chapter 6.2.1) and a middle cover height along the pipe is calculated (h_{mean}).

The new fill load condition assessment class for the pipe is defined:

- $h_{mean} < h_{max}$ (+1m). The pipe is at least one meter above the critical depth and is marked with the color GREEN. Equal to NV150 condition class 1 and 2.
- $h_{mean} = h_{max}$ (+/-1m), The pipe is situated at the critical depth +/- one meter and is marked with the color YELLOW. Equal to NV150 condition class 3.
- $h_{mean} > h_{max}$ (-1m), The pipe is at least one meter below the critical depth and marked with the color RED. Equal to NV150 condition class 4 and 5.

<table>
<tr><th colspan="9">DCA surface damage: (Deterioration model base point)</th></tr>
<tr><td>Age
Measured diameter</td><td>→</td><td>Tensile strength
Wall thickness</td><td>→</td><td>h_{max}
h_{Tmax}</td><td rowspan="2">→</td><td rowspan="2">YDT-L</td><td rowspan="2">→</td><td rowspan="2">RED/
YELLOW/
GREEN
(Fill)
(Traff)</td></tr>
<tr><td>Depth
Length
Road length</td><td>→</td><td>h_1 and h_2
L_{Tot}
L_{T1} and L_{T2}</td><td>→</td><td>h_{mean}
h_{Tmean}</td></tr>
</table>

Table 6.4: DCA surface damage

If it is not possible to measure the inside diameter of the pipe, a manual evaluation of the surface damage and a manual adjustment of the ICA class have to be done. The area of decreased wall thickness is measured. The manual evaluation criteria is similar as in the NV150 (see 2.3.2):

- A*ggregates are visible* and the pipe is affected. An adjustment is done if the pipe is close to the lower category. If the pipe is in the green zone but close to the yellow zone (less than 0.5m) the pipe is adjusted into yellow zone. Same in the yellow zone when close to red, the pipe is moved down a zone. If the pipe is in the green zone and the cover height is fairly low no adjustment is needed.

- *Missing aggregates* and the pipe is severely affected. An adjustment down one or two zones can be necessary. A pipe in the green zone is adjusted to yellow and further adjustment is evaluated, regarding the amount of missing aggregates. A pipe in the yellow zone is adjusted to red zone.

6.3.2 DCA surface damage due to sulfuric acid attacks

In this situation where the wall thickness and strength are decreased, we also have to assess the deterioration speed. The deterioration process is also ongoing in the case presented above, but that is a stabile situation. If a sulfuric acid attack is ongoing, the deterioration speed could be higher than usual during summer season, and the situation is highly unstable. In this case we have to look for possible source of sulfuric acid, and assess the production level. If an ICA of sulfuric acid is done, and a possible area of such production is found, this must be monitored. By measuring the concentration of sulfuric acid, actions to reduce the attacks must be implemented.

Proposal how to categorize surface damage related to deterioration speed:

- GREEN zone, regular deterioration speed. No traces of sulfuric acid and there is no possible sources of production upstream, or the source is so far upstream that there are no traces.
  This case is evaluated to be a stable situation, the deterioration speed is not considered to be any higher than in other regular sewers. I would advice to monitor as regular pipes.
- YELLOW zone, increased deterioration speed. There are traces of sulfuric acid, but there is no possible source of production or it is far upstream.
  There have been sulfuric acid attacks and it can happen again. The deterioration speed could be higher than in other regular sewers, but the situation is still not critical. I would advise to monitor every 5 years.
- RED zone, high deterioration speed. There are traces of sulfuric acid and there is possible production upstream.
  This is a critical scenario and the deterioration speed could be high. The levels of sulfuric acid from the source needs to be monitored and actions to prevent further damages needs to be done. I would advise to monitor the pipe situation every 6-12 months, or continually monitoring during summer months (June-August).

DCA sulfuric acid attacks: (Deterioration model speed)			
Observed sulfuric acid attacks?	Yes / No	→	RED/ YELLOW/ GREEN
Upstream source?	Yes /No		

Table 6.5: DCA Sulfuric acid attacks

6.3.3 DCA due to observation of cracking

As explained in chapter 3.1 cracking is a result of a pipe being overloaded. In this case it is important to evaluate the cause of the crack. If it is assessed that the pipe could be overloaded through ICA, because of fill loads, and a crack is observed we must adjust the ICA category. Also if a pipe is cracking in a traffic load zone the ICA category must be adjusted. This is exemplified in chapter 5.2.

If the ICA indicates that there is no overload situation due to traffic or fill load, other reasons must be evaluated, for instance frost heave, soil settlement or poor installation work, which may be factors influencing the load situation (explained in chapter 3.3.3).
The stability and the driving factors of the cracking need to be evaluated to give advise on adjusting the ICA.

6.3.3.1 Longitudinal cracks

Too high force on a pipe is the most important driving force of longitudinal cracking. As explained in chapter 3.1 the strength capacity of the pipe is reached and it makes the pipe unstable. Small changes can cause a pipe to collapse. ***Table 3.1*** describes assessment and acceptance criteria for longitudinal cracking.

Proposal for categorization of longitudinal cracks related to crack sizes (assessment thresholds):

- < 0.15mm means that it is unlikely that the crack extends through the pipe wall, which means that it is no need for adjusting the ICA. The crack is in the GREEN category – no action is required. If such cracking is the most severe damage in a pipe the DCA would correspond to NV150 condition class of 1 and 2.
- 0.15-0.5 mm wide cracks must be monitored for stability. Cracks up to 0.5 mm is not considered to be critical for the pipe strength, but if the pipe is under strong forces the crack can grow. The crack is in the YELLOW category – monitoring required, every 2-3 year. If such cracking is the most severe damage in a pipe the DCA would correspond to NV150 condition class of 3.
- > 0.5 mm cracks can be critical for the pipe stability/strength. The effect and magnitude of fill and traffic loads must be evaluated. Frequent monitoring is needed and possible actions must be considered. The crack is in the RED category – actions and monitoring is required, every 6-12 months. If such cracking is the most severe damage in a pipe the DCA would correspond to NV150 condition class of 4.

6.3.3.2 Circumferential cracks

The driving factor for circumferential crack growth is soil settlement in the length direction of the pipe as explained in character 3.2. The reason for this could be the soil type, ground water level, dynamic forces etc. The soil settlements could be an ongoing process and the crack can be evolving. To be able to evaluate the settlement process ground penetrating radar can be a useful tool (see chapter 3.5) to investigate the soil surrounding the pipe. ***Table 3.2*** describes assessment and acceptance criteria for circumferential cracking.

Proposal for categorization of circumferential cracks related to crack sizes (assessment thresholds):

- < 0.5 mm means that the crack is not critical for the pipe strength. The crack is in the GREEN category – no action is required. If such cracking is the most severe damage in a pipe the DCA would correspond to NV150 condition class of 1 and 2.
- 0.5-1.0 mm wide cracks needs to be monitored. These cracks are not critical for the pipe, but are considered to be in the transition area. Some infiltration/exfiltration can happen. The crack is in the YELLOW category – monitoring is required, every 2-3 year. If such cracking is the most severe damage in a pipe the DCA would correspond to NV150 condition class of 3.
- > 1.0 mm are critical cracks and the potential of exfiltration and infiltration must be assessed. Exfiltration may lead to degradation of the trench bedding, which can increase the cracking. Is the crack wider than 2.0 mm it is likely to grow around the pipe. Frequent monitoring is needed and possible actions must be considered. The crack is in the RED category – actions and monitoring are required, every 6-12 months. If such cracking is the most severe damage in a pipe the DCA would correspond to NV150 condition class of 4.

<table>
<tr><th colspan="4">DCA cracking: (Deterioration model speed)</th></tr>
<tr><td>Observed cracking</td><td>Longitudinal
Circumferential</td><td rowspan="2">→</td><td rowspan="2">RED/
YELLOW/
GREEN</td></tr>
<tr><td>Measured</td><td>Crack width
(mm)</td></tr>
</table>

Table 6.6: DCA cracking

6.3.3.3 Crushed pipe

If a crushed or collapsed part of a storm or waste water pipe is observed through DCA, this is a situation where it is most likely that the pipe is not able to perform required service. The pipes ability to transport large amounts of storm water is highly degraded because of infiltration and/or if the crushed material reduces the pipe cross-section. Collapsed pipes will exfiltrate contaminated wastewater, degrading the quality of groundwater and recipients. If such damage is the most severe observations in a pipe the DCA would correspond to a NV150 condition class of 5. Immediate rehabilitation actions or replacement are advised.

7 Summary and Conclusion

7.1 Objective and Recommendations

The objective of the Master Thesis has been to provide an improved method for condition assessment, which will give a better correlation between *Condition class* and *actual Condition* of concrete pipes with cracking and/or surface damages. Additionally improvement of the characterization of cracking (SR) and surface damages (KO) was a sub goal.

Based on the findings described in my Thesis and my Specialization Project (Hauge 2012)*, I recommend that the Norwegian condition assessment method based on NV150, is revised.*

The revised condition assessment should focus on the severity of the damages based on measurable damage thresholds. According to NV150, cracks are not characterized and graded based on the measured crack width (see ***Table 2.2*** and ***Table 2.3***). There is no evaluation of the loads and forces influencing the pipe deterioration. My Specialization Project concluded that The Norwegian method gives too little weight to severe single damages when the pipe length exceeds 40 meters, and gives too high weight to distributed damages, as the condition score is very dependent of the damage length (Hauge 2012).

These facts show that adjusting the formula (***Equation 2.2***) for condition score calculation or the damage grade system (***Table 2.4***) in NV150 is not enough to improve the condition assessment method.

My recommendation is that a new way of assessing the pipe conditions, founded on research-based damage thresholds (see chapter 6.3) in the context of pipe deterioration processes (see chapter 6.2) is developed.

7.2 The Methodology Applied

I have studied the deterioration processes, loads and forces influencing concrete wastewater pipes (chapters 3 and 4). Through this work I have been able to establish damage thresholds for crack width and calculate critical strength in a pipe with decreased wall thickness (chapters 6.1 and 6.3).

Additionally I have developed a general method of assessing the loads and forces influencing a pipe without inspection, which is defined as indirect condition assessment (ICA). This method can be used to assess how critical the influencing loads and observed damages are, and can be used to correct the condition classes defined by NV150 if that result is uncertain.

Also I have developed a method of assessing the observations made through inspection defined as direct condition assessment (DCA). This method is an enhanced assessment of cracking and surface damages which are observed in a pipe. The observations and assessment of these inside the pipe are supporting the ICA.

A total assessment with both these two methods (ICA and DCA) is expressing the actual pipe condition better and more correctly than the NV150.

The table below summarizes the input parameters used in NV150, ICA and DCA

Factors	**NV150**	**ICA**	**DCA**
Pipe Data	X	X	X
External Loads		X	X
Acid Production		X	X
Visual Inspection	X		
Inspection with measurement			X
Condition Classification	**X**	**X**	**X**

Table 7.1: Summary of input to NV150, ICA and DCA

7.3 Summary of the Results

The calculation of fill load and pipe wall moment in chapter 5.1, tells us that poor installation work may have a significant influence on the pipe's ability to withstand the forces applied to it.

- The fill load (***Figure 5.1***) on pipes with poor side support has a 50% higher increase, compared to pipes with good side support pr. meter of cover height. The moment (***Figure 5.2***) on pipes with poor side support has a 100% higher increase, compared to pipes with good side support pr. meter of cover height.
- Installation work done in the period from 1950-1970 was often of reduced quality, which means that these pipelines are more vulnerable for cracking. Pipes from the period from 1945-59 ***(Figure 5.4)*** have a critical cover height of 4.3 m, and by a wall thickness reduction of 5 mm is the critical cover height is reduced to 2.8 m. This is because of the combination low tensile strength and poor installation work. Pipes with cover height lower than 0.6 m are vulnerable to traffic load.
- Pipes from 1920-44 (***Figure 5.3***) and 1960-69 (***Figure 5.5***) have almost the same critical cover height, 6.5 m and 6 m, respectively. By a wall thickness reduction of 5 mm it is reduced to 4.5 m and 4.3 m, respectively. Pipes from 1920-44 have lower tensile strength than those from 1960-69, but they have more often a fair quality of installation work. 1920-44 pipes are vulnerable to traffic load at cover heights lower than 0.5 m.
- Pipes from 1970-79 (***Figure 5.6***) have the highest critical cover height at 9 m and highest tensile strength, but the installation work is of varying quality.

7.4 Summary of the Revised Condition Assessment

7.4.1 Indirect condition assessment (ICA)

The ICA is assessing the most important factors creating cracks and reduced strength capacity in concrete pipes: filling loads, traffic loads and sulphuric acid attacks. The magnitudes of these forces are estimated and their criticality regarding the pipe strength capacity is categorised. This information leads to pointing out areas or specific pipes in the wastewater system that have the highest possibility and consequences of failure, i.e. which pipes have the highest failure risk, and need most frequent inspection so that eventually maintenance or rehabilitation can be planned in due time.

All this is done without inspection and visualized through a ICA GIS-tool. The method and the information needed to produce such a tool are summarized below.

ICA of fill loads on pipes can be made by using the information about the pipe to establish the critical cover height (h_{max}) and comparing it to the mean cover height (h_{mean}) (chapter 6.2.1).

Three levels are then defined:

- GREEN: $h_{mean} < h_{max}$ (+1m)
- YELLOW: $h_{mean} = h_{max}$ (+/-1m)
- RED: $h_{mean} > h_{max}$ (-1m)

The information needed to calculate this:

- Age of the pipe – assumed wall thickness and tensile strength
- Depth of upstream manhole (h_1)
- Depth of downstream manhole (h_2)

ICA of traffic loads on pipes can be made by using traffic amount information (chapter 6.2.2). The amount of traffic (YDT-L) could for instance be categorized in these three levels:

- GREEN, light traffic (<100 YDT-L)
- YELLOW, intermediate traffic (600-100 YDT-L)
- RED, heavy traffic (>600 YDT-L)

The retention time in a pressure main can be an indirect measured factor for sulfide production and whether sulfuric acid attacks the pipelines downstream. I recommend using the following procedure to evaluate the possibility for sulfuric acid production:

- Register all pressure mains and grease separators
- Evaluate the wastewater quality regarding organic matter:
    1. Normal wastewater
    2. Normal wastewater with industrial sewage (high concentration of organic matter)
    3. Industrial sewage (high concentration of organic matter)
- Evaluate the retention time

Using ICA to adjust and correct NV150 condition classes:
When ICA is done for an area, the result might be used to adjust the NV150 pipe condition class for the pipes.

ICA evaluates how critical the load situation and the deterioration speed are. If the NV150 condition class is not satisfactory this can be corrected as exemplified in chapter 5.2.1 and 5.2.2.

7.4.2 Direct condition assessment (DCA)

DCA is an enhanced assessment of cracking and surface damages which are observed in a pipe. This method characterise the pipes residual strength and stability based on measurable damage thresholds. The observations and assessment of the damages inside the pipe are confirming the ICA assumptions or correcting them. Together DCA and ICA are resulting in a thorough condition assessment.

DCA due to observed surface damage (see chapter 6.3.1).
By measuring the inside diameter and the wall thickness the DCA fill load classes for the pipe can be defined:

- GREEN: $h_{mean} < h_{max}$ (+1m). Equal to NV150 condition class 1 or 2
- YELLOW: $h_{mean} = h_{max}$ (+/-1m). Equal to NV150 condition class 3
- RED: $h_{mean} > h_{max}$ (-1m). Equal to NV150 condition class 4 or 5

DCA of surface damage due to sulfuric acid (see chapter 6.3.2).
In a case of sulfuric acid attacks, the deterioration speed is highly unstable. A separate evaluation is necessary:

- GREEN: no traces of Sulfuric acid attacks. Equal to NV150 condition class of 3 or lower
- YELLOW: lower degradation speed further from the source. Equal to NV150 condition class 4
- RED: high degradation speed close to the source. Equal to NV150 condition class 5

In a case of Sulfuric acid attacks it is advised to monitor the situation.

DCA due to observed cracking (see chapter 6.3.3).
Longitudinal cracking:

- GREEN: crack is < 0.15mm – no action is required. Equal to NV150 condition class of 1 or 2
- YELLOW: crack is 0.15-0.5 mm – monitoring required, every 2-3 years. Equal to NV150 condition class of 3
- RED: crack is > 0.5 mm – actions and monitoring are required, every 6-12 months. Equal to NV150 condition class of 4

Circumferential cracking:

- GREEN: crack is < 0.5 mm – no action is required. Equal to NV150 condition class of 1 or 2
- YELLOW: crack is 0.5-1.0 mm – monitoring is required, every 2-3 year. Equal to NV150 condition class of 3
- RED: crack is > 1.0 mm – actions and monitoring are required, every 6-12 months. Equal to NV150 condition class of 4

Crushed pipe:
If a crushed or collapsed part of a storm or wastewater pipe is observed through DCA, is this a situation where most likely the pipe is not able to perform required service. If such damages are the most severe observations in a pipe, the DCA would correspond to a NV150 condition class of 5. Immediate rehabilitation actions or replacement is advised.

7.4.3 Proposed condition classification workflow

The present *condition classification workflow* based on NV150 is illustrated in chapter 2.4.1 (***Figure 2.1***). Based on my work and development of the ICA and DCA methods, a revised and recommended workflow is illustrated below (***Figure 7.1***). Assessment of Condition classification through this workflow will be based on facts about the pipes and installation combined with measured values, which will produce a more reliable assessment.

This tool will be an important part of planning of maintenance and replacement of sewage infrastructure in Norway and secure efficient use of budgets.

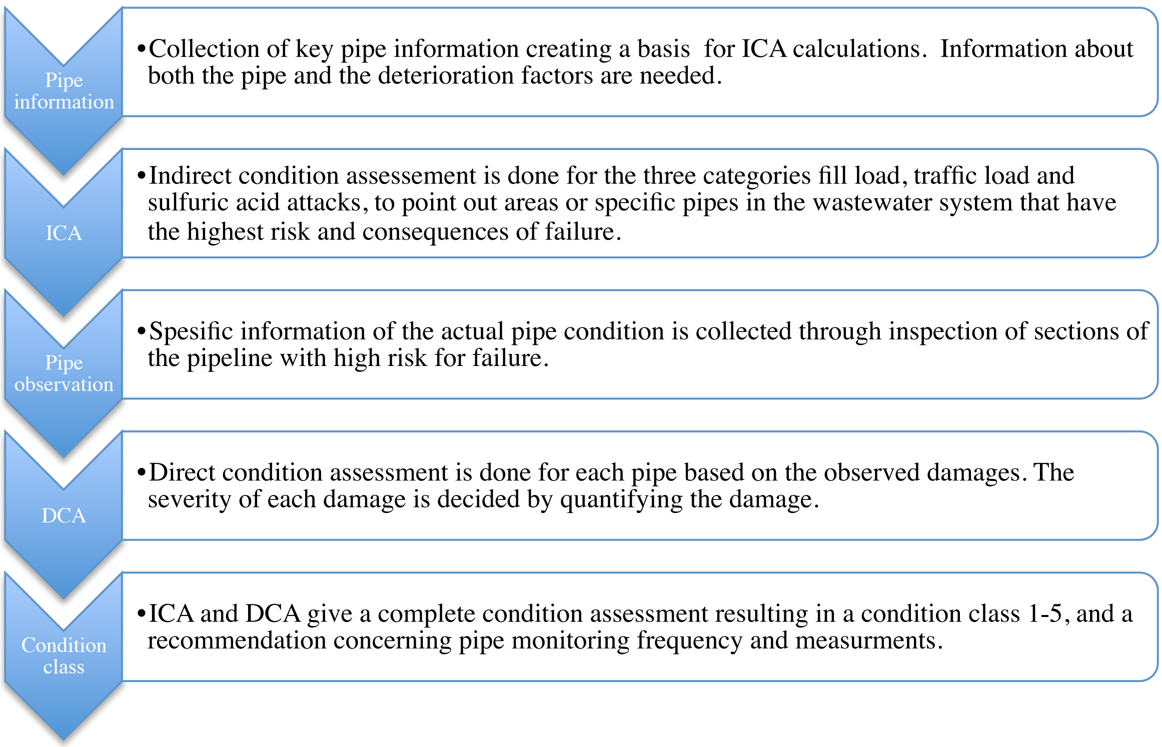


Figure 7.1: The revised condition classification method

7.5 Further Research Proposals

To improve both the ICA and DCA further research is needed, and below some research topics are listed based on the findings in this thesis.

- It is important to use available tools like Norwegian Road Data Base (NVDB, see chapter 6.2.2) to develop the ICA tool for traffic load assessment. The actual correlation between traffic load influence and YDT-L must be studied more closely. Also critical cover heights for all pipe sizes and ages must be calculated for indirect condition assessment of fill loads.

- The use and implementation of new technics for trench and pipe bedding inspection, like geophysical methods, must be studied. Information about the bedding and sidefilling support of a pipe is very important to estimate the capacity of the pipe to withstand loading and forces.

- It is important to develop accurate equipment to measure cracking and internal diameter (wall thickness). This information is needed for the development of critical fill load assessment as motioned above, especially when the critical cover height decreases when the wall thickness is reduced.

- It is important to study the production of sulfide production in Norwegian sewer environment to determine critical retention time in pumping and pressure mains. Inspection robots can be equipped with gas sensors. Also it is useful to register all possible sources, and evaluate their activity and sulfide production. This will give us knowledge to understand the correlation between pipes affected by sulfuric acid attacks and the possible cause.

References

3D-Radar. *3D-Radar, A new dimension for Ground Penetrating Radar.* 2009. http://www.3d-radar.com/military/technology-2 (accessed February 2013).

Bernhus, C., Oslo kommune VAV, O. Nilssen, Trondheim kommune Byteknikk, G. Mosevoll, Skien kommune, and A. Hansen, VA teknikk as. *R150 - Dataflyt-Klassifisering av avløpsledninger.* Guideline, Hamar: Norsk Vann, 2007.

CARE-S. *Computer Aided Rehbilitation of Sewer Networks.* 2008. http://www.sintef.no/Projectweb/CARE-S/ (accessed January 2013).

Cassidy, N. *A comparison between GPR and Ultrasonic Techniques for Void Detection beneath Reinforced Concrete Sections.* Paper, Keele University, Leicester, UK: Near Surface 2011 - 17th European Meeting of Environmental and Engineering Geophysics, 2011.

CCPA. "Circumferential Cracking - Engineering Guideline." *Concrete Pipe Association of Australasia - Technical Publications.* 2011. http://www.cpaa.asn.au/images/publications/engineering_guidelines/circumferential%20cracking.pdf (accessed February 2013).

CCPA. "Longitudinal Carcking - Engineering Guideline." *Concrete Pipe Association of Australasia - Technical Publications.* 2011. http://www.cpaa.asn.au/images/publications/engineering_guidelines/longitudinal%20cracking.pdf (accessed February 2013).

CEN. *Investigation and assessment of drain and sewer systems outside buildings, Part 2: Visual inspection coding system.* Standard, Brussels: European Committee for Standardization (CEN), 2011.

DIVA. "DIVA, Project description." 2011.

Donckels, B.M.R., L. Verhoest, C. Thoeye, G. DeGueldre, and B. VanDeSteene. *Hydrogen sulphide related problems in sewers: examples from practice.* IWA, 2010.

Ekbäck, D., and B. Hedman. *RÖRBOK.* Stockholm: AB Svensk Byggtjänst, 1983.

Hauge, P. "Condition assessment for wastewater pipes - case study of Oslo." Specialization Project, Department of Hyraulic and Environmental Engineering, NTNU, Trondheim, 2012.

Haugen, H.J., VA teknikk AS, S. Nilo, Oslo kommune-VAV, E. Bøhleng, and NorVar BA. *R145 - Inspeksjonsmanual for avløpssystemer. Del1 - Ledninger.* Guideline, Hamar: NorskVann, 2005.

Liu, Y.W., T. Yen, and T.H. Hsu. *Abrasion erosion of concrete by water-borne sand.* Cement and Concrete Research, 2006.

Moser, A.P. *Buride Pipe Design.* Vol. 2. 3 vols. Logan, Utah: McGraw-Hill, 2001.

Nielsen, P.H., K. Raunkjær, and T. Hvitved-Jacobsen. *Sulfide Production and Wastewater Quality in Pressure Mains.* Environmental Engineering Labratory, Aalborg University, Aalborg: Elsevier Science Ltd, 1998.

Søpler, B. *Erosjon og korrosjon av betongrør. Del 1, Variasjon i betongrørs erosjonsmotstand.* SINTEF, Trondheim: SINTEF, 1976.

Schei, I., and T. Tekle. *Slitasje av avløpsrør.* Prosjektkomiteen for rensing av avløpsvann, SFT, Oslo: SFT, 1976.

Sægrov, S. *Ledningsteknologi Kompendium.* Trondheim: Institutt for vann- og miljøteknikk, NTNU, 2012.

Sægrov, S. *Tilstand og tilstandsendring for betongavløpsledninger.* Doktor Ingeniøravhandling, Institutt for vassbygging, Norges Tekniske Høgskole, Trondheim: NTH, 1992.

Sprangler, M.G., and R.L. Handy. *Soil Engineering.* New York: Harper & Row, 1982.

SSB. *KOSTRA: municipal waste services 2011.* June 21st, 2012. http://www.ssb.no/english/subjects/01/04/20/var_kostra_en/ (accessed January 2013).

Standard Norge. *NS 3121:2003 Rør og rørdeler av betong - Uarmert, stålfiberarmert og armert betong.* Standard, Standard Norge, 2003.

Statens vegvesen. *Nasjonal vegdatabank.* Edited by Statens vegvesen. 2013. https://www.vegvesen.no/vegkart/vegkart/#!/datakatalog/objekttyper/798 (accessed 2013).

Trott, J.J., and O.C. Young. *Buried Rigid Pipes, Structural Design of Pipelines.* Wokingham: Elsevier Applied Science Publichers LTD, 1984.

U.S. Envitonmental Protection Agency. *Process Design Manual for Sulfide Control in Sanitary Sewerage Systems.* Technology Transfer Office of EPA, 1974.

Ugarelli, R. "Condition Assessment (CA) of buried pipes and role of CA in Asset Management of urbane water system." *NTNU Lecture.* Trondheim: NTNU / Sintef, 2012.

Appendix 1

Sprukket rør, SR

Definisjon

Sprekk: Det er synlig sprekk på rørveggen, men røret har ikke endret form og alle rørbiter sitter på plass.

Brudd: Rørbiter har løsnet, men mangler ikke.

Kollaps: Fullstendig konstruksjonsmessig sammenbrudd.

Gradering

1. Overflatesprekker.
   Avskalling eller teglstein er delvis løse.
2. Sprekkene er åpne.
   Løse teglstein.
3. Rørbiter har løsnet eller mangler.
   Teglstein mangler fra sin opprinnelige posisjon.
4. Kollaps.

Karakterisering

- Langsgående.
- Tangentiell.
- Kompleks.
- Avskalling.

Inspeksjonskrav

Tangentiell plassering skal rapporteres (urviser-referanse).

Målte verdier

For grad 1 og 2:
Hvis sprekkbredde måles, skal verdien rapporteres.
For grad 3 og 4:
Hvis lengden på bruddet / kollapsen måles, skal verdien rapporteres.

Retningslinjer

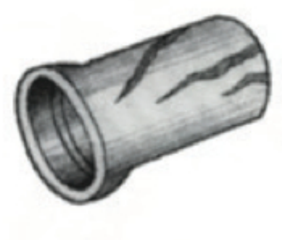
Sprekkbredde skal måles i mm.

Grad	Sprukket rør, SR / foto-eksempler

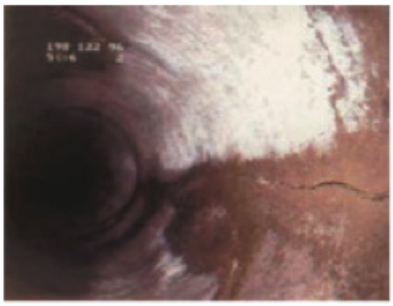
1

Overflatesprekker.

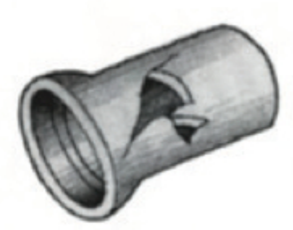
2



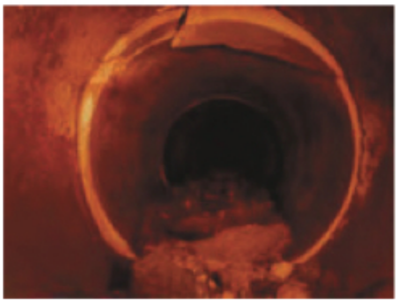
Sprekkene er åpne.



3



Rørbiter har løsnet eller mangler.



4

Kollaps.

Korrosjon / Slitasje, KO

Definisjon

Røroverflaten har blitt skadet av kjemiske stoffer, korrosjon eller slitasje.

Gradering

1. Økende ruhet (rørveggen er noe påvirket).

2. Synlig tilslag (rørveggen er betydelig påvirket).
   - I betongrør er tilslagsmaterialet tydelig blottlagt.
   - I lerrør mangler glassuren.
   - I støpejernsrør et det begynnende rustangrep.
   - I teglsteinskulverter er overflaten angrepet og mørtelen delvis borte.

3. Manglende tilslag eller synlig armering (rørveggen er sterkt påvirket).
   - I betongrør er tilslagsmaterialet delvis borte eller armering synlig.
   - I støpejernsrør er det kraftig rustangrep.
   - I teglsteinskulverter er overflaten porøs og mørtelen mangler.

4. Rørveggen mangler.

Karakterisering

Inspeksjonskrav

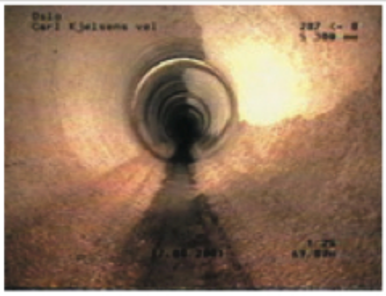
Tangentiell plassering skal rapporteres (urviser-referanse).

Målte verdier

Retningslinjer

Grad	Korrosjon / Slitasje, KO / foto-eksempler

1

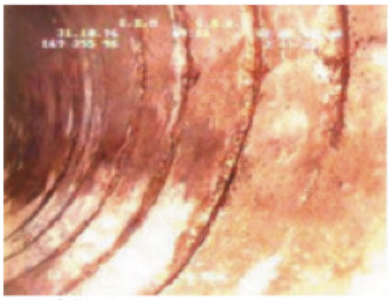


Økende ruhet (rørveggen er noe påvirket).

2

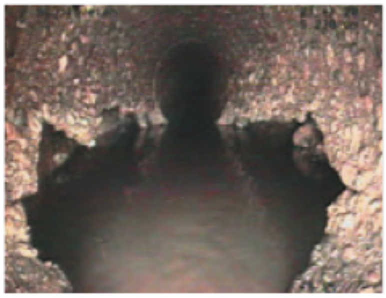
Synlig tilslag (rørveggen er betydelig påvirket).

3



Manglende tilslag eller synlig armering (rørveggen er sterkt påvirket).

4



Rørveggen mangler.

Appendix 2

Concrete pipe production demands.

Wall thickness corresponding to previous standards [mm] (Sægrov 1992):

DN:	NKIF 1909	NKIF 1919	NKIF 1928	NKIF 1936	NS 460 1948	NS 461 1966	NS 3027 1970	NS 3027 1975
100	20	20	As 1919	22	Not spec.	19	24	24
125	22	22		22		21	25	25
150	24	24		24		23	28	28
200						28	32	32
225	26	26		26		29		
250						30	37	37
300	30	30		30		32	44	44
375	40	37		37				
400						39	50	50
450	45	45						
500						49	60	60
525		47		47				
600		50		50		59	65	65
800						79		
1000						80		

Table of crush load by standard demands [kN/m] (Sægrov 1992):

DN:	NKIF 1909	NKIF 1919	NKIF 1928	NKIF 1936	NS 460 1948	NS 461 1966	NS 3027 1970	NS 3027 1975
100	16.4	16.4	16.4	23.4	22.7	20	36	40
125	16.4	16.4	16.4	23.4	22.7	20	36	40
150	19.7	19.7	19.7	23.4	22.7	22	40	40
200			19.7			25	40	45
225	18.4	18.4	18.4	23.4	23.0	28		
250						29	43	50
300	18.4	18.4	18.4	23.4	23.0	32	46	50
375		23.6	23.6	23.4	25.0			
400						35	53	60
450		26.2	26.2	26.0	29.0			
500						37	61	60
525		25.0	25.0	26.0	34.0			
600		30.0	30.0	26.0	38.0	39	70	70

Appendix 3

Theoretical calculations excel sheet. Also a digital Appendix.

Earthfill load on rigid concrete pipe as a function of depth

Case 1: *With side support, compressed sidefillings:*

Wc = γ(Bc^2)(1.157(H/Bc)-0.477)

Pipewall moment

M = 0.059*(Wc+Wt)*Bc

Pipe wall stress/strain

σ = 6M/(s^2)

$$\sigma = \frac{6M}{s^2}$$

Case 2: *With out side support, poorly compressed sidefillings:*

Wc = γ(Bc^2)(1.642(H/Bc)-0.690)

M = 0.079*(Wc+Wt)*Bc

γ (gravel) =	2000	kg/m3
	20	kN/m3

Max stress (SS Appendix 4)

σmax	[kPa]	dy [m]	S(new) [mm]
1920-44	7320.00	0.36	30
1945-59	9350.00	0.36	30
1960-69	11780.00	0.364	32
1970-79	12900.00	0.388	44

Above, wall thickness when the pipe was new

H [m]	Wc [kN/m]						Wt [kN/m]			M [kNm/m]					
	1920-59		1960-69		1970-79		1920-69	1970-79		1920-59		1960-69		1970-79	
	With SS	*With out SS*	*With SS*	*With out SS*	*With SS*	*With out SS*	*Bc:360-370*	*Bc:380-390*	*Bc:480*	*With SS*	*With out SS*	*With SS*	*With out SS*	*With SS*	*With out SS*
0.20	0.4	0.6	0.4	0.6	0.4	0.6	39.4	51.5	46.7	0.85	1.14	0.85	1.15	1.19	1.60
0.30	1.3	1.8	1.3	1.8	1.3	1.8	36.2	47.5	43.5	0.80	1.08	0.80	1.09	1.12	1.51
0.40	2.1	2.9	2.1	3.0	2.1	3.0	34.8	43.3	40.0	0.78	1.07	0.79	1.08	1.04	1.42
0.50	2.9	4.1	2.9	4.1	2.9	4.1	31.4	39.8	36.4	0.73	1.01	0.74	1.02	0.98	1.35
0.60	3.8	5.3	3.8	5.3	3.8	5.3	28.7	36.2	32.6	0.69	0.97	0.70	0.98	0.91	1.27
0.70	4.6	6.5	4.6	6.5	4.6	6.5	25.8	32.3	27.6	0.65	0.92	0.65	0.93	0.84	1.19
0.80	5.4	7.7	5.5	7.7	5.5	7.7	21.5	25.8	23.5	0.57	0.83	0.58	0.84	0.72	1.03
0.90	6.3	8.9	6.3	8.9	6.3	8.9	18.3	23.9	19.3	0.52	0.77	0.53	0.78	0.69	1.01
1.00	7.1	10.0	7.2	10.1	7.2	10.1	16.2	20.7	16.1	0.49	0.74	0.50	0.76	0.64	0.94
1.10	7.9	11.2	8.0	11.3	8.0	11.3	13.9	18.5	13.4	0.46	0.71	0.47	0.73	0.61	0.91
1.20	8.8	12.4	8.8	12.5	8.8	12.5	12.9	16.3	11.2	0.46	0.72	0.47	0.73	0.58	0.88
1.30	9.6	13.6	9.7	13.7	9.7	13.7	11.7	14.0	9.5	0.45	0.72	0.46	0.73	0.54	0.85
1.40	10.4	14.8	10.5	14.9	10.5	14.9	10.6	12.6	8.4	0.45	0.72	0.45	0.73	0.53	0.84
1.50	11.3	15.9	11.4	16.1	11.4	16.1	9.4	11.1	7.3	0.44	0.72	0.45	0.73	0.51	0.83
1.60	12.1	17.1	12.2	17.3	12.2	17.3	8.4	9.9	6.1	0.44	0.73	0.44	0.74	0.51	0.83
1.70	12.9	18.3	13.1	18.5	13.1	18.5	8.1	9.3	5.7	0.45	0.75	0.45	0.76	0.51	0.85
1.80	13.8	19.5	13.9	19.7	13.9	19.7	7.7	8.2	5.3	0.46	0.77	0.46	0.79	0.51	0.86
1.90	14.6	20.7	14.7	20.9	14.7	20.9	7.3	7.9	5.1	0.47	0.80	0.47	0.81	0.52	0.88
2.00	15.4	21.9	15.6	22.1	15.6	22.1	6.5	7.1	5.0	0.46	0.81	0.47	0.82	0.52	0.89
2.10	16.3	23.0	16.4	23.3	16.4	23.3	6.1	6.1	4.8	0.48	0.83	0.48	0.85	0.52	0.90
2.20	17.1	24.2	17.3	24.5	17.3	24.5	3.8	3.8	4.4	0.44	0.80	0.45	0.81	0.48	0.87
2.30	17.9	25.4	18.1	25.7	18.1	25.7	3.1	3.1	4.1	0.45	0.81	0.46	0.83	0.49	0.88
2.40	18.8	26.6	19.0	26.9	19.0	26.9	2.8	2.8	3.8	0.46	0.84	0.47	0.85	0.50	0.91
2.50	19.6	27.8	19.8	28.1	19.8	28.1	2.5	2.5	3.6	0.47	0.86	0.48	0.88	0.51	0.94
2.60	20.4	28.9	20.6	29.3	20.6	29.3	2.4	2.4	3.4	0.48	0.89	0.49	0.91	0.53	0.97
2.70	21.3	30.1	21.5	30.4	21.5	30.4	2.2	2.2	3.1	0.50	0.92	0.51	0.94	0.54	1.00
2.80	22.1	31.3	22.3	31.6	22.3	31.6	2.1	2.1	2.8	0.51	0.95	0.52	0.97	0.56	1.03
2.90	22.9	32.5	23.2	32.8	23.2	32.8	1.9	1.9	2.5	0.53	0.98	0.54	1.00	0.57	1.07
3.00	23.8	33.7	24.0	34.0	24.0	34.0	1.8	1.8	2.2	0.54	1.01	0.55	1.03	0.59	1.10
3.10	24.6	34.9	24.8	35.2	24.8	35.2				0.52	0.99	0.53	1.01	0.57	1.08
3.20	25.4	36.0	25.7	36.4	25.7	36.4				0.54	1.03	0.55	1.05	0.59	1.12
3.30	26.3	37.2	26.5	37.6	26.5	37.6				0.56	1.06	0.57	1.08	0.61	1.15
3.40	27.1	38.4	27.4	38.8	27.4	38.8				0.58	1.09	0.59	1.12	0.63	1.19
3.50	27.9	39.6	28.2	40.0	28.2	40.0				0.59	1.13	0.61	1.15	0.65	1.23
3.60	28.8	40.8	29.1	41.2	29.1	41.2				0.61	1.16	0.62	1.18	0.67	1.26
3.70	29.6	42.0	29.9	42.4	29.9	42.4				0.63	1.19	0.64	1.22	0.68	1.30
3.80	30.4	43.1	30.7	43.6	30.7	43.6				0.65	1.23	0.66	1.25	0.70	1.34
3.90	31.3	44.3	31.6	44.8	31.6	44.8				0.66	1.26	0.68	1.29	0.72	1.37
4.00	32.1	45.5	32.4	46.0	32.4	46.0				0.68	1.29	0.70	1.32	0.74	1.41
4.10	32.9	46.7	33.3	47.2	33.3	47.2				0.70	1.33	0.71	1.36	0.76	1.45
4.20	33.8	47.9	34.1	48.4	34.1	48.4				0.72	1.36	0.73	1.39	0.78	1.48
4.30	34.6	49.0	35.0	49.6	35.0	49.6				0.73	1.39	0.75	1.43	0.80	1.52
4.40	35.4	50.2	35.8	50.8	35.8	50.8				0.75	1.43	0.77	1.46	0.82	1.56
4.50	36.3	51.4	36.6	52.0	36.6	52.0				0.77	1.46	0.79	1.49	0.84	1.59
4.60	37.1	52.6	37.5	53.2	37.5	53.2				0.79	1.50	0.80	1.53	0.86	1.63
4.70	37.9	53.8	38.3	54.4	38.3	54.4				0.81	1.53	0.82	1.56	0.88	1.67
4.80	38.7	55.0	39.2	55.5	39.2	55.5				0.82	1.56	0.84	1.60	0.90	1.70
4.90	39.6	56.1	40.0	56.7	40.0	56.7				0.84	1.60	0.86	1.63	0.92	1.74
5.00	40.4	57.3	40.9	57.9	40.9	57.9				0.86	1.63	0.88	1.67	0.94	1.78
5.10	41.2	58.5	41.7	59.1	41.7	59.1				0.88	1.66	0.90	1.70	0.95	1.81
5.20	42.1	59.7	42.5	60.3	42.5	60.3				0.89	1.70	0.91	1.73	0.97	1.85
5.30	42.9	60.9	43.4	61.5	43.4	61.5				0.91	1.73	0.93	1.77	0.99	1.89
5.40	43.7	62.1	44.2	62.7	44.2	62.7				0.93	1.76	0.95	1.80	1.01	1.92
5.50	44.6	63.2	45.1	63.9	45.1	63.9				0.95	1.80	0.97	1.84	1.03	1.96
5.60	45.4	64.4	45.9	65.1	45.9	65.1				0.96	1.83	0.99	1.87	1.05	2.00
5.70	46.2	65.6	46.7	66.3	46.7	66.3				0.98	1.87	1.00	1.91	1.07	2.03
5.80	47.1	66.8	47.6	67.5	47.6	67.5				1.00	1.90	1.02	1.94	1.09	2.07
5.90	47.9	68.0	48.4	68.7	48.4	68.7				1.02	1.93	1.04	1.98	1.11	2.11
6.00	48.7	69.1	49.3	69.9	49.3	69.9				1.04	1.97	1.06	2.01	1.13	2.14
6.10	49.6	70.3	50.1	71.1	50.1	71.1				1.05	2.00	1.08	2.04	1.15	2.18
6.20	50.4	71.5	51.0	72.3	51.0	72.3				1.07	2.03	1.09	2.08	1.17	2.22
6.30	51.2	72.7	51.8	73.5	51.8	73.5				1.09	2.07	1.11	2.11	1.19	2.25
6.40	52.1	73.9	52.6	74.7	52.6	74.7				1.11	2.10	1.13	2.15	1.21	2.29
6.50	52.9	75.1	53.5	75.9	53.5	75.9				1.12	2.13	1.15	2.18	1.22	2.33
6.60	53.7	76.2	54.3	77.1	54.3	77.1				1.14	2.17	1.17	2.22	1.24	2.36
6.70	54.6	77.4	55.2	78.3	55.2	78.3				1.16	2.20	1.18	2.25	1.26	2.40
6.80	55.4	78.6	56.0	79.5	56.0	79.5				1.18	2.24	1.20	2.28	1.28	2.44
6.90	56.2	79.8	56.9	80.7	56.9	80.7				1.19	2.27	1.22	2.32	1.30	2.47
7.00	57.1	81.0	57.7	81.8	57.7	81.8				1.21	2.30	1.24	2.35	1.32	2.51
7.10	57.9	82.2	58.5	83.0	58.5	83.0				1.23	2.34	1.26	2.39	1.34	2.55
7.20	58.7	83.3	59.4	84.2	59.4	84.2				1.25	2.37	1.28	2.42	1.36	2.58
7.30	59.6	84.5	60.2	85.4	60.2	85.4				1.27	2.40	1.29	2.46	1.38	2.62
7.40	60.4	85.7	61.1	86.6	61.1	86.6				1.28	2.44	1.31	2.49	1.40	2.66
7.50	61.2	86.9	61.9	87.8	61.9	87.8				1.30	2.47	1.33	2.53	1.42	2.69
7.60	62.1	88.1	62.8	89.0	62.8	89.0				1.32	2.50	1.35	2.56	1.44	2.73
7.70	62.9	89.2	63.6	90.2	63.6	90.2				1.34	2.54	1.37	2.59	1.46	2.77
7.80	63.7	90.4	64.4	91.4	64.4	91.4				1.35	2.57	1.38	2.63	1.48	2.80
7.90	64.6	91.6	65.3	92.6	65.3	92.6				1.37	2.61	1.40	2.66	1.49	2.84
8.00	65.4	92.8	66.1	93.8	66.1	93.8				1.39	2.64	1.42	2.70	1.51	2.88
8.10	66.2	94.0	67.0	95.0	67.0	95.0				1.41	2.67	1.44	2.73	1.53	2.91
8.20	67.1	95.2	67.8	96.2	67.8	96.2				1.42	2.71	1.46	2.77	1.55	2.95
8.30	67.9	96.3	68.6	97.4	68.6	97.4				1.44	2.74	1.47	2.80	1.57	2.99
8.40	68.7	97.5	69.5	98.6	69.5	98.6				1.46	2.77	1.49	2.83	1.59	3.02
8.50	69.6	98.7	70.3	99.8	70.3	99.8				1.48	2.81	1.51	2.87	1.61	3.06
8.60	70.4	99.9	71.2	101.0	71.2	101.0				1.50	2.84	1.53	2.90	1.63	3.10
8.70	71.2	101.1	72.0	102.2	72.0	102.2				1.51	2.87	1.55	2.94	1.65	3.13
8.80	72.1	102.2	72.9	103.4	72.9	103.4				1.53	2.91	1.56	2.97	1.67	3.17
8.90	72.9	103.4	73.7	104.6	73.7	104.6				1.55	2.94	1.58	3.01	1.69	3.20
9.00	73.7	104.6	74.5	105.8	74.5	105.8				1.57	2.98	1.60	3.04	1.71	3.24

Min. wall thickness: [mm]							
1920-44		1945-59		1960-69		1970-79	
With SS	*With out SS*	*With SS*	*With out SS*	*With SS*	*With out SS*	*With SS*	*With out SS*
26	31	23	27	21	24	24	27
26	30	23	26	20	24	23	27
25	30	22	26	20	24	22	26
24	29	22	25	19	23	21	25
24	28	21	25	19	22	21	24
23	27	20	24	18	22	20	24
22	26	19	23	17	21	18	22
21	25	18	22	16	20	18	22
20	25	18	22	16	20	17	21
19	24	17	21	15	19	17	21
19	24	17	21	15	19	16	20
19	24	17	21	15	19	16	20
19	24	17	22	15	19	16	20
19	24	17	22	15	19	15	20
19	24	17	22	15	19	15	20
19	25	17	22	15	20	15	20
19	25	17	22	15	20	15	20
20	26	17	23	16	20	16	20
20	26	17	23	16	20	16	20
20	26	17	23	16	21	15	20
19	26	17	23	15	20	15	20
19	26	17	23	15	21	15	20
19	26	17	23	15	21	15	21
20	27	17	24	16	21	15	21
20	27	18	24	16	22	16	21
20	27	18	24	16	22	16	22
21	28	18	25	16	22	16	22
21	28	18	25	17	23	16	22
21	29	19	25	17	23	17	23
21	29	18	25	16	23	16	22
21	29	19	26	17	23	17	23
21	29	19	26	17	23	17	23
22	30	19	26	17	24	17	24
22	30	20	27	18	24	17	24
22	31	20	27	18	25	18	24
23	31	20	28	18	25	18	25
23	32	20	28	18	25	18	25
23	32	21	28	19	26	18	25
24	33	21	29	19	26	19	26
24	33	21	29	19	26	19	26
24	33	21	30	19	27	19	26
25	34	22	30	20	27	19	27
25	34	22	30	20	27	20	27
25	35	22	31	20	28	20	27
25	35	22	31	20	28	20	28
26	35	23	31	20	28	20	28
26	36	23	32	21	29	20	28
26	36	23	32	21	29	21	28
27	37	23	32	21	29	21	29

27	37	24	33	21	29	21	29
27	37	24	33	22	30	21	29
27	38	24	33	22	30	21	30
28	38	24	34	22	30	22	30
28	38	25	34	22	31	22	30
28	39	25	34	22	31	22	30
28	39	25	35	23	31	22	31
29	39	25	35	23	31	23	31
29	40	26	35	23	32	23	31
29	40	26	36	23	32	23	32
29	40	26	36	23	32	23	32
30	41	26	36	24	33	23	32
30	41	26	36	24	33	23	32
30	41	27	37	24	33	24	33
30	42	27	37	24	33	24	33
31	42	27	37	24	34	24	33
31	42	27	38	25	34	24	33
31	43	27	38	25	34	24	34
31	43	28	38	25	34	25	34
32	43	28	38	25	35	25	34
32	44	28	39	25	35	25	34
32	44	28	39	25	35	25	35
32	44	28	39	26	35	25	35
32	45	29	40	26	36	25	35
33	45	29	40	26	36	26	35
33	45	29	40	26	36	26	36
33	46	29	40	26	36	26	36
33	46	29	41	27	37	26	36
34	46	30	41	27	37	26	36
34	47	30	41	27	37	27	37
34	47	30	41	27	37	27	37
34	47	30	42	27	38	27	37
34	47	30	42	27	38	27	37
35	48	31	42	28	38	27	37
35	48	31	42	28	38	27	38
35	48	31	43	28	38	28	38
35	49	31	43	28	39	28	38
35	49	31	43	28	39	28	38
36	49	32	43	28	39	28	39
36	49	32	44	29	39	28	39