Prevention of pipeline floatation during dredge-based backfilling

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Preface

This thesis report is the final product to obtain the Master of Science degree in Geo-engineering at Delft University of Technology. The study is carried out in cooperation with the department Hydronamic of Royal Boskalis Westminster NV.

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Summary

Underwater pipelines have become a vital part of modern civilisation. Transport by pipeline is relatively inflexible compared to other means of transport, however it consumes less energy. Most offshore pipelines carry oil or gas, but they can also transport water or other fluids. More and more oil and gas is produced from offshore fields. The product has to be carried onshore and that is usually done by pipeline. Pipelines can be laid uncovered on the seabed, or can be embedded in the seafloor when stabilisation or protection is required. Pipelines are primarily covered when they are located in shallow waters. The material used to refill an excavated trench is called backfill. The typical material used as artificial backfill is coarse granular soil (rock, gravel, or coarse sand). When there is no suitable backfill material available close to the pipeline route, then material has to be transported from another location.

A subsea pipeline should remain stable during its entire lifetime. Normal engineering practice focuses on the operational phase, while the installation phase is almost neglected. However, this study focuses on the installation phase and particularly on dredge-based backfilling. Dredge-based backfilling means using the suction pipe of a Trailing Suction Hopper Dredger (TSHD) as discharge pipe. It is not always clear if a particular soil can be used as backfill material. In the past this lack of knowledge resulted a few times in pipeline floatation during the installation phase. This lack of knowledge can also result in the disposal of perfectly suitable backfill material.

A pipeline can be lifted up from the bottom of the trench if the weight of the pipeline is lower than the weight of the liquefied backfill. Pipeline floatation occurs if a soil-water mixture remains liquefied over a too large distance. In general, coarse granular soils remain liquefied for only a short period of time. If the backfill is composed of fine material, it will take a relatively long period of time before the soil settles and develops structural density. G.C. Sills (1998) defines structural density as the density which marks the change from a fluid-supported suspension to a soil in which effective stresses exist.

The subject of this study is prevention of pipeline floatation during dredge-based backfilling. It is therefore necessary to understand which characteristics cause pipeline floatation. Due to economic reasons, pipelines have become lighter than a typical soil-water mixture. This is a potential risk if the sedimentation time of a soil-water mixture is relatively long. Sedimentation time is the time required for soil particles to settle out of suspension. The sedimentation time and the backfilling process are analysed using the two dimensional (horizontal and vertical) sedimentation model (2DV model) developed by Van Rhee. The 2DV model is designed to simulate the sedimentation process inside a TSHD. However, in this thesis the model is used to simulate the backfilling of an offshore trench. To simulate this process correctly, it is necessary to implement a moving discharge point in the model. Loads due to waves and currents are not considered,
since pipelines are usually installed during relatively calm weather. These considerations make it possible to use the 2DV model. The backfilling process is studied in three steps. First, the process is simulated as a one-dimensional sedimentation test. Secondly, the backfilling of a trench using a stationary TSHD is evaluated. Finally, the backfilling of a trench using a moving TSHD is simulated.

The moving TSHD tests produce a distribution of the soil-water mixture density. This distribution is converted to a grid which is used as input for the beam-model (Matlab). The beam-model simulates pipeline displacement at a certain moment during the backfilling operation. The beam-model is based on the theory of the beam on elastic foundation. In this model only static loads are considered; the own weight of the pipeline and the buoyant force caused by the displaced fluid. The beam-model combined with the 2DV model provides an approach to model pipeline displacement during dredge-based backfilling.

The 2DV sedimentation model is tested to its limits in this thesis. The simulation of the backfilling operation of a moving hopper requires a model area with a great length and a relatively small height. This results in a model composed of stretched grid cells (in longitudinal direction). The 2DV model is at his numerical boundaries due to these stretched grid cells. The results of the ‘moving hopper’ simulations are therefore questionable. The results of the static simulations can be considered as reliable, since in these simulations the 2DV model remains well within the boundaries.
Symbol list

The list of symbols and abbreviations with corresponding unities is not exhaustive, but includes at least the most important symbols used in this report.

**Latin symbols**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unity</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Activity of a soil</td>
<td>-</td>
</tr>
<tr>
<td>c</td>
<td>Volumetric concentration</td>
<td>-</td>
</tr>
<tr>
<td>$c_v$</td>
<td>Consolidation coefficient</td>
<td>m$^{-2}$s</td>
</tr>
<tr>
<td>$C_v$</td>
<td>Volumetric concentration</td>
<td>-</td>
</tr>
<tr>
<td>$C_D$</td>
<td>Drag coefficient</td>
<td>-</td>
</tr>
<tr>
<td>$C_u$</td>
<td>Undrained shear strength</td>
<td>Pa</td>
</tr>
<tr>
<td>D</td>
<td>Particle diameter</td>
<td>m</td>
</tr>
<tr>
<td>$D_{10}$</td>
<td>Grain size for which 10% is smaller by weight</td>
<td>m</td>
</tr>
<tr>
<td>$D_{50}$</td>
<td>Median grain size for which 50% is smaller by weight</td>
<td>m</td>
</tr>
<tr>
<td>$D_{60}$</td>
<td>Grain size for which 60% is smaller by weight</td>
<td>m</td>
</tr>
<tr>
<td>e</td>
<td>Void ratio</td>
<td>-</td>
</tr>
<tr>
<td>E</td>
<td>Young’s modulus</td>
<td>N·m$^{-2}$</td>
</tr>
<tr>
<td>g</td>
<td>Gravitation acceleration</td>
<td>m$^{-2}$s</td>
</tr>
<tr>
<td>h</td>
<td>Mixture flow depth</td>
<td>m</td>
</tr>
<tr>
<td>k</td>
<td>Hydraulic conductivity</td>
<td>m·s</td>
</tr>
<tr>
<td>$k_h$</td>
<td>Horizontal bedding constant</td>
<td>N·m$^{-2}$</td>
</tr>
<tr>
<td>$k_v$</td>
<td>Vertical bedding constant</td>
<td>N·m$^{-2}$</td>
</tr>
<tr>
<td>I</td>
<td>Moment of inertia</td>
<td>m$^{-4}$</td>
</tr>
<tr>
<td>LL</td>
<td>Liquid limit</td>
<td>%</td>
</tr>
<tr>
<td>$L_{sed}$</td>
<td>Characteristic sedimentation length</td>
<td>m</td>
</tr>
<tr>
<td>$m_v$</td>
<td>Compressibility coefficient</td>
<td>-</td>
</tr>
<tr>
<td>$M_w$</td>
<td>Mass of water</td>
<td>kg</td>
</tr>
<tr>
<td>$M_{sed}$</td>
<td>Mass of (dry) sediment</td>
<td>kg</td>
</tr>
<tr>
<td>n</td>
<td>Porosity</td>
<td>-</td>
</tr>
<tr>
<td>p</td>
<td>Pore pressure</td>
<td>Pa</td>
</tr>
<tr>
<td>PI</td>
<td>Plasticity index</td>
<td>%</td>
</tr>
<tr>
<td>PL</td>
<td>Plasticity limit</td>
<td>%</td>
</tr>
<tr>
<td>q</td>
<td>Specific mixture discharge</td>
<td>m$^2$·s$^{-1}$</td>
</tr>
<tr>
<td>Q</td>
<td>Discharge</td>
<td>m$^3$·s$^{-1}$</td>
</tr>
<tr>
<td>R</td>
<td>Reduction factor</td>
<td>-</td>
</tr>
<tr>
<td>Re</td>
<td>Reynolds number</td>
<td>-</td>
</tr>
<tr>
<td>s</td>
<td>Specific gravity</td>
<td>-</td>
</tr>
<tr>
<td>S</td>
<td>Degree of saturation</td>
<td>-</td>
</tr>
<tr>
<td>u</td>
<td>Velocity mixture flow</td>
<td>m·s$^{-1}$</td>
</tr>
</tbody>
</table>
Friction velocity \( u_* \) \( \text{m} \cdot \text{s}^{-1} \)
Total volume \( V_{\text{tot}} \) \( \text{m}^3 \)
Volume solids \( V_s \) \( \text{m}^3 \)
Total volume \( V_t \) \( \text{m}^3 \)
Total volume of voids \( V_v \) \( \text{m}^3 \)
Volume water \( V_w \) \( \text{m}^3 \)
Water content \( W \) %
Fall velocity single particle \( w_0 \) \( \text{m} \cdot \text{s}^{-1} \)
Reduced fall velocity suspended sediment \( w_s \) \( \text{m} \cdot \text{s}^{-1} \)
Horizontal coordinate \( x \) m
Vertical coordinate \( z \) m

Greek symbols

\( \beta \) Compressibility of water \( \text{m}^{-2} \cdot \text{N} \)
\( \gamma \) Volumetric weight \( \text{kg} \cdot \text{m}^{-3} \)
\( \Delta \) Specific density of sediment \( \text{kg} \cdot \text{m}^{-3} \)
\( \theta \) Shields parameter -
\( \nu \) Kinematic viscosity of water \( \text{m}^2 \cdot \text{s}^{-1} \)
\( \xi_{\text{cl}} \) Solids content of clay %
\( \xi_{\text{cl},0} \) Critical clay content for cohesive behaviour %
\( \rho_s \) Density of sediment \( \text{kg} \cdot \text{m}^{-3} \)
\( \rho_{\text{sed}} \) Density of sediment \( \text{kg} \cdot \text{m}^{-3} \)
\( \rho_w \) Density of water \( \text{kg} \cdot \text{m}^{-3} \)
\( \rho_{\text{bulk}} \) Bulk density \( \text{kg} \cdot \text{m}^{-3} \)
\( \phi_0 \) Initial volume concentration of solids -
\( \phi_{\text{cl}} \) Volume concentration of clay -
\( \phi_h \) Horizontal mass flow \( \text{kg} \cdot \text{m}^{-2} \)
\( \phi_{\text{max}} \) Maximum concentration of solids -
\( \phi_{\text{mu}} \) Volume concentration of mud -
\( \phi_{\text{sa}} \) Volume concentration of sand -
\( \phi_{\text{sed}} \) Volume concentration of sediment -
\( \phi_{\text{si}} \) Volume concentration of silt -
\( \tau_b \) Bed shear stress Pa
\( \psi_{\text{sa}} \) Solids fraction of sand %
\( \psi_{\text{si}} \) Solids fraction of silt %
# Abbreviation list

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>2DV</td>
<td>Two dimensional vertical (model)</td>
</tr>
<tr>
<td>CPT</td>
<td>Cone penetration test</td>
</tr>
<tr>
<td>CSP</td>
<td>Corry shape factor</td>
</tr>
<tr>
<td>FDM</td>
<td>Finite difference method</td>
</tr>
<tr>
<td>FVM</td>
<td>Finite volume method</td>
</tr>
<tr>
<td>NS</td>
<td>Navier-Stokes</td>
</tr>
<tr>
<td>RANS</td>
<td>Reynolds averaged Navier-Stokes</td>
</tr>
<tr>
<td>SPT</td>
<td>Standard penetration test</td>
</tr>
<tr>
<td>TOP</td>
<td>Top of pipeline</td>
</tr>
<tr>
<td>TSHD</td>
<td>Trailer suction hopper dredger</td>
</tr>
<tr>
<td>TSL</td>
<td>Total sedimentation length</td>
</tr>
</tbody>
</table>
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1 Introduction

1.1 General development

Pipelines are a vital part of modern civilisation. Pipeline transport has become the most important way of moving fluids from place to place. Transport by pipeline is relatively inflexible compared to other means of transport. A pipeline is a fixed asset with high construction costs. Once a pipeline is placed, the operation and maintenance costs are relatively low. Pipeline transport consumes little energy compared to other transportation methods. Most offshore pipelines carry oil or gas, but they can also transport water or other fluids. More and more oil and gas is produced from offshore fields. The product has to be carried onshore and that is usually done by pipeline. Intra-field pipelines carry oil and gas from wellheads and manifolds to platforms and from one manifold to another. According to Guo et al. (2005) offshore pipelines can be classified as follows (see Figure 1):

- Flow lines transporting oil and/or gas from satellite subsea Wells to subsea manifolds.
- Flow lines transporting oil and/or gas from subsea manifolds to production facility platforms.
- Infield flow lines transporting oil and/or gas between production facility platforms.
- Export pipelines transporting oil and/or gas from production facility platforms to shore.
- Flow lines transporting water or chemicals from production facility platforms, through subsea injection manifolds, to injection wellheads.
1.2 Covering of pipelines

Pipelines can be laid uncovered on the seabed, but offshore pipelines can also be embedded in the seafloor when stabilisation and protection is required. Pipelines are primarily covered when they are located in shallow waters, in the near shore areas or in busy shipping routes. Covering secures protection against fishing gear, against dragging anchors and protection against dropped objects. It also increases the thermal resistance between the pipeline and the seawater. The temperature of the fluid is kept high, which helps to reduce hydration problems in gas pipelines. It minimises also wax deposition and pumping costs for oil pipelines. Last, covered pipelines resist buckling. Buckling is the bending of the pipeline due to expansion caused by thermal stress. The main disadvantage of covered pipelines is that it is complicated to locate leaks and carry out repairs.

A pipeline placed in a previously excavated trench in the seabed can be backfilled or it can be left uncovered. The material used to refill an excavated trench is called backfill. The typical material used as artificial backfill is coarse granular soil (rock, gravel, or coarse sand). When there is no suitable backfill material available close to the pipeline route, then material has to be transported from another location.

The backfill can be placed by different methods. This study is focused on dredge-based backfilling. Dredge-based backfilling means that the trench is backfilled using the suction pipe of a trailing suction hopper dredger (TSHD) as discharge pipe.
1.3 Relevance of this research

A subsea pipeline should remain stable in all circumstances. During its lifetime it may have to withstand wave action, heavy impacts, temperature changes or even earthquakes. Currently, most knowledge and concerns are related to the operational phase of the pipeline. However, the pipeline should also remain stable during installation. It is not exactly clear when the soil is suitable as backfill material and when it is not. In the past this lack of knowledge sometimes resulted in pipeline floatation during, or shortly after installation. In other cases material was disposed which would have been perfectly suitable serving as backfill. These two risks indicate the importance of accurately predicting the behaviour of a pipeline in a soil-water mixture during and after backfilling.

The most desirable way of pipeline backfilling is to use the excavated soil from the trench or local material. The engineering properties of the spoil are in general variable and not always certain. If the material is composed of large granular particles the water will disperse quickly from the soil-water mixture. In this case there will probably be no problem regarding pipeline floatation during installation. If the excavated material is composed of fine material the pipeline may experience floatation during backfilling. It will take some time before (fine) material settles out from suspension and develops structural density. The structural density has been defined as the density which marks the change from a fluid-supported suspension to a soil in which effective stresses exist (Sills, 1998). The significant processes regarding dredge-based backfilling are explained in Figure 2. The mixture of sand and water leaves the discharge pipe. Material may erode from the trench bottom when the course of the flow changes from a vertical to a horizontal direction. The suspension becomes stagnant and soil will settle from this mixture. The settled soil particles will develop strength. If there is a clay fraction there will also be a consolidation phase.

![Figure 2. Visualisation of the process of dredge-based backfilling (using a moving Trailling Suction Hopper Dredger). At the top of this figure the discharge pipe from which the slurry originates is visible. The slurry first falls on the trench bottom. Then it flows in horizontal direction. While the slurry loses velocity particles start to settle out of suspension and start to gain strength and form soil. This illustration is not drawn to scale.](image-url)
1.4 Problem definition

A potential problem with backfilling using a soil-water mixture consisting of a relatively fine material is that the pipeline can be lifted up from the bottom of the trench during, or shortly after the backfill operation (Palmer & King, 2008). This problem can occur if the density of the pipe is lower than the density of the backfill. In this thesis, the specific density of the pipeline is assumed to be 1.25.

The schematisation of the cross-section of a typical backfill operation is showed in Figure 3. Pipeline floatation is expected when the specific gravity of the soil-water mixture is higher than the specific gravity of the pipeline. When the fines content (clay, silt, fine sand) of a slurry is high, it is likely that it will take a relatively long time for the suspension to settle out and obtain strength. Figure 3 shows in brief an overview of the backfilling process. The significant phases regarding the transition from suspension to soil are considered. Note that not every phase is relevant for each material.

**Figure 3.** The backfilling of an underwater trench is visualised. The schematisation is focussed on the transition from suspension to soil. The floating pipeline is indicated with a red cross.
1.5 Aims and objectives

This study focuses on the behaviour of fine grained material when used as hydraulic backfill to cover subsea pipelines. In the previous sections, the importance of an increase in knowledge on this subject is explained. At the end an answer to the following main objective will be given:

How can pipeline floatation be prevented during dredge-based backfilling?

A tool is designed that can calculate whether a pipeline will float when it is backfilled using a dredge-based method. Input for this tool are the properties of the pipeline (dimensions, the specific weight), dimensions of the trench and the soil parameters. A number of research questions have been defined:

- What is the sedimentation length of a particular soil-water mixture? To achieve this, it is necessary to know the density of the slurry, the gradation of the soil, the initial mixture speed and trailing speed of the TSHD.
- How does the density of a particular soil-water mixture develop in time?
- How does the strength of the deposited material develop? In case of coarse sand or gravel the strength development will be instantaneous. Strength development of a clayey soil-water mixture may take a few days.
- The properties of the pipeline in combination with the three points above will provide an estimation whether the pipeline will remain stable during backfilling.

1.6 Structure of the thesis
2 Theoretical background

An underwater pipeline is susceptible for floatation during dredge-based backfilling if its density is lower than the density of the surrounding soil-water mixture. Another factor which determines whether a pipeline will float or not, is the length over which the soil-water mixture extends. The properties of the pipeline, in combination with the density of the soil-water mixture, determine the necessary length of the soil-water mixture to make the pipeline float. The process of pipeline floatation during dredge-based backfilling is illustrated in Figure 4.

![Figure 4. Visualisation of pipeline floatation during dredge-based backfilling.](image)

The length over which a soil-water mixture remains liquefied, depends on the sedimentation process. Sedimentation is the tendency for suspended particles to settle out of the soil-water mixture. The motion of the particles through the supporting fluid is in response to the forces acting on them. In nature this is usually gravity. The deposited particles form a saturated soil layer on the bottom of the fluid column. Sedimentation occurs in the soil suspension without any effective stress and structure. During sedimentation the density of the slurry does not change, except at the bottom of the slurry layer.

According to Schiffman et al. (1998) the deposited soil-water mixture passes through three phases. A representation is shown in Figure 5. During the first stage, the solid particles are suspended in the fluid. In this stage each particle behaves more or less independently from the others. In the second stage, there is a progressive deposition of
the solids, producing an interface with clear water above the sedimentation zone. The third zone is at the base, where particles accumulate. The thickness of the sedimented bed progressively increases in time.

![Figure 5. A schematic representation of sedimentation and consolidation processes in a vertical column, adapted from Schiffman et al. (1998).](image)

This chapter is divided in three parts according to the three phases in Figure 5: the inlet zone (Section 2.1), the settling zone (2.2) and the settled bed (2.3).

### 2.1 Inlet zone

A hydraulic backfill is a mixture composed of soil particles and water. Water is a homogeneous substance, while soil particles are usually heterogeneous. The heterogeneous nature of soil particles is defined by the difference in properties between particles. Soil properties directly related to single particles are the size and the specific density. However, not all soil properties are related to single particles. Some properties are related to a volume of particles. Examples of these so-called bulk
properties are: the grain size distribution, the porosity, the permeability and the plasticity index. These soil properties influence processes which are closely related to pipeline floatation. These processes are: mixture flows, sedimentation, erosion and the response of the seabed. To understand these processes, it is therefore necessary to understand the soil properties first.

The soil-water mixture in the inlet zone can be regarded as a single phase material. The density of the soil-water mixture ($\rho_m$) is influenced by the fraction of solids present in the carrying liquid. This fraction is determined by the volumetric concentration ($C_v$). The volumetric concentration determines the fraction of the mixture volume that is occupied by soil particles. The mixture density can be determined using the following equation:

$$\rho_m = \rho_s (C_v) + \rho_w (1 - C_v) \quad (2.1)$$

With:
- $\rho_m$ = Density mixture [kg·m$^{-3}$]
- $\rho_s$ = Density solids [kg·m$^{-3}$]
- $\rho_w$ = Density carrying fluid (seawater) [kg·m$^{-3}$]
- $C_v$ = Volumetric concentration [-]

### 2.2 Sedimentation zone

The sedimentation zone can be divided in three parts: clear water, soil-water mixture and sedimented bed (saturated soil). The soil-water mixture is described in the previous section. The sedimented bed is considered to be a homogeneous mechanical mixture of two phases. One phase represents the structure of solid particles in the soil aggregate and the other phase represents the fluid water in the pores or voids of the aggregate.

To obtain a soil structure, material has to settle out of suspension. This is called sedimentation. To describe sedimentation, sediment properties and flow conditions need to be taken into account. Sediment can be considered at the scale of a suspension or at the scale of individual particles. The most important properties are: the density, the shape and the size of the particle. In nature sediment consists of particles with different sizes and shapes. It is therefore not enough to only consider the properties of a single particle size. It is necessary to analyse mean values and other statistical properties. As a consequence classification of particles is rather arbitrary.
In order to describe sedimentation it is first necessary to describe the relevant sediment properties. Sediment and (geo-)morphology can be considered at four spatial and temporal scales. De Vriend (1991) argues that it is necessary to identify these (micro, meso, macro and mega) scales and interactions when studying morphodynamical processes. Properties can be related to the particle itself (micro scale) or properties can be related to the soil mixture or bulk soil properties (meso scale).

### 2.2.1 Grain size

This study focuses on soils that can be used as backfill for an underwater trench and placed with a Trailing Suction Hopper Dredger (TSHD). Therefore, soils with a large fraction of clay or fine silt are disregarded in this thesis. These soils are not suitable to be used as backfill, because the sedimentation time of these soils is too long compared with the backfilling speed. Also coarse granular soils are disregarded in this study, because these soils settle out of suspension relatively fast. Therefore, no problems regarding pipeline floatation are expected using these soils. For these reasons, the material regarded in this study ranges between 63 μm and 500 μm.

Soils are usually classified according to their individual particle size (d [m]), with sand, silt, clay and peat as the principle classes. Silt and clay are sometimes regarded as one soil fraction, called mud or fines. See Table 1 for a classification by size of clay, silt, sand and gravel.

<table>
<thead>
<tr>
<th>soil type</th>
<th>minimum (μm)</th>
<th>maximum (μm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>clay</td>
<td>-</td>
<td>2</td>
</tr>
<tr>
<td>silt</td>
<td>2</td>
<td>63</td>
</tr>
<tr>
<td>sand</td>
<td>63</td>
<td>2000</td>
</tr>
<tr>
<td>gravel</td>
<td>2000</td>
<td>63000</td>
</tr>
</tbody>
</table>

Table 1. Classification of soil fractions based on grain size according to NEN 5104.

### 2.2.2 Specific density

The most common mineral found in natural non-cohesive minerals is quartz. Due to weathering and abrasion, quartz has a specific gravity of 2.65. Particle specific gravity in natural soils will vary between 2.60 and 2.80. The lower values are typically of the coarser soils, while the higher values are of the finer soils. The average specific gravity of a soil (mixture) is normally close to that of quartz. The specific gravity of a soil is therefore usually assumed to be 2.65.
2.2.3 Grain size distribution

The variation of particle size can be described with a grain size distribution curve. A grain size distribution curve is a cumulative size distribution curve showing particle size versus accumulated percent finer, by weight. According to (Staple, 1975; Gupta and Larson, 1979) the more widely graded sand, the more closely it is able to pack. This is caused by the ability of smaller grains to fit in the space between larger grains. The particle size of a certain soil type can be presented in a grain size distribution diagram. Useful values determined from the grain size distribution curve are:

Maximum grain size:
Smallest screen size through which all particles will pass.

Median grain size:
The median grain size ($d_{50}$) is the midpoint of the grain size distribution. Fifty percent of the sediment is finer by weight than the median grain size.

Effective size:
Grain diameter ($d_{10}$) corresponding to the ten percent finer ordinate on the grain size distribution curve.

Coefficient of uniformity:
The steepness of the curve can be expressed in the coefficient of uniformity ($C_u$).

\[
C_u = \frac{D_{60}}{D_{10}} \quad (2.2)
\]

With:
- $C_u$ = coefficient of uniformity [-]
- $D_{60}$ = 60% of the sample is smaller than this diameter [m]
- $D_{10}$ = 10% of the sample is smaller than this diameter [m]

A soil is well graded, if the distribution of the grain sizes extends over a rather large range. The coefficient of uniformity will be large in that case. A soil is called poorly graded, if most particles in a soil mass are of approximately the same size. The value of $C_u$ is then close to one. The $C_u$ of a well graded soil, is larger than approximately five. A soil can have a combination of two or more well graded soil fractions. This is called gap graded.
2.2.4 Fall velocity of individual sphere in still water

The fall velocity of particles in a sand-water mixture is influenced by a number of aspects:

- undisturbed fall velocity
- sediment concentration (hindered settling)
- particle shape
- particle gradation

The settling is analysed on a step by step basis. First, the fall velocity of a single sphere in still water is described. Secondly, the fall velocity for non-spherical particles is described. Finally, the effect of sediment concentration and particle size distribution is described.

The fall velocity is a behavioural property. Settling particles are underwater influenced by two forces: a gravity force and a fluid drag force. The settling velocity of a particle becomes constant, when there is equilibrium between these two forces.

\[ F_g = F_D \rightarrow \frac{1}{6} (\rho_s - \rho_w) g D = \frac{1}{2} (\rho_w w^2 C_D) \frac{1}{4} (\pi D^2) \]  \hspace{1cm} (2.3)

From equation 2.3 follows the fall velocity \( w \) (for a spherical particle):

\[ w_0 = \sqrt{\frac{4 R_s g D}{3 C_D}} \quad \text{with} \quad R_s = \frac{\rho_s - \rho_w}{\rho_w} \]  \hspace{1cm} (2.4)

With:
- \( w_0 \) = fall velocity single particle [m s\(^{-1}\)]
- \( D \) = particle diameter [m]
- \( C_D \) = drag coefficient [-]
- \( g \) = gravitational acceleration [m s\(^{-2}\)]

The value of drag coefficient \( C_D \) depends on the value of the Reynolds number \( (Re) \) and the shape factor. This relationship is represented in Figure 6. The Reynolds number depends on fall velocity \( w \), the normative grain diameter \( D \) and the kinematic viscosity \( \nu \). The Reynolds number is valid for a settling particle in a still fluid.

\[ Re_p = \frac{w D}{\nu} \]  \hspace{1cm} (2.5)

With:
- \( w \) = fall velocity [m s\(^{-1}\)]
- \( D \) = particle diameter [m]
- \( \nu \) = kinematic viscosity coefficient water [m\(^2\) s\(^{-1}\)]
2.2.5 Non spherical particles

Expressions valid for a sphere cannot be applied for a natural sediment particle, because of the differences in shape. According to Van Rijn (1993) the shape effect is largest for relatively large particles (> 300 μm). These large particles deviate more from a sphere than a small particle. The fall velocity of non-spherical sediment particles can be determined from the following formulas.

\[
\begin{align*}
    w_0 &= \frac{(s - 1)gD^2}{18v} & \text{For } 1 < D \leq 100 \mu m \\
    w_0 &= \frac{10v}{D} \left[ 1 + \frac{(s - 1)gD^3}{100v^2} - 1 \right] & \text{For } 100 < D \leq 1000 \mu m \\
    w_0 &= 1.1\sqrt{(s - 1)gD} & \text{D } \geq 1000 \mu m
\end{align*}
\]

With:
- \(w_0\) = fall velocity individual particle [m\cdot s^{-1}]
- \(D\) = particle diameter [m]
- \(v\) = kinematic viscosity coefficient water [m^2\cdot s^{-1}]
- \(s\) = specific gravity (= 2.65) [-]

2.2.6 Effect of sediment concentration

The fall velocity of a single particle is influenced by the presence of other particles. To some extent a cloud of particles in a fluid will have a larger fall velocity than that of a
single particle in the same fluid. Fall velocity will be reduced, when sediment concentration becomes larger. This effect is known as hindered settling. It is partly caused by the fluid return flow. This return flow is induced by the downward movement of the settling particles. Furthermore, shear stresses will be larger in a concentrated suspension. Fluidisation occurs when the vertical upward fluid flow is so strong that the upward drag forces on the particles become equal to the downward gravity forces. This results in no net vertical movement of the particles. According to Richardson & Zaki (1954) the fall velocity in a fluid can be calculated with the following formula:

\[ w_s = w_0 (1 - c)^n \]  

(2.9)

With:  
- \( w_s \) = reduced fall velocity in a suspension [m\(\cdot\)s\(^{-1}\)]  
- \( w_0 \) = fall velocity of a single particle in a fluid [m\(\cdot\)s\(^{-1}\)]  
- \( c \) = volume concentration [-]  
- \( n \) = coefficient which is a function of particle Reynolds number

Volume concentration of a mixture:

\[ c = \frac{V_s}{V_t} \]  

(2.10)

With:  
- \( V_s \) = volume solids [m\(^3\)]  
- \( V_w \) = volume water [m\(^3\)]  
- \( V_t \) = \( V_s + V_w \) = volume total [m\(^3\)]

- \( Re < 0.2 \) \( n \) = 4.65  
- \( 0.2 < Re < 1 \) \( n \) = 4.35\( Re^{-0.03} \)  
- \( 1 < Re < 500 \) \( n \) = 4.45\( Re^{-0.1} \)  
- \( Re > 500 \) \( n \) = 2.39

For high and low (turbulent and laminar) Reynolds numbers the coefficient is constant.

### 2.2.7 Effect of grain size distribution

In case of a sand-water mixture consisting of multiple sand fractions, with each a different grain size diameter, there will be segregation. This segregation is due to the different fall velocities of the smaller and the larger grains. At high sediment concentrations, there will be interaction between the grains. Large particles will have a larger fall velocity and will collide with the slower small particles. Due to these collisions, the fall velocity of the larger particles will be reduced.

Starting point for the next example is a homogenous sand-water mixture with two uniform fractions with each a different grain size. When the particles settle out of the sand-water mixture, four zones will emerge. One zone on top with clear liquid and one
zone with deposited material on the bottom. Settlement will take place in the two zones in between. Zone B is a suspension with only small particles. Zone C is a suspension with both large and small particles. According to the theory of Mirza and Richardson (1979), the concentration in zone C is equal to the initial concentration.

For a mixture with two fractions the continuity equation (2.9), which evaluates zone C, becomes:

\[ w_F(1 - C_2) = w_{L,2} c_{L,2} + w_{C,S,2} c_{S,2} \]  \hspace{1cm} (2.11)

With:
- \( C_2 = c_{L,2} + c_{S,2} \)
- \( W_F \) = fall velocity \( \text{[m}\cdot\text{s}^{-1}] \)

Suffixes:
- \( L \) = large particles
- \( S \) = small particles

This makes equation 2.11 for the fine and the coarse fraction in zone C:

\[ w_{L,2} = w_{0,L,2} (1 - C_{L,2}) - v_{S,S,2} c_{S,2} \]  \hspace{1cm} (2.12)

\[ w_{S,2} = w_{S,S,2} (1 - C_{S,2}) - v_{S,L,2} c_{L,2} \]  \hspace{1cm} (2.13)

The relation between the concentration and the reduced fall velocity in comparison to the water is described in equation 2.9. It can be described for each fraction:

\[ w_{S,L,2} = w_{0,L,2} (1 - C_2)^{0.5} \]  \hspace{1cm} (2.14)

\[ w_{S,S,2} = w_{0,S,2} (1 - C_2)^{0.5} \]  \hspace{1cm} (2.15)

Equation 2.11 can be rewritten for the settlement of a mixture of two fractions in zone C when equation 2.12 is combined with 2.14 and 2.13 with 2.15. The reduced fall velocities can be derived for both fractions.
Mirza and Richardson introduce a correction factor as proposed by Richardson and Zaki. In this way the reduced fall velocity in comparison to the water can be obtained:

Slip velocity:

\[
\begin{align*}
    w_{s,l} &= w_{0,s,l}(1 - C_2)0.5(1 - C_{l,2}) - w_{0,s,2}(1 - C_2)0.5C_{l,2} \\
    w_{s,S} &= w_{0,s,2}(1 - C_2)0.5(1 - C_{S,2}) - w_{0,l,2}(1 - C_2)0.5C_{l,2}
\end{align*}
\] (2.16) (2.17)

2.2.8 Kynch’s sedimentation theory

Kynch presented in 1951 his paper ‘A theory of sedimentation’. In this paper, he proposed a kinematical theory of sedimentation based on the propagation of kinematic waves in an idealised suspension. The suspension is considered as a continuum. He introduced an empirical relationship between settling velocity and local sediment concentration. Assuming that at any point in a suspension, the settling velocity of particles depends only on the local concentration of particles. The sedimentation process is represented by the continuity equation of the solid phase:

\[
\frac{\partial \phi}{\partial t} + \frac{\partial f_{bk}(\phi)}{\partial z} = 0
\] (2.20)

\[
0 \leq z \leq L, \quad t \geq 0
\]

With:

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi$</td>
<td>local volume fraction of solids</td>
<td>[-]</td>
</tr>
<tr>
<td>$z$</td>
<td>height</td>
<td>[m]</td>
</tr>
<tr>
<td>$t$</td>
<td>time</td>
<td>[s]</td>
</tr>
<tr>
<td>$f_{bk}(\phi)$</td>
<td>Kynch batch flux density function, where $v_s$ is the solids-phase velocity. The basic assumption is that the local solid-liquid velocity is a function of the volumetric concentration $\phi$ only.</td>
<td></td>
</tr>
</tbody>
</table>

Equation 2.20 is considered together with the initial condition for the sedimentation of an initially homogeneous suspension of concentration $\phi_0$. 

\[
\frac{\partial \phi}{\partial t} + \frac{\partial f_{bk}(\phi)}{\partial z} = 0
\] (2.20)
\[ \phi(z,0) = \begin{cases} \phi_0 & \text{for } z = 0 \\ \phi_{\text{max}} & \text{for } 0 < z < L \\ \phi_{\text{max}} & \text{for } z = L \end{cases} \] (2.21)

With:  
\( \phi_0 \) = initial solids concentration [-]  
\( \phi_{\text{max}} \) = maximum solids concentration [-]

To construct the solution of the initial value problem, the method of characteristics is employed. This method is based on the propagation of \( \phi_0(z_0) \) in a \( z \) versus \( t \) diagram (Figure 8). These straight lines, the characteristics may intersect. This makes the solutions of equation 2.21 discontinuous in general. This is due to the nonlinearity of the flux-density function. Even for smooth initial data, a scalar conservation law with nonlinear flux density function may produce discontinuous solutions.

Kynch’s model assumes that no effective stresses develop. Kynch’s model is therefore applicable in the earlier phase of the settling and strength development process, when effective stresses are small.

Figure 8. Modes of sedimentation. From the left to the right, the flux plot, the settling plot showing characteristics and shock lines and the concentration profile are shown for each mode.
For the problem of sedimentation of an initially homogeneous suspension, a solution can be explicitly constructed by the method of characteristics. According to Kynch, for a flux-density function with exactly one inflection point, there are three qualitatively different solutions (Figure 8). Kynch presented a discussion of stable and instable kinematic discontinuities. He relied on physical insight, and suggested that only stable discontinuities could occur.

2.2.9 Sedimentation length

Sedimentation is the settling of particles from a suspension under influence of gravity. When the soil-water mixture is placed in a trench, it will flow in two spatial directions. Next to a vertical motion, due to gravity, the soil particles also move in a horizontal direction.

Granular mixture flows

A soil-water mixture that moves over a sandbed, is represented by the following formula. Horizontal mass flow \( \phi_h \) indicates the amount of soil that passes a certain surface per unit of time.

\[
\phi_h = \rho_s \cdot u \cdot c \tag{2.22}
\]

With:
- \( \phi_h \) = horizontal mass flow \([\text{kg} \cdot \text{s}^{-1} \cdot \text{m}^{-2}]\)
- \( u \) = velocity mixture flow \([\text{m} \cdot \text{s}^{-1}]\)
- \( c \) = volume concentration \([-]\)
- \( \rho_s \) = specific density sand \([\text{kg} \cdot \text{m}^{-3}]\)

Specific mixture discharge \((q)\):

\[
q = u \cdot h = \frac{Q}{B} \tag{2.23}
\]

With:
- \( q \) = specific mixture discharge \([\text{m}^2 \cdot \text{s}^{-1}]\)
- \( h \) = depth mixture flow \([\text{m}]\)
- \( Q \) = discharge \([\text{m}^3 \cdot \text{s}^{-1}]\)
- \( B \) = width \([\text{m}]\)

Specific particle transport \((s)\):

\[
s = \rho_s \cdot u \cdot c \cdot h \tag{2.24}
\]

With:
- \( s \) = specific particle transport \([\text{kg} \cdot \text{m}^{-1}]\)
- \( u \) = velocity mixture flow \([\text{m} \cdot \text{s}^{-1}]\)
- \( c \) = volume concentration \([-]\)
- \( h \) = depth mixture flow \([\text{m}]\)
Sedimentation length

The length, at which a turbulent soil-water mixture can extend, is determined by the degree of turbulence and the fall velocity of the soil particles. The degree of turbulence is determined by flow conditions, such as discharge, bed gradient and bottom friction. When the degree of turbulence in the suspension becomes too low, the turbulent diffusion is not able anymore to compensate sedimentation. Soil will settle out of suspension and the bed level will rise. The length, over which sedimentation takes place, indicates the distance that the suspension can flow. Only suspended soil provides a driving force for the soil-water mixture flow. A characteristic value for the process of sedimentation is the sedimentation length \((L_{sed})\). The sedimentation length is the distance a particle has travelled in horizontal direction, while in vertical direction it has passed the mixture flow depth. The sedimentation length is represented by the following formula:

\[
L_{sed} = u \frac{h}{w_s} = \frac{q}{w_0(1 - c)^n}
\]

With:

- \(L_{sed}\) = characteristic sedimentation length \([m]\)
- \(u\) = velocity mixture flow \([m\cdot s^{-1}]\)
- \(w_s\) = reduced fall velocity in a suspension \([m\cdot s^{-1}]\)
- \(h\) = mixture flow depth \([m]\)

2.2.10 Porosity

When considering a soil-water mixture, the terms ‘porosity’ and ‘void ratio’ makes little sense. In this case the term ‘solids content’ would be preferred. The terms ‘porosity’ and ‘void ratio’ become more meaningful when the soil-water mixture has become a soil. This occurs at the end of sedimentation, when the particles come into contact with each other and start the transfer of effective stress.

Soil porosity is a measure of pores found within a soil or sediment. It determines the total amount of water, a soil can hold. The greater the volume of pore spaces a material contains, the higher its porosity and the more water it can hold. Porosity can be expressed as the ratio of the volume of the pore space to the total volume of the material as given by the formula below:

\[
n = \frac{V_v}{V_{tot}}
\]

With:

- \(V_{tot}\) = total volume \([m^3]\)
- \(V_v\) = total volume of voids \([m^3]\)

Porosity is largely influenced by grain size, shape and grain size distribution. Well sorted sediment is composed of grains with the same size and shape. The overall size of the grains is not important. Sediment characterised by large grain size can have the
same porosity as one composed of much smaller particles. Poorly sorted soils contain particles of many sizes. The smaller particles fill up the pore spaces between the larger grains, making the sediment less porous. Sediments that are poorly sorted, often possess a low porosity.

In addition to size and sorting, another factor affecting porosity is grain shape. Soil with rounded particles will pack tighter than a soil with irregular shaped grains. The orientation of these grains can also affect the porosity. For example, water trying to flow directly through a clay layer, will be greatly hindered.

The quantity of voids can also be defined in the void ratio $e$ [-]. $e$ is the ratio of the volume of voids to the volume of the solid substance. It is possible to express the porosity in void ratio en vice versa.

$$
e = \frac{V_v}{V_{tot} - V_v} = \frac{1}{1 - n}$$

(2.27)

2.2.11 Permeability

Permeability is closely related to porosity. Both of these terms reflect the capacity of a soil to transmit water. It is controlled by the size of the pores and the amount of interconnections. A soil can have a high porosity, but a low permeability if the pores are not properly connected. The permeability influences the erosion process (Section 2.2.12).

For example, sandy soils are often quite porous. In these soils there is a relatively high percentage of void spaces between the grains. Sandy soils are also very permeable. The pore spaces are usually large and well connected. This allows water to flow through them easily. In soils where clays and silts are predominate, the permeability can decrease much due to several factors. Clay can display a higher porosity than sand. However, the clay particles are much finer and the spacing between them is very small. Clays and silts also may not pack together particularly well. This is caused by irregular grain shapes and the fact that certain clay minerals have electrostatic charges which will repel each other. The molecular attraction on the water trapped in the small pore spaces between clay particles is much stronger than can be found in the larger sand pores.

2.2.12 Erosion

The opposite mechanism of sedimentation is erosion. Traditional sediment erosion formulas such as the Van Rijn pick-up function (Van Rijn, 1993), are pick-up functions of single particles ($D_{50}$, $\rho_s$) suitable for low velocities ($0.5 - 2$ m/s). This approach cannot give very accurate results for high concentrated sediment flows, where significant higher velocities are suspected. This would lead to considerable overestimation of the size of the scour hole and the erosion due to mass flow in general. According to Van Rhee (2007) erosion is hindered by the properties of the soil mass, especially taking place at higher flow velocities. Dilatancy, permeability and the
(un)drained shear stress of the soil mass are important parameters in the theory of hindered erosion.

2.3 Settled bed

The third phase is the settled bed. In this phase the soil-water mixture has settled out completely. This phase consists of two parts: clear water and sedimented bed. Geomechanical forces are governing in this phase.

2.3.1 Structural density

The concept of effective stress has been a cornerstone in the analysis of soil behaviour. Terzaghi (1936) suggested that the structure of soil and its behaviour was uniquely associated with the existence of effective stress.

Studying the phenomena of the end of sedimentation and the beginning of consolidation phases, Been & Sills (1981) concluded that the basic difference between a suspension (sedimentation phase) and a solid skeleton consisting of the same particles (consolidation phase) is the presence of effective stresses. This implies that during the sedimentation phase, the measured pore water pressures are equal to the total vertical stress. However, they have lower values once the transition to consolidation has occurred. The maximum density, at which a soil-water mixture can exist as a suspension without the presence of effective stresses, is termed the structural density. At higher density values, there is a soil structure that supports some or all of the sediment weight through non-zero effective stresses. There is a corresponding structural void ratio, at which the effective stress is zero.

The fluid pressure is equal to the total load, because it behaves as a fluid. This is no longer true when an extended structure is being formed, because this structure can carry at least part of its own weight. As a result, the pore water pressure will be lower than the total weight of the mixture. When the structural bonds are not all rigid, the structure will slowly collapse under its weight, expelling the pore water. The pore water flow has to overcome relatively high friction losses when moving through the narrow channels on its way to the soil layer surface. The pore water pressure will be larger than
the hydrostatic pressure as long as the system is not in equilibrium. The difference is called excess pore pressure.

Experimental and theoretical studies of settling processes are reported by Pane (1985) and Toorman (1996). These studies connect sedimentation and consolidation processes. All the work reported so far, has been restricted to the solution of vertical deposition problems. Nevertheless, the study of a backfilled trench should not be limited to the modelling of sedimentation and strength development only. Transportation should also be included. Transportation of sediments, sedimentation and strength development are simultaneous parts of the settling process.

2.3.2 Transition from sand like to clay like behaviour

This thesis focuses on dredge-based backfilling using non-cohesive (relatively fine grained) material. It is therefore necessary to determine the difference between sand like and clay like behaviour. Fine grained soils show a transition from behaviour that is more fundamentally like sands to behaviour that is more fundamentally like clays over a fairly narrow range of Atterberg limits. At one end of the transition, fine grained soils are non-plastic and act very similar to sands in most respects. On the other end of the transition are soils that behave very similar to clay.

The Atterberg limits represent water contents defining the transition from liquid (Liquid Limit: LL [%]) to plastic behaviour and from plastic to solid behaviour (Plastic Limit: PL [%]). The difference between LL and PL is the Plasticity Index (PI [%]):

\[ PI = LL - PL \]  \hspace{1cm} (2.28)

Figure 9. The Atterberg limits reflect the transition in water content for which the behaviour of a soil changes from solid to plastic (PL) and from plastic to liquid (LL). The difference between these water contents is the plasticity index (PI).

Figure 9 illustrates the classification of the Atterberg limits. The undrained shear strength of sediments at the LL is in general around 1 kPa, and around 100 kPa at the PL. PI is the difference in water content between LL and PL for which a sediment mixture exhibits plastic behaviour. It depends on the clay content, the clay mineralogy and the chemical properties of the pore water. The PI determines if a soil behaves clayey or sandy. For a PI < 7% the material behaves sandy. Another definition of PI:

\[ PI = A(\xi_{cl} - \xi_{cl,0}) \]  \hspace{1cm} (2.29)

With:  
\[ A \] = activity of a soil  
\[ \xi_{cl,0} \] = offset for cohesive behaviour
Parameter A characterises the water-binding property of a soil which can vary considerably (0 – 10) as a function of clay mineralogy, pore water characteristics and polymeric bonding.

The boundaries for the transition between a cohesive and a network structure are visualised in a sand-silt-clay triangle (see Figure 10). In this triangle the sand, silt and clay content by dry weight are given on the axes. The horizontal solid line indicates the transition between non-cohesive (below) and cohesive (above) mixtures. A clay content of 5 – 10 % by dry weight appears to be the transition value. This parameter is therefore set at 7 % herein. The dotted lines in the left-hand corner indicate the boundary of a sand-dominated network structure for different volume fractions of water. This transition is determined by the volume content of sand.

Based on the transition for a cohesive and network structure, six bed types can be distinguished in the sand-silt-clay triangle. Sediment beds in the lower left hand corner and lower right hand corner of the sediment triangle have a sand dominated (I and II) and silt-dominated (V and VI) network structure. In the remainder, the network structure is dominated by clay if sufficient clay is present (IV). In area III none of the fractions of sand, silt or clay are large enough to build the network structure itself, and all soil fractions contribute to the network structure. The horizontal line at a clay content of 7 % divides sediment mixtures which are cohesive (II, IV and VI) or non-cohesive (I, III and V).

![Figure 10. Soil classification diagram.](image)

Previous observations in this section have focused on fine grained soils for which the fines content is greater than 50%. According to Boulanger et al. (2006), the same findings may be extended to soils with slightly lower fines content in certain cases. The key issue is whether or not the fines fraction constitutes the stress carrying matrix or skeleton for the soil mass, with the larger particles essentially floating (isolated from each other) within the matrix. For many soils, it is likely that the fines fraction forms the load carrying matrix when the fines fraction exceeds roughly 35%.

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2.3.3 Consolidation

The last phase in the process of soil formation is consolidation. Consolidation is the time dependent process of compacting a fluid filled permeable medium under load. Sedimentary deposits consolidate in response to a load applied to their surface. However, they can also consolidate in response to gravitational body force (self-weight consolidation). There are three ways water saturated sediment can consolidate: compression of the pore water, deformation of the sediment grains, or the rearrangement of the grains into a more closely packed configuration by squeezing water out of the pores and closing pore space. Pore water is generally considered as incompressible. Grain deformation is negligible compared with grain rearrangement. Therefore, one dimensional consolidation of saturated sediment occurs only if the voids can shrink and water can drain from the sediment.

Settlement refers to the vertical displacement that occurs as sediment compacts. If sediment is laterally constrained, vertical settlement of a saturated deposit can occur only if pore fluid can escape. If there is significant lateral strain, some vertical settlement occurs immediately after loading without loss of pore fluid. Vertical settlement is most easily quantified by measuring surface displacement. One measure of the magnitude of consolidation is the temporal change of surface displacement.

Traditional consolidation analyses examine the response that follows application of an external surface load, and they typically focus on consolidation of clay rich sediment (Terzaghi, 1943). However a few studies have considered the self-weight consolidation problem (Gibson et al., 1967; Been and Sills, 1981; Toorman, 1996). Self-weight consolidation occurs in sediment not in equilibrium with the gravitational stress field.

In a soil-water mixture, the fluid pressure $u_w$ is equal to the total load $\sigma$. This is no longer true when a soil skeleton is being formed, because this structure can carry at least a part of its own weight. The pore water pressure will become lower than the total weight of the soil-water mixture. The formed structure will slowly collapse under its own weight, since the structural bonds are not all rigid. Pore water will be expelled during this process. When the pore water moves through the narrow channels, on its way to the soil surface, it has to overcome relatively high friction losses. As a consequence, the pore water pressure $u_w$ will be larger than the hydrostatic pressure $u_0$ as long as the system is not in equilibrium. The difference between hydrostatic pressure and pore water pressure is called excess pore water pressure, $u = u_w - u_0$.

The load that is effectively carried by the soil skeleton is called the effective stress $\sigma'$. The effective stress is defined as:

$$\sigma' = \sigma - u_w = \sigma - u_0 - u$$  \hspace{1cm} (2.30)

The total stress ($\sigma$), of a certain point in the sediment layer, can be obtained by integration of the total density ($\rho$) from the depth ($z$) of this point to the soil water interface $z_i$.

$$\sigma' = \sigma_0 \int_{z}^{z_i} \rho g \, dz$$  \hspace{1cm} (2.31)
\( \sigma_0 \) is the overburden pressure at \( z_i \). In the case of backfilled offshore trench this can only be the weight of the water column, since there is no externally added load. As long as there are excess pressure gradients, consolidation will continue. The excess pressure gradients are the driving force for expelling pore water. When these excess pore pressures are dissipated, equilibrium will be reached and consolidation will be completed.

**One dimensional sedimentation theory**

The consolidation stage can be described with equations using an equation initially proposed by Terzaghi (1942). This equation can be formulated as:

\[
\frac{\partial p}{\partial t} = c_v \frac{\partial^2 p}{\partial z^2} \tag{2.32}
\]

With:
- \( p \) = pore pressure [Pa]
- \( c_v \) = consolidation coefficient = \( \frac{k}{\gamma_w (m_v + n \beta)} \) [m\(^2\)·s]
- \( k \) = hydraulic conductivity [m·s]
- \( \gamma_w \) = volumetric weight of water [kg·m\(^{-3}\)]
- \( m_v \) = compressibility coefficient [-]
- \( n \) = porosity [-]
- \( \beta \) = compressibility of water [m\(^{-2}\)·N]
- \( z \) = vertical position [m]
3 Two dimensional sedimentation model

In this thesis a two dimensional numerical model is used to simulate the backfilling process of an offshore trench. The trench is backfilled using the suction pipe of a TSHD as discharge pipe. The 2DV (horizontal and vertical) model is a flow model. The model is based on the Reynolds Averaged Navier-Stokes (RANS) equations. In the 2DV model momentum and sediment transport equations are solved. It is originally designed to simulate the sedimentation process inside a TSHD. The 2DV model is described in more detail in Van Rhee (2002) and Van Rhee (2011). Van Rhee (2011) verifies and validates the model using a data set found in Mastbergen and Winterwerp (1988). The aim of this experimental study was to obtain a better knowledge of the processes occurring during hydraulic sand fill operations.

An empirical relation is used to model the interaction between the flow and the settled sediment. This relation is valid as long as flow mechanisms are governing. The model becomes inaccurate when soil mechanical mechanisms are dominant. This is the case when the backfill material is composed of relative coarse sand. This can also be predicted in a simple model and it is therefore not necessary to model this type of sand with the 2DV model. Furthermore, it is unlikely that this type of sand will cause pipeline floatation during dredge-based backfilling. Another mechanism which is not included in the 2DV model is consolidation. Consolidation can occur when the soil contains a substantial fraction of clay and/or silt. However, this type of material is not relevant for modelling with the 2DV model since it is not recommended to apply a backfill with this type of material using a dredge-based method. A clayey soil-water mixture would need a relatively long time to settle out completely.
3.1 Overview of the 2DV model

The 2DV model is composed of three different modules (see Figure 11). These modules contain three different transport equations:

1. Momentum or 2DV RANS module: Transport of momentum.
2. Turbulence or k-ε module: Transport of turbulent quantities.

In the 2DV RANS module the flow field is solved. The distribution of suspended sediment is calculated in the sediment transport module. The k-ε module is required for turbulence closure. The separate equations are strongly coupled and have to be solved simultaneously. Each module produces a parameter which is used as input for the other two modules. The 2DV RANS module generates the mixture velocity \((u, w)\). The sediment transport module produces the mixture density \((\rho_m)\) and the turbulence model generates the eddy viscosity \((\nu_e)\). The eddy viscosity is not a fluid property, but a property of the flow field.

![Figure 11. Overview of the 2DV sedimentation model. The model is composed of three different modules. In the 2DV RANS module the flow field is solved. The distribution of suspended sediment is calculated in the sediment transport module. The k-ε module is required for turbulence closure. The separate equations are strongly coupled and have to be solved simultaneously.](image-url)
Momentum module

The sediment water mixture is regarded as a single fluid. Therefore the density used in the momentum equations is the mixture density. The mixture density will change in time and space depending on the local sediment concentration.

The soil-water mixture is considered as a whole fluid in the momentum equations. This approach is called a drift-flux method. The acceleration of a volume of fluid is the sum of three forces acting on that fluid. The first one is a normal stress or pressure gradient acting on the fluid volume. The second one is caused by gravity. The third one results from shear stress. All stresses are converted to velocity gradients. This generates a system of three equations (vertical moment, horizontal moment and continuity). The horizontal momentum equation applied on a control volume \( d\Omega \) with surface \( dS \) is defined as:

\[
\frac{\partial}{\partial t} \int_{\Omega} \rho u \, d\Omega + \int_{S} \rho u \vec{v} \cdot \vec{n} \, dS = \int_{S} \tau_{x,j} \vec{i}_j \cdot \vec{n} \, dS - \int_{S} p \vec{i}_x \cdot \vec{n} \, dS \tag{3.1}
\]

The vertical momentum equation applied on a control volume \( d\Omega \) with surface \( dS \) is defined as:

\[
\frac{\partial}{\partial t} \int_{\Omega} \rho w \, d\Omega + \int_{S} \rho w \vec{v} \cdot \vec{n} \, dS = \int_{S} \tau_{z,j} \vec{i}_j \cdot \vec{n} \, dS - \int_{S} p \vec{i}_z \cdot \vec{n} \, dS - \int \rho g \, d\Omega \tag{3.2}
\]

With:

- \( u \) = horizontal flow velocity \([\text{m} \cdot \text{s}^{-1}]\)
- \( w \) = vertical flow velocity \([\text{m} \cdot \text{s}^{-1}]\)
- \( p \) = pressure \([\text{Pa}]\)
- \( \rho \) = density of the soil-water mixture \([\text{kg} \cdot \text{m}^{-3}]\)
- \( \tau \) = shear stress \([\text{Pa}]\)
- \( \vec{i}_x \) = unity vector in x direction \([-]\)
- \( \vec{i}_z \) = unity vector in z direction \([-]\)
- \( \vec{v} \) = kinematic viscosity \([\text{m}^2 \cdot \text{s}^{-1}]\)
- \( \vec{n} \) = normal vector \([-]\)

The shear stresses in the momentum equations are related to the velocity gradients. In the 2DV model, the time averaged cross products of the turbulent velocity fluctuations are called Reynolds stresses. These are regarded as shear stresses. The total shear stress \( \tau_{xz} \) is therefore the sum of a laminar and a turbulent contribution. This is written as:

\[
\tau_{xz} = \rho (v + v_e) \frac{\partial u}{\partial z} \tag{[\text{Pa}]}
\]

With:

- \( v_e = c_{\mu} \frac{k^2}{\epsilon} \) \([\text{m}^2 \cdot \text{s}^{-1}]\)
The continuity equation in conservative notation is defined as:

$$\int_S \vec{\nabla} \cdot \vec{n} \, dS = 0 \quad (3.3)$$

**Turbulence module**

The output of this module is the previously mentioned eddy viscosity. The $k$-$\varepsilon$ model is applied in the turbulence module. This model consists of two transport equations. These are solved together with the Navier-Stokes equation. The turbulent energy is indicated by parameter $k$, while the dissipation of turbulence energy is indicated by parameter $\varepsilon$.

The transport equation for $k$ reads:

$$\frac{\partial k}{\partial t} + \frac{\partial (uk)}{\partial x} + \frac{\partial (wk)}{\partial z} = \frac{\partial}{\partial x} \left( \frac{\nu_k}{\sigma_k} \frac{\partial k}{\partial x} \right) + \frac{\partial}{\partial z} \left( \frac{\nu_k}{\sigma_k} \frac{\partial k}{\partial z} \right) + P + P_b - \varepsilon \quad (3.4)$$

With:
- Turbulent production $P$: $\nu_k \frac{\partial u_i}{\partial x_j} \frac{\partial u_i}{\partial x_j}$
- Buoyancy $P_b$: $\frac{g_i \nu_k \frac{\partial \rho}{\partial x_i}}{\rho \sigma_k}$

The values of the other parameters are shown in Table 2:

<table>
<thead>
<tr>
<th>$c_\mu$</th>
<th>$c_{1\varepsilon}$</th>
<th>$c_{2\varepsilon}$</th>
<th>$c_k$</th>
<th>$c_\varepsilon$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.09</td>
<td>1.44</td>
<td>1.92</td>
<td>1.0</td>
<td>1.3</td>
</tr>
</tbody>
</table>

Table 2. Standard coefficients of the $k$-$\varepsilon$ model.

Parameter $c_{3\varepsilon}$ can have the value 1 for stable stratification and 0 for unstable stratification.

The transport equation for $\varepsilon$ reads:

$$\frac{\partial \varepsilon}{\partial t} + \frac{\partial (\varepsilon u_i)}{\partial x} + \frac{\partial (\varepsilon w)}{\partial z} = \frac{\partial}{\partial x} \left( \frac{\nu_k}{\sigma_\varepsilon} \frac{\partial \varepsilon}{\partial x} \right) + \frac{\partial}{\partial z} \left( \frac{\nu_k}{\sigma_\varepsilon} \frac{\partial \varepsilon}{\partial z} \right) + \frac{c_{1\varepsilon}}{k} P + (1 - c_{3\varepsilon}) \frac{\varepsilon}{k} P_b - c_{2\varepsilon} \frac{\varepsilon^2}{k} \quad (3.5)$$

Production – dissipation
Sediment transport module

The grain size distribution is approximated with a certain number of fractions. Ten fractions are usually enough to represent a full grain size distribution. Every fraction has a representing mean diameter and a mixture concentration. The total mixture concentration is composed of the mixture concentrations of all fractions combined together. The mixture velocities are used in the momentum equations, while the sediment transport equations use the grain velocity. The number of equations is equal to the number of fractions. The grain velocity in horizontal direction is equal to the mixture velocity of the suspension. The horizontal component of the mixture velocity is directly substituted in the sediment transport equations. In vertical direction a slip velocity is assumed. The slip velocity is the difference between the vertical fluid velocity and the vertical grain velocity. The slip velocity is influenced by the hindered settling effect (see Section 2.2.6).

Grain-grain interactions have no important influence on the most part of the flow field. Only the settling velocity of a certain fraction is influenced by other fractions. However, the interaction between grains has a substantial influence on the settling of particles in the seabed. The transport equation for a certain fraction in conservative form is defined as:

\[
\frac{\partial}{\partial t} \int_{\Omega} c_j \, d\Omega + \int_{S} c_j \bar{v}_{z,j} \cdot \bar{n} \, dS = \int_{S} \left( \frac{\nu_0}{\sigma} \nabla c_j \right) \cdot \bar{u} \, dS
\]  

(3.6)

With:
- \( \sigma \) = Schmidt-Prandtl number [-]
- \( v_{z,j} \) = \( w + \sum_{k=1}^{n} c_k v_{s,k} - v_{s,j} \) [m\cdot s^{-1}]
- \( v_{s,j} \) = \( w_{0,j}(1 - \bar{c})^n \bar{n}^{-1} \) [m\cdot s^{-1}]
- \( w_{0,j} \) = settling velocity of a single particle [m\cdot s^{-1}]
- \( c_j \) = concentration [-]
- \( n_j \) = hindered settling exponent [-]

The vertical velocity \( v_{z,j} \) of a particle sized \( D_j \) in a mixture of \( n \) different particle fractions is defined according to Mirza and Richardson (1979) and Van Rhee (2002). The slip velocity is denoted by \( v_{s,j} \).

3.2 Boundary conditions

Boundary conditions must be prescribed to solve the partial differential equations. Different boundaries are present: vertical walls, bed, water surface, inflow sections and outflow sections. The normal flow velocity is zero at the walls. The left and the right boundary are placed far enough away from the discharge location to avoid influences of these boundaries with the solution. The left and the right boundaries are (partly) acting as an outlet zone. The boundary condition for the flow velocity at the wall is computed using a wall function. The boundary conditions for the turbulent energy \( k \) and dissipation
rate $\varepsilon$ are consistent with this wall function approach. This method assumes that there is a relation between the velocity and the shear stress at a certain distance from the wall. The wall function is defined as:

$$
\tau_b = \frac{\rho u_Y^2}{\left(\frac{1}{\kappa} \ln \left(\frac{32Y}{k_s}\right)\right)^2}
$$

(3.7)

With:
- $\tau_b$ = shear stress on bottom cell [Pa]
- $Y$ = distance to wall (bed) of velocity grid point [m]
- $u_Y$ = velocity in the grid point close to the wall [m·s$^{-1}$]
- $\kappa$ = Von Kármán constant [-]
- $k_s$ = roughness height according to Nikuradse [m]

For the sediment transport equations, the fluxes through vertical walls and the water surface are equal to zero. No sediment enters or leaves the domain at these boundaries. The sedimentation velocity $u_{sed}$ determines the sedimentation or erosion rate at the seabed. The sedimentation velocity is defined as:

$$
u_{sed} = \frac{S}{(1 - n_0 - c_b)\rho_s} R(\theta)
$$

(3.8)

With:
- $S = \sum_{k=1}^{n} \frac{\rho_s \sum_{k=1}^{n} u_{z,k} c_{b,k}}{\theta}$ [kg·m$^{-2}$·s$^{-1}$]
- $R(\theta) = \begin{cases} 1 - \frac{\theta}{\theta_0} & \text{if } \theta < \theta_0 \\ 0 & \text{if } \theta \geq \theta_0 \end{cases}$ [-]
- $\theta_0 = \frac{u^2}{\Delta g \rho D}$ [-]
- $u_*$ = friction velocity, by definition $\sqrt{\tau_b / \rho}$ [m·s$^{-1}$]
- $\Delta$ = specific density of particles [kg·m$^{-3}$]
- $D$ = particle diameter [m]
- $\tau_b$ = bed shear stress [kg·m$^{-1}$·s$^{-2}$]

$S$ is the sedimentation flux for multi-sized mixtures. The influence of the bottom shear stress on the sedimentation is modelled using a reduction factor $R(\theta)$. The reduction factor is an empirical relation based on experiments (Van Rhee 2002). The reduction factor is a function of the Shields parameter $\theta$. The relation between the reduction factor and Shields parameter $\theta$ is based on flume tests (Van Rhee, 2002). The critical value for the Shields parameter proved to be independent of the grain size for the sands tested ($d_{50} < 300 \mu$m).
3.3 Numerical procedure

The momentum and sediment transport equations are solved using the Finite Volume Method (FVM), because conservation is very important. The finite volume method is a numerical method for solving partial differential equations. The method calculates the values of the conserved variables averaged across the volume. The values of the conserved variables are located within the volume element (grid cell), and not at nodes or surfaces. A detailed description of FVM is beyond the scope of this thesis. Reference is made to Ferziger and Perić (1999).

The transport equations for the turbulent quantities \( k, \varepsilon \) and \( \nu_\varepsilon \) are solved using the Finite Difference Method (FDM), because conservation is less important. The advantage of the FDM is the simplicity, however conservation is not ensured. The FDM is based upon the approximation that permits replacing of differential equations by finite difference equations. These finite difference equations are algebraic in form and the solutions are related to grid points.

A Finite Difference Method is always implemented on a fixed rectangular (Cartesian) grid (Figure 12). A Finite Volume Method can be applied on any grid. However, it is advantageous to use a Cartesian approach for this method, especially when a staggered arrangement of variables is used. The staggered arrangement of variables provides good conservation properties and avoids oscillations in the pressure field encountered when all variables are defined at the same grid point. To avoid a staircase shape representation of a sloping seabed, the size of the bottom cells are adjusted. This approach of adjusting the bottom cells is called the cut-cell method.

The pressure and concentration values are located at the centre of the grid cells (N, W, P, E, S in Figure 12). The concentration, pressure and mixture density are defined in the points with the closed circular symbols. The vertical velocity is defined in the points with the open rectangular symbols (n and s). The horizontal velocity is defined in the points with the closed rectangular symbols. The turbulent quantities \( k, \varepsilon \) and \( \nu_\varepsilon \) are defined in the points with the open circular symbols.
Figure 12. Staggered arrangement of variables. The pressure and concentration values are located at N, W, P, E and S. The fluxes are located at n, w, e and s. The symbols are defined as follows:

- Circle: Concentration, density and pressure
- Rectangle: Horizontal velocity
- Open rectangle: Vertical velocity
- Open circle: Eddy viscosity, $k, \varepsilon$

Overview of the computational procedure
A brief overview of the computational procedure is given. The next steps are repeated during time:

- Update of the flow field to time $t_{n+1}$. This is done by solving the Navier-Stokes equations together with the continuity equation. These equations are solved using a pressure correction method, which uses the mixture density and the eddy viscosity of the old time step ($t_n$).
- Update of the turbulent quantities to time $t_{n+1}$. This is done using the flow field of $t_{n+1}$ and the mixture velocity of $t_n$. This step generates the new eddy viscosity.
- Update of the concentrations for all fractions and the total mixture density to time $t_{n+1}$. This is done by using the flow field and the eddy viscosity of $t_{n+1}$ to compute the grain velocities for the next time step.
- Determine the new location of the bed level.
3.4 Validation of the 2DV model

The 2DV model should be validated before it is used to simulate the backfilling operation of an offshore trench. This validation should be carried out using experiments or prototype data. In this thesis no validation is carried out, since a validation for a comparable problem is already described in Van Rhee (2011). The data set used for this validation was found in Mastbergen and Winterwerp (1988). A test was executed in a flume of 32 m long, 2.5 m deep and 0.5 m width. At one side a sand-water mixture was discharged. The side wall of the flume contained a large area of glass windows. This allowed the registration of the sand transport phenomena and the slope development by video and photographic equipment. A representation of the experimental set-up is visualised in Figure 13. The sand-water mixture was discharged in the flume through a vertical pipe. The water level was kept constant using an overflow. This resulted in a two dimensional simulation of the sand-fill process.

![Figure 13. Representation of the experimental set-up of a 2D sand slope (from Mastbergen and Winterwerp, 1988)](image)

In Van Rhee (2011) a comparison is made between the experimental data (from Mastbergen and Winterwerp, 1988) and the computed results using the 2DV model. The model is used to predict the shape of underwater discharged sand bodies. The boundary conditions used in the 2DV model corresponded with the flume tests (Figure 13). The mixture concentration, sand bed porosity and specific sand production were consistent with the laboratory experiments. The result of this comparison is shown in Figure 14. This figure shows that there is a good agreement between the measured and the computed slope.

Earlier validations of the 2DV model are described in Van Rhee (2002). One part of this validation is based on multiple laboratory experiments and one part is based on measurements taken on board of the TSHD “Cornelia”.
Figure 14. Measured slope angles (experimental data from Mastbergen and Winterwerp, 1988) versus simulated slope angles (2DV model). Results experimental data (exp) versus calculated results (calc).
4 Implementation of the 2DV model

This chapter describes the implementation of the 2DV sedimentation model with respect to pipeline floatation during dredge-based backfilling. The general outline of the 2DV model is described in the previous chapter. The 2DV model was designed to model the sedimentation process inside a Trailing Suction Hopper Dredger (TSHD). However, in this study it is used to model the sedimentation inside a subsea trench.

The two most important characteristics that are investigated in this thesis are the Total Sedimentation Length (TSL) and the slurry density. The TSL is the necessary distance for a soil-water mixture to settle. The TSL is visualised in Figure 15. The TSL and the slurry density are expected to indicate if a pipeline is prone to floatation during dredge-based backfilling.

A large Total Sedimentation Length means that the backfill remains liquefied for a relatively long time. This may lead to pipeline floatation if the pipeline density is lower than the density of the surrounding soil-water mixture. The problem of pipeline floatation during-dredge based backfilling is illustrated in Figure 4.

Another important definition used in this thesis is the 90% sedimentation length. The 90% sedimentation length is defined as the horizontal distance covered by the sand-water mixture flow before 90% of the sand from the mixture is settled (Mastbergen, 1988). This definition is used because soil samples can contain a small fraction of fine material. These fines can have a large influence on the total sedimentation length, although they only represent a small fraction of the material. According to Mastbergen (1988) the sedimentation length is strongly influenced by the particle size, the particle shape, the initial mixture concentration, the initial flow rate and the fall height (distance between bottom trench and bottom suction pipe).
The backfilling process is studied in three steps. First, the sedimentation inside a column is simulated (Section 4.2). This is a one-dimensional sedimentation test. Secondly, the backfilling of a trench using a stationary TSHD is evaluated (Section 4.3). Finally, the backfilling process using a moving TSHD is simulated (Section 4.4). Section 4.1 evaluates the reference cases which provide the input for these simulations.

4.1 Reference cases

In this section theoretical reference cases are presented. These reference cases are used as input for the simulations in the next sections. The problem of pipeline floatation during dredge-based backfilling is relatively unknown, because the risk of floatation is avoided at all costs. Soil is usually not used as backfill material when it is not 100% certain that the soil is suitable for this purpose. Coarse material is supplied from elsewhere when the excavated material is considered as unsuitable. Unsuitable soils mostly contain a large fraction of relatively fine material.

Pipeline floatation is usually avoided. Still, there are some projects in which it occurred. Two of these projects (Kuwait and Europipe II) faced pipeline floatation during or short after backfilling. At one project (Varandey) it was decided that the excavated soil was not suitable as backfill material and soil was supplied from elsewhere. Analysing these projects and using their grain size distribution as input for the 2DV model can validate that pipeline floatation was indeed a major risk in these projects. Figure 16 presents the grain size distributions of the previously mentioned reference projects.

To obtain a more complete understanding of the problem it is also necessary to consider material in which pipeline floatation is highly unlikely. No cases of pipeline floatation are reported in the Markermeer region. Therefore the grain size distribution of a typically Markermeer sand is used. To complete the picture also three theoretical soil-water mixtures are evaluated. These theoretical mixtures consist of mono-sized particles. In this manner the basic behaviour of coarse and fine soil-water mixtures can be compared. The above-mentioned reference cases are evaluated in the next section.

- Kuwait
A pipeline was installed according to the specifications of the client. The backfilling operation was not successful. The survey, which took place shortly after the backfilling operation, indicated that the pipeline was located on top of the backfill material. This indicates that pipeline floatation had taken place during or shortly after backfilling. The backfill material is mostly composed of sand, silt and fine grained deltaic mud, according to the geotechnical investigation carried out by Fugro (Van der Halst, M., 1998).

- Varandey, Russia
The soil contains a fines content of 5 – 35%. The median grain size ($D_{50}$) varies between 78 and 112 μm. The low specific gravity of the pipeline in combination with the poor quality of the available backfill material results in a pipeline susceptible for uplift (Mathijsen, F., 2007). It was decided that the soil was not
suitable as backfill material. Therefore, coarse material was supplied from elsewhere.

Figure 16. Grain size distributions of reference projects and theoretical mixtures used as input for the 2DV sedimentation model.

- **Europipe II, North Sea**
The installation of the Europipe II pipeline was completed in 1999. For a length of 140 kilometres the pipeline was backfilled. This post-trenching was done by jetting. The Europipe II floatation took place in this part of the pipeline between the surveys in 1999 and 2000. The exposure of the Europipe II pipeline cannot be directly attributed to the pipeline jetting operation. However, the time interval (± 35 days) between trenching, in the flooded situation and dewatering may have been insufficient to restore the initial soil density and strength according to an assessment report of Snamprogetti (Tura et al., 2005).

- **Markermeer sand**
Next to the previous considered cases also another reference case is used in this thesis. The sand from this reference case is coarser than the previous considered soils. Therefore this Markermeer sand extends the range of considered material. The used grain size distribution originates from a report of Gemeente Amsterdam Ingenieursbureau (Meisner, 2006).

- **Theoretical soil-water mixtures**
The theoretical mixtures are examined to obtain a more complete picture of the sedimentation process. These mixtures contain mono-sized particles. Three particle sizes are used; 125 µm, 250 µm and 500 µm. In this manner it is possible to compare the general behaviour of coarse and fine material.
4.2 One dimensional tests

One dimensional (1D) sedimentation tests can be used as a first guess of the sedimentation time during the backfilling process of a subsea pipeline. The 1D tests are carried out using the 2DV sedimentation model. It is not a complete representation of all aspects of the backfilling process. For instance all flow is disregarded. However, these tests give an insight in the sedimentation time for a specific sediment-water mixture.

The purpose of the 1D sedimentation tests is to compare the sedimentation time scale of different soil-water mixtures. Sedimentation time is the time required for soil particles to settle out of suspension. Soil can contain a fraction of relative fine material. This fine material will remain in suspension for a long time compared to the coarser soil particles. Therefore, the total sedimentation time is not regarded. Only the period of time is regarded before 90% of the soil particles have settled out of the soil-water mixture (T_{90%}).

4.2.1 Model setup

The 1D sedimentation tests are performed in a closed box environment. This implies that the model has no in- or outflow boundaries. No sediment will enter or exit the model and there will be no flow velocity. At the start of each simulation, the concentration of solids is uniformly distributed over the column. This is shown in the column at the left of Figure 17.

![Figure 17](image)

*Figure 17. Typical results of a one-dimensional simulation of a sedimentation column at four time steps. The solids concentration is displayed by the colour bar at the right side of the figure.*

The solids concentration in the four columns is represented with different colours which correspond with the colour bar at the right side of the figure. The sedimentation column has a height of five meters and a width of one meter. Key in this set of simulations is the comparison of the sedimentation time for different soils. For all
simulations the same initial concentration of solids is applied. The applied initial volume concentration is 20%. Settled soil particles form a bed when a concentration of 60% is reached. This means that 90% of the sedimentation is completed when the sandbed reaches 1.5 m. Grain size distributions are applied as presented in Section 5.2.

Figure 17 shows an example of a 1D sedimentation column at four moments during a sedimentation test. The first column ($T_0$) of Figure 17 contains the initial soil-water mixture. The volume concentration is at every point of the column 20%. The second column ($T_1$) shows the sedimentation process further in time. Soil particles have already settled out of suspension and have formed a bed at the bottom of the column. The bed material is marked with a red-brown colour and the bed height is marked with a thin white line. The dark blue colour at the top of the column indicates clear water without suspended particles. Segregation of particles with a different grain size is visible. The coarse particles settle out relatively fast, while the fine particles remain longer suspended. Segregation is more detailed described in Section 3.14. The third column ($T_2$) shows the sedimentation process when 90% of the initial concentration has settled. In the fourth column ($T_3$) sedimentation has finished. The column contains only clear water and bed material. However, still some fine particles remain in suspension.

Figure 18 shows the implementation of the 1DV tests in the Schiffman (1998) model. A single sedimentation column passes through the same stages as the model proposed by Schiffman (1998).

![Figure 18. Visualisation of the 1DV sedimentation columns applied in the Schiffman (1998) model.](image)
4.2.2 Results

The one dimensional sedimentation tests give insight in the sedimentation time of the different soil-water mixtures. A soil-water mixture with a relatively large fraction of fine material has a relatively long sedimentation time. This is shown by the results of the Kuwait, the Varandey and the ‘diameter 125 μm’ simulations. A representation of all 1D sedimentation tests can be found in Appendix E. Relatively coarse material provides a relatively short sedimentation time. This is shown in the columns filled with coarser material: ‘diameter 500 μm’, ‘diameter 250 μm’, the North-Sea and the Markermeer simulations.

In this thesis it is assumed that sedimentation is completed when 90% of all material is settled out of suspension. Figure 19 shows the 90% sedimentation time ($T_{90\%}$) as function of the median grain sizes ($D_{50}$). These median grain sizes are derived from the grain size distributions in Figure 16 and correspond with the soil-water mixtures mentioned above.

\[
T_{90\%} = 2.92E6 \cdot D_{50}^{-1.66}
\]

Figure 19. Results of the 1DV sedimentation tests. Showing the 90% sedimentation time ($T_{90\%}$) as a function of the $D_{50}$ values which are derived from the grain size distributions in Figure 16.

The results of the 1D sedimentation tests are extrapolated in Figure 19. The extrapolation provides an indication of the sedimentation time outside the area of the simulated soil-water mixtures. The next equation can be derived from the results of the 1D sedimentation tests:

\[
T_{90\%} = 2.92E6 \cdot D_{50}^{-1.66} \quad (4.1)
\]

The solid line visualises the most reliable part of the extrapolation. This is the area between 63 μm and 2000 μm. The particles in this area can be regarded as sand. The extrapolation outside this area is less reliable and is marked with a dotted line. The 2DV model is not suitable to simulate the behaviour outside this area. However, the extrapolation is made to provide information on this area. The 2DV model cannot
simulate soil-water mixtures containing cohesive material. The model is also inaccurate when it has model mixtures with large particles. The empirical formula, derived from the results of the 2DV sedimentation tests, is only valid for an initial mixture density of 20%. This relation could be converted to a more generally applicable term if sailing velocity, initial mixture velocity, mixture density and the trench profile are taken into account.

It was not possible to perform a complete sedimentation test of the Kuwait soil-water mixture. The large fraction of fine material and the low median grain size (23 μm) resulted in an extremely long sedimentation time. Using Figure 19 it is possible to give an estimation of the sedimentation time (2.5 hours).

4.3 Static analysis

In this section, the discharge process of a stationary TSHD is considered. The backfill is placed using the suction pipe of a TSHD as discharge pipe. In this thesis the 2DV model is used to simulate the backfilling process of a pipeline in longitudinal direction. The modelled area is marked by the light blue rectangle in Figure 20. The dotted line marks the top of the trench. The top of the modelled area is defined by the bottom of the suction head. The lower boundary is defined by the trench bottom.

The static analysis is carried out to gain information on the Total Sedimentation Length (TSL) and the build-up height of the soil body. Figure 23 visualises the TSL and the build-up height. During these computations, erosion is not taken into account. Modelling erosion would result in a deep erosion pit. This erosion pit would influence the build-up of the soil body. Therefore the model is run until the soil-water mixture reaches the boundaries of the modelled area.

![Figure 20. Visualisation of the backfilling of subsea trench using a stationary Trailer Suction Hopper Dredger. The modelled area is marked by the blue rectangle. The dotted line indicates the top of the trench.](image-url)
Figure 21 shows the typical cross section of a trench. The computation plane of the modelled area is marked with the dotted line.

Figure 21. Cross section of a typical trench. The dotted line indicates the plane of Figure 20.

Figure 22 shows typical plots made using the 2DV model. The area displayed in red-brown has the highest concentration of particles, while the dark blue area contains no soil particles at all. The height of the formed bed is marked with a white line. Beneath this line the soil-water mixture is regarded as soil.

Figure 22. Example of the contour plot of a typical static simulation. The height and width of the modelled area are displayed on respectively the vertical and horizontal axis. The two axes are not on the same scale. The solids concentration is displayed by the colour bar on the right side of the figure.
4.3.1 Model setup

In this section the configuration of the 2DV model is described. All necessary considerations to achieve a realistic simulation of the backfilling of an offshore trench are explained.

**Discharge pipe**
The backfill is placed using the suction pipe of the TSHD as discharge pipe. The results of a simulation with or without a discharge pipe show only small differences. It is therefore not necessary to actual model this suction pipe in the 2DV model. This is explained in Appendix A.

**Boundaries**
The boundaries of the model have a large influence on the behaviour of the pressure and the flow inside the model. The model is designed for sedimentation inside a TSHD. The model is simplified as a box. A trench does not look like a box. Therefore it is necessary to make sure that the wall of the model is open, so that pressure can leave the model. It is also required to make the model large enough to prevent that the soil-water mixture reaches the boundaries. When the soil-water mixture reaches the boundary, this will result in a large accumulation of sediment at this location.

**Height**
In practice most trenched pipelines are laid at relatively shallow depth. This means that the seabed is somewhere between 0 to 50 meters below sea level. However, it is not necessary to model this total height. It is enough to model the distance from the bottom of the suction pipe to the bottom of the trench. At a recent pipeline project in the Pomeranian Bay area, Germany (part of Nordstream), distance between the top of pipe (TOP) and the bottom of the drag head was three meters.

**Initial mixture velocity**
It is important to keep the mixture velocity as low as possible when it leaves the suction pipe. The goal of the backfilling operation is placing material inside the trench. The rate of lost material should be minimal. Also erosion of the bed material is a potential problem, when the mixture speed is too high. Generally it is possible to backfill with an initial mixture velocity between 4 – 5 m/s. It is also required to keep the mixture speed above the critical value of 3.5 – 4 m/s to prevent clogging of the pipes of the TSHD. The simulations of the 2DV model are carried out using an initial mixture velocity of 4 m/s.

**Concentration**
To obtain high production rates it is desirable to have a soil-water mixture with a high solids concentration. A high solids concentration also helps emptying the TSHD in the shortest amount of time. In general, a soil-water mixture with an average density between 1.3 and 1.4 t/m$^3$ will be discharged back into the trench. The average of these densities results in a volume concentration of roughly 20%. Most simulations in this study use this value.
Sand bed porosity
The porosity of the formed bed is in the 2DV model called sandbed porosity. In reality the bed porosity is influenced by the next factors:
- Particle shape (roundness)
- \( C_u \left( \frac{D_{60}}{D_{10}} \right) \)
- Sand dominated/ silt dominated

The sandbed porosity used in the tests is 40%. This value is corresponding with the porosity value of standard marine sands.

4.3.2 Results

Comparison of grain size distributions

Firstly the behaviour of different soil-water mixtures is compared. All parameters are kept constant, except for the grain size distributions. The static simulations provide information of the Total Sedimentation Length and the build-up height of the sand body. Figure 23 visualises the backfilling process of a stationary TSHD. The Total Sedimentation Length, as used in the static simulations, is defined as the distance from the centre of the discharge point (discharge pipe in reality) to the end of the sand body. The TSL is indicated in Figure 23.

![Figure 23. Visualisation of the backfilling process of a stationary Trailing Suction Hopper Dredger. The Total Sedimentation Length (TS) is defined as the distance from the discharge point to the end of the soil body. Build up height is distance from the bed to the top of the soil body.](image)

Figure 24 shows the TSL as a function of the runtime of the 2DV model. The legend at the right side of the figure shows the different soil-water mixtures used during these static simulations. These mixtures are ordered by size. The ‘500 µm’ mixture has the largest median grain size and the ‘Kuwait’ mixture has the smallest grain size. The simulation is stopped when the soil-water mixture reaches the boundaries or when soil-mechanical behaviour becomes more important than hydraulic behaviour.
Figure 24 reveals that the coarse soil-water mixtures settle closer to the discharge point than the relatively fine mixtures. This results in a short TSL for the coarse mixtures and longer TSL for the finer mixtures. Initially the TSL increases for all mixtures at the same rate. This implies that in the beginning the TSL is independent from the grain size.

However, at a given point the rate at which the TSL increases slows down. This moment is different for every soil-water mixture. All parameters, except for the grain size distribution, are kept constant. It can therefore be concluded that the TSL is related to the grain size distribution and/or the median grain size. Figure 24 illustrates that the increase in TSL slows down first for soil-water mixtures with a larger median grain size.

Figure 24. Results of the static 2DV sedimentation tests. Showing the TSL as a function of the runtime of the model. In these simulations soil-water mixtures with different grain size distribution are compared.

The build-up height is the maximum height of the sand body at a given moment, which is visualised in Figure 23. Figure 25 shows the build-up height as a function of the runtime of the 2DV model. This graph makes clear that the build-up height of the sand body of a certain soil-water mixture remains constant in time. The build-up height increases linearly over time, because at a random place along the sand body settles the same amount of soil particles at any given time. This changes when the angle of the soil body becomes larger. In this case the soil-water mixture velocity increases and the equilibrium is disturbed.
Figure 25. Results of the static 2DV sedimentation tests. Showing the build-up height as a function of the runtime of the model. In these simulations soil-water mixtures with different grain size distribution are compared.

The legend at the right side of the Figure 25 shows the same arrangement of soil-water mixtures as in Figure 24. It seems obvious that the mixtures with the larger median grain size build up faster than the mixtures with a smaller median grain size. This statement is true for all mixtures, except for the ‘Varandey’ and the ‘Kuwait’ mixture. These mixtures build up faster than expected, because they contain a substantial fraction of coarse material. This coarse material settles faster and forms a mound of sand relatively close to the discharge point (this is shown in Appendix D). This disturbs the build-up rate for these soil-water mixtures, since the maximum height of the sand body is measured.

Figure 26 shows the TSL divided by the build-up height as a function of median grain size of the soil-water mixtures. This graph shows the results of the seven simulations which are previously mentioned. The TSL and the build-up height of each simulation are measured after 2 minutes and 4 minutes.

There seems to be a correlation between the median grain size and the TSL divided by the build-up height. The value of this dimensionless parameter becomes larger with decreasing median grain size. However, there is one value that does not fit in this regime; the mono-sized ‘125 µm’ sample. This is caused by the fact that the finer soil-water mixtures contain a fraction of coarse material which dominates the build-up height. This is represented in Figure 27.

Relative fine soil-water mixtures can be applied as hydraulic backfill when they contain a coarse fraction. Examples of such mixtures are the ‘Varandey’ and the ‘Kuwait’ mixture. The sedimentation behaviour of these mixtures is illustrated in Figure 27 a. During these sedimentation tests the coarse fraction sedimented close to the inflow point, while the fine fraction sedimented further away. This segregation can be used during backfilling. Theoretically, a pipeline can be hold down by the coarse fraction, while the fine fraction settles down.
Figure 26. Results of static 2DV sedimentation tests. Showing the TSL divided by the height as a function of the median grain size. In these simulations seven soil-water mixtures with different grain size distribution are compared. The TSL and the build-up height of each simulation are measured after 2 minutes and 4 minutes. The grain size distributions are taken from Figure 16.

Figure 27. Visualisation of the typical results of the static sedimentation tests. Showing the build-up height as a function of the sedimentation length. Only one half is shown (compare with Figure 22 for the total picture). Figure ‘a’ shows the result of relatively fine soil-water mixture with a coarse fraction (Varandey, Kuwait). Figure ‘b’ shows the result of a relatively fine mono-sized soil-water mixture (‘125 µm’ sample).

Comparison of initial mixture concentrations

Figure 28 shows the TSL as a function of the runtime of the 2DV model. This graph shows the results of four simulations of the same soil-water mixture (‘250 µm’) with four different initial concentrations. The legend at the right side of Figure 28 lists the used soil-water mixtures. The mixtures are listed by the initial volume concentration, ranging from 10% to 40%.

Figure 28 reveals that soil-water mixtures with a high initial concentration settle further from the inflow point than mixtures with a low initial concentration. This corresponds with the theory on TSL in Section 2.2.9.
Figure 28. Results of the static 2DV sedimentation tests. Showing the TSL as a function of the runtime of the model. In this graph soil-water mixtures with different initial concentrations are compared.

Figure 29 shows the build-up height as a function of the runtime of the 2DV model. This graph represents the results of three sets of four simulations. Each set of simulations is carried out for a mixture with an initial volume concentration of 10%, 20%, 30% and 40%.

The results reveal that the build-up height of each soil-water mixture increases at a constant rate. The different grain sizes show the same behaviour as in Figure 25. The initial volume concentration does not influence the build-up rate.

Figure 29. Results of the static 2DV sedimentation tests. Showing the build-up height as a function of the runtime of the model. These simulations are carried out for three theoretical mono-sized samples of different grain size (‘500 µm’, ‘250 µm’ and ‘125 µm’). For each of the mono-sized sample four mixtures the same initial concentrations (10%, 20%, 30% and 40%) are compared.
4.4 Moving hopper

In this section the discharge process of a moving TSHD is analysed. This analysis is carried out to gain knowledge of the behaviour of soil-water mixtures during backfilling operations. The 2DV sedimentation model is used to simulate this process. The 2DV model is usually used to model the sedimentation process inside a hopper. In the 2DV model the hopper is simplified to a box. Sediment enters the model through one or more fixed inflow points. Therefore, the model was originally not equipped with a moving inflow point. However, to simulate the backfilling process it is necessary to implement a moving inflow point in the 2DV model, since the TSHD moves during backfilling. There are basically two approaches to implement a moving inflow point in the 2DV model. A moving inflow point can be implemented by placing multiple inflow points next to each other and let them discharge one after another. This is explained in more detail in Section 5.4.2. A different approach to implement a moving inflow point is by placing an actually moving inflow point in the model. This is explained in more detail in Section 4.4.3.

The 2DV model is applied in longitudinal direction. This is visualised in Figure 30. The blue rectangle indicates the modelled area. At the start of the simulation there is already an existing seabed. This seabed is actually the trench bottom. This is shown in Figure 21.

![Figure 30. Visualisation of the backfilling of subsea trench using a (moving) Trailer Suction Hopper Dredger. The modelled area is marked by the blue rectangle. The dotted line indicates the top of the trench.](image)

4.4.1 Models setup

The model setup of the moving hopper analysis is for the most part the same as the setup for the static tests in Section 4.3.1. The simulations are carried out with closed boundaries, except for the largest part of the vertical walls. Momentum and sediment can be transported through these open walls. Suspended sediment can leave the model area through these open boundaries. However, a substantial part of the material will be
reflected back into the model. Therefore it is important that the vertical boundaries are set up wide enough to avoid this. The horizontal scale required for these simulations is approximately 1500 – 2000 meters and depends on the mixture velocity, volume concentration, grain size distribution (especially the amount of relative fine material), trailing speed and the distance travelled by the TSHD. The used horizontal scale for all simulations is 1700 meter. In these simulations all parameters are kept constant except for the grain size distribution. The trailing speed of the TSHD (the speed at which the inflow point moves) is set at 1 m/s. This is a conventional speed at which a typical subsea trench can be filled.

To avoid sediment reaching the vertical boundaries it is necessary to keep 800 meters between the first inflow point and the left boundary. The last inflow point is located at 500 meters from the right boundary.

Figure 30 shows the grain size distributions used in the “moving hopper” simulations. The Kuwait grain size distribution is not applied in these simulations, because this material reaches the vertical walls too soon. Two artificial grain size distributions are applied to widen the analysed spectrum.

![Figure 31. Grain size distributions of reference projects and theoretical mixtures used as input for the moving hopper analysis.](image)

A potential problem for these simulations is the grid size. To achieve the desired mixture behaviour in vertical direction it is required to have relatively small grid cells (in vertical direction) at the top of the model and at the seabed. To obtain a realistic simulation it is desirable to have square grid cells. However, due to the large difference in horizontal and vertical scale it is also preferred to reduce the number of grid cells. This results in grid cells which are small in vertical direction, but relatively large in horizontal direction. These stretched rectangular grid cells increases the risk of numerical instability.
4.4.2 Multiple fixed inflow points

The method of the ‘multiple fixed inflow points’ is described in Figure 32. Due to restrictions in the 2DV model it is only possible to define 50 inflow points. To model the backfilling process over 500 meters it is necessary that each inflow point has a length of 10 meters. The initial concentration is the same as in Section 4.3.2. It is therefore required to alter the inflow velocity in such a way that the sediment influx is the same as it would be if the inflow width was one meter.

The sediment inflow progresses at a virtual trailing speed of 1 m/s, starting at inflow point 1 and ending at point 50 (or point 10 in Figure 32). The inflow points are basically used sequentially. This is more thoroughly described in Appendix B.

![model space](image)

*Figure 32. Visualisation of the backfilling of subsea trench using a moving Trailer Suction Hopper Dredger. The modelled area is marked by the blue rectangle. The dotted line indicates the top of the trench. The numbered sections indicate the fixed inflow points, which are turned on sequentially.*

Figure 33 shows the typical plot of a simulation using the ‘multiple fixed inflow points’ approach. This plot provides information which is used as input for the Matlab model in Chapter 6. This model calculates the displacement of the pipeline due to the larger density of the horizontal mixture flow. Information on slurry density is therefore required.

![Typical plot of the 2DV model using the ‘Multiple fixed inflow points’ method](image)

*Figure 33. Typical plot of the 2DV model using the ‘Multiple fixed inflow points’ method. The solids concentration is displayed by the colour bar at the right side of the figure. The bed height is defined by the with line.*
Figure 34 shows the build-up height as a function of the runtime of the 2DV model. The legend at the right side of Figure 33 shows the different soil-water mixtures used during these static simulations. These mixtures are ordered by size. The ‘500 µm’ mixture has the largest median grain size and the ‘Varandey’ mixture has the smallest grain size.

It seems obvious that the mixtures with the larger median grain size build up faster than the mixtures with a smaller median grain size. This statement is true for all mixtures, except for the ‘Varandey’. This mixture builds up faster than expected, because they contain a substantial fraction of coarse material. This coarse material settles faster and forms a mound of sand relatively close to the discharge point (this is shown in Appendix F). This disturbs the build-up rate for these soil-water mixtures, since the maximum height of the sand body is measured.

![Figure 34. Results of the “multiple fixed inflow points” simulations. Showing the build-up height as a function of the runtime of the model. In these simulations soil-water mixtures with different grain size distribution are compared.](image)

### 4.4.3 Moving boundary

The ‘moving boundary’ method has basically the same approach as the ‘multiple fixed inflow points’ method. However, there are some distinctive differences. The ‘multiple fixed inflow points’ approach, introduced in Section 4.4.2, required an extensive amount of input and is therefore prone to mistakes. The ‘moving boundary’ method requires very little input. The ‘moving boundary’ method is illustrated in Figure 35. Sediment enters the model through a moving inflow point which starts at point ‘a’ and progresses at trailing speeds (1 m/s) to point ‘b’. This is more thoroughly explained in Appendix B.
Figure 35. Visualisation of the backfilling of subsea trench using a moving Trailer Suction Hopper Dredger. The modelled area is marked by the blue rectangle. The dotted line indicates the top of the trench. The moving boundary method consists of one moving inflow point which moves from point ‘a’ to ‘b’ at a given speed.

Figure 36 shows the typical plot of a simulation using the ‘moving boundary’ method. The results of the ‘moving boundary’ method are different from the results of the ‘multiple fixed inflow points’ method. In the ‘moving boundary’ method soil particles linger at the top of the modelled area. Even while the grid size and the size of the inflow point are the same as they are in the method of the ‘multiple fixed inflow points’.

Figure 36. Typical plot of the 2DV model using the ‘moving boundary’ method.

The density of the horizontal mixture flow at bed level is significantly lower as with the method of the multiple inflow points. This is caused by the fact that the vertical mixing of sediment is overestimated in the moving boundary method. The buoyancy of the turbulence option in the 2DV model is not used and therefore there is no vertical mixing of sediments. The ‘moving boundary’ method in combination with the buoyancy of turbulence caused instabilities in the 2DV model. Therefore the buoyancy of turbulence is not used together with the ‘moving boundary’ method.

The ‘moving boundary’ method is supposed to generate comparable results as the ‘multiple fixed inflow points’ method. However, there are some remarkable differences. Soil particles remain lingering at the top of the modelled area. The build-up
of soil is extremely fast. The density of soil-water mixtures never increases, even when relative fine soils are applied. The ‘moving boundary’ method has some errors which need to be corrected before this method can be applied.

4.4.4 Comparison of the Moving boundary and the Multiple fixed inflow points method

The two different approaches to simulate the backfilling process of a moving TSHD generate different results. The method of the multiple inflow points produces a density current at bed level, while the moving boundary method produces no density current. The simulated soil-water mixture has a large fraction of relative fine material. Therefore, a density current or a mixing layer is expected. However, the results of the moving boundary method show an abrupt transition from the initial mixture density to the deposited sediment. The method of ‘multiple fixed inflow points’ generates in this respect more satisfying results. However, this method has some disadvantages. The required input is extensive and therefore prone to mistakes, especially when you compare it to the moving boundary method. The ‘moving boundary method’ has to be corrected before it can be applied as a reliable method. Therefore, the results from the ‘multiple fixed inflow points’ are used as input in Chapter 5.
5 Pipeline floatation

During dredge-based backfilling, pipeline floatation can occur. If the applied backfill remains liquefied over a (too) large distance, the pipeline will float. A pipeline is only susceptible for floatation if its density is lower than the density of the surrounding soil-water mixture. The problem of pipeline floatation during dredge-based backfilling is illustrated in Figure 4. This chapter provides an approach to analyse pipeline displacement at a certain moment during the backfilling of a pipeline.

Pipeline displacement is analysed using a Matlab script which is based on the theory of the beam on elastic foundation. In this model only static loads are considered; the own weight of the pipeline and the buoyant force caused by the displaced fluid. Dynamic loads due to waves and currents are not considered. Risk of damage is low, since pipelines are usually installed during relatively calm weather.

Pipeline floatation is modelled using the beam-model (Matlab). The theory used in the beam-model is introduced in Section 5.1. The beam-model is described in Section 5.2. Analyses made using the beam-model are evaluated in Section 5.3, 5.4 and 5.5. The beam-model is used to analyse the influence of the soil-water mixture density in Section 5.3. The influence of the length of the soil-water mixture is evaluated in Section 5.4. The backfilling of an offshore pipeline is analysed in Section 5.5.
5.1 Beam on elastic foundation

The theory of the elastic supported beam is introduced by Winkler (1867). It is assumed that the beam (pipeline) is placed on an elastic foundation. The reaction forces of the foundation are proportional at every point to the deflection of the beam at this point. Interaction at the soil-pipe interface can be modelled as an elastic spring as long as the relative displacement is less than the maximum elastic deformation. The vertical deformation of the foundation is defined by means of identical, independent and linearly elastic springs. The stiffness ratio of these springs is known as the modulus of subgrade reaction (or vertical bedding constant) $k_v$. Figure 37 gives a representation of the beam on elastic foundation theory. This figure also provides the corresponding cross section.

![Figure 37](image)

Figure 37. Pipeline represented by the beam on elastic foundation. At the right side of the figure a cross section of a typical offshore trench is showed.

The Winkler model is very simple but has some deficiencies. One of the most important deficiencies of the Winkler model is that a displacement discontinuity appears between the loaded and unloaded part of the foundation surface. This is illustrated in Figure 38. In reality the soil surface does not show any discontinuity. In the model introduced by Hetényi (1946), interaction between the independent spring elements is accomplished by incorporating an elastic beam. The equation proposed by Hetényi reads:

$$\frac{E}{l} \frac{d^4 w(i)}{dx^4} = -k_v w$$

(5.1)

With:  
E = Young’s modules of the pipe material  
l = moment of inertia  
w = vertical displacement  
x = coordinate along the pipeline axis  
k_v = vertical bedding constant

![Figure 38](image)

Figure 38. Deflections of elastic foundations under uniform pressure. The left figure is a representation of the Winkler foundation. The right figure is a visualisation of the model proposed by Hetényi (1946).
Reaction forces of the subgrade act vertically and opposing the deflection of the beam. If the deflection acts downward, the supporting medium will be compressed. At locations where the deflection acts upward, there will be no contact with the subgrade. The upward deflection is caused by the buoyant force which acts on the pipeline. A visualisation of a deflected pipeline is shown in Figure 39. The buoyant force is determined by the density of the displaced fluid and the dimensions of the pipeline. The density of the soil-water mixture depends on the solids concentration.

Figure 39. Visualisation of the buoyant force acting on a pipeline.

5.2 Beam-model

The pipeline displacement is simulated using a Matlab script. This script can be found in Appendix C. This model consists of two components. One component is activated if there is a downward deflection of the pipeline. A downward deflection exists if the own weight of the pipeline is larger than the buoyant force. In this component the downward deflection is reduced, because the supporting foundation is compressed. This is showed at the right side of Figure 40. The other component is activated if there is an upward deflection of the pipeline. An upward deflection exists if the buoyant force exceeds the own weight of the pipeline. This will result in pipeline floatation, if the length at which this force is active, is large enough. This is shown at the left side of Figure 40.

Figure 40. Visualisation of the beam-model. The soil-water mixture has a higher density than the pipeline which causes a buoyant force on this pipeline.
5.2.1 Overview of the model

The beam-model has no time element. Therefore, it can not analyse the total backfilling process. The model can only analyse a certain moment during the backfilling operation. The 2DV model is used to provide the sediment distribution. This distribution is used as input for the beam-model. Since, there is no element of time in the model, this distribution is static and does not change.

The pipeline is equally divided in 200 elements. Each element is connected with a spring to the earth. These springs only function when they are compressed. This implies that any upward movement of an element is not hindered by the related spring. However, the movement of a single element is influenced by the connecting elements. This ensures that the total pipeline displacement is smooth and has no discontinuities.

The model performs 100,000 iterations during each simulation. The displacement of each element is calculated during each of these iterations. The displacement of a single element is influenced by the properties of the pipeline, the soil-water mixture density, the bedding constant, and the displacement of nearby elements. Furthermore, the soil-water mixture density is distributed as function of the length of the pipeline $x(i)$ and the displaced height $w(i)$. The mixture density can vary over the length and the height of the modelled area. This means that a pipeline may encounter different mixture densities when it moves upwards.

5.2.2 Model setup

In this section the configuration of the beam-model is described. All necessary considerations to achieve a realistic simulation of the pipeline displacements are explained. The described model setup is relevant for the analyses in Section 5.3, 5.4 and 5.5.

**Pipeline**

The considered pipeline has a diameter of one meter and a wall thickness of 50 millimetres. It is assumed that the pipeline is completely made out of steel. A pipeline with these dimensions has a specific gravity of 1.25 (in seawater). The pipeline is therefore heavier than seawater. A subsea pipeline has normally a thinner wall. To acquire the same specific gravity a concrete casing is placed around the pipeline to add weight. However, in this study the concrete casing is not considered to simplify the calculation.

**Soil-water mixture density**

The soil-water mixture density distribution is different for all analyses. The mixture density depends on the density of the (sea)water and the percentage of solids. It is assumed that the seawater has a constant value of 1025 kg/m$^3$. The solids have an assumed density of 2650 kg/m$^3$. This density equals the density of sand (quartz). For instance a solids concentration of 20% has a mixture density of 1350 kg/m$^3$.
Model length
The length of the modelled pipeline is 2000 m. This value is a little larger than the modelled length in the 2DV model. This length is chosen to avoid problems at the boundaries of the model.

Bedding constant
The uppermost layer of the seabed is affected by a combined loading of waves and currents. Therefore the seabed usually consists of relatively loose sand. According to CUR 166 the horizontal bedding constant $k_h$ for loose sand is $12000 \text{ N/m}^2$. Due to gravity, the soil is stiffer in vertical direction. The soil is also stiffer because the soil is completely saturated. The vertical bedding constant $k_v$ is therefore higher. In this study it assumed that the value of parameter $k_v$ is about $35000 \text{ N/m}^2$.

Other parameters
To achieve a realistic evaluation of pipeline displacement due to dredge-based backfilling, more parameters are required. The parameters, which are not mentioned in this section, are more or less standard. These values are kept constant during all simulations. Examples of these parameters are the density and elasticity modulus of steel. These parameters can be found in Appendix C.

5.3 Variation in fluid density

This section evaluates the influence of soil-water mixture density in the beam-model. A series of tests is made with the same model setup. Only the mixture density is varied. The mixture is applied over a distance of 500 m and the height of the mixture is not defined. The soil-water mixture density varied between two boundary values. Starting point is a mixture without solids (0%). The density of this mixture equals the density of clear seawater. The mixture density is for each simulation increased with one percent. The second boundary value is reached when the pipeline is lifted from the soil.

5.3.1 Results

Firstly, the situation of a not-backfilled pipeline is evaluated. This is illustrated in Figure 41. In this case there will be no upward deflection, because there is no buoyant force. The own weight of the pipeline is larger than the weight of the seawater. The resulting downward force will cause pipeline displacement. This force is uniformly distributed along the total length of the pipeline. This generates a uniform displacement.
Figure 41. Example of a not-backfilled pipeline submerged in seawater. The seabed level is located at 0 m (the vertical axis shows only the location of the pipeline).

Figure 42 shows the simulation of a pipeline backfilled with a soil-water mixture with the same weight as the pipeline itself. The backfill is placed between two locations along the horizontal axis (800 and 1300 m). This backfill generates virtually no pipeline displacement. Outside this area the pipeline is surrounded by seawater and the pipeline behaves similar to Figure 41.

Figure 42. Example of a pipeline which is backfilled with a soil-water mixture with a solids concentration of 28%. The weight of the soil-water mixture almost equals the weight of the pipeline. The backfill is present between 800 and 1300 m along the horizontal axis. The backfill has no defined height.

Figure 43 shows the simulation of a pipeline backfilled with a soil-water mixture which is heavier than the pipeline. The backfill is placed at the same locations as the previous example. This backfill generates a significant pipeline displacement of 3.8 m. Outside the area in which the pipeline has displaced, the pipeline is surrounded by seawater and the pipeline behaves similar to Figure 41.

Figure 43. Example of a pipeline which is backfilled with a soil-water mixture with a solids concentration of 29%. The displaced mixture is heavier than the weight of the pipeline. This causes a pipeline displacement of 3.8 m. The backfill is present between 800 and 1300 m along the horizontal axis. The backfill has an unlimited height.
The pipeline in Figure 42 is backfilled with a soil-water mixture with a solids concentration of 28%. The weight of this soil-water mixture is just slightly lower than the weight of the pipeline. This makes it impossible for the pipeline to float. The soil-water mixture applied on the pipeline in Figure 43 has a solids concentration of 29%. The weight of this mixture is slightly larger than the weight of the pipeline. However, this difference in weight is enough to cause a large displacement.

The difference in pipeline behaviour between Figure 42 and Figure 43 seems remarkable. However, the influence of the stiffness of the pipeline is low when a soil-water mixture is applied over a distance of 500m. The only thing, which keeps the pipeline from floating is the own weight. So if this mass balance changes to the wrong direction, this would lead directly to pipeline floatation.

Figure 44 visualises the results of the beam-model. The pipeline displacement is plotted as a function of the solids concentration. The weight of the pipeline is also included in the graph and is indicated with the dotted line. At the top of the graph also the weight of the displaced soil water mixture is shown. The mixture is applied to a height of 3 meters.

![Graph showing pipeline displacement as function of solids concentration](image)

**Figure 44.** Pipeline displacement as function of the percentage of solids. The weight of the pipeline is indicated with the dotted line. The weight of the displaced soil-water mixture is shown at the top of the graph. The mixture is applied to a height of 3 meters. The weight of the pipeline is 11.48 kN/m.

The pipeline displacement increases linearly between a solids concentration of 0% and 28%. Between 28% and 29% there is a sudden increase in pipeline displacement. The location of this increase is defined at the place where the weight of the pipeline equals the weight of the surrounding soil-water mixture.
5.4 Fluidised length

This section evaluates the influence of the length of the soil-water mixture (or fluidised length) in the beam-model. Four series of tests are made with the same model setup. Each series of tests is performed with a different soil-water mixture. During each series of tests the density is kept constant and the length of the soil-water mixture is varied. The length soil-water mixture is varied between two boundary values. Starting point for each test is a mixture length of zero meters. The mixture length is for each simulation increased with 10 meters. The second boundary value is reached when the pipeline is lifted from the seabed.

5.4.1 Results

In this set of simulations soil-water mixture density is kept constant and the length of the applied backfill is variable. These simulations will therefore provide information on the pipeline displacement as a function of the length of the soil-water mixture.

Figure 45 shows the typical result of these simulations. The applied soil-water mixture density has solids concentration of 30%. The displaced weight of this mixture is larger than the weight of the pipeline. This pipeline is therefore prone to flotation. However, in this example the length of the applied soil-water mixture (100 m) is not large enough to cause significant pipeline displacement. The stiffness of the pipeline prevents a larger displacement. The pipeline has moved 5 cm upwards. This seems a small displacement, however small displacements over a large distance can eventually lead to an exposed pipeline.

![Graph showing pipeline displacement](image)

**Figure 45.** Example of a pipeline at which a backfill is applied with a length of 100 m. The soil-water mixture has a solids concentration of 30%. The observed pipeline displacement is 5 cm (above the seabed).

Figure 46 shows an example of pipeline which has experienced significant displacement. The soil-mixture concentration used in this example is 30%. The soil-water mixture is applied over a length of 300 m. This causes a pipeline displacement of 1.5 m. The stiffness of the pipeline is apparently not enough to prevent this displacement.
Figure 46. Example of a pipeline at which a backfill is applied with a length of 300 m. The soil-water mixture has a solids concentration of 30%. The pipeline displacement is 1.5 m (above the seabed).

The results of the four series of tests are combined in Figure 47. The pipeline displacement is plotted as a function of fluidised length. The soil-water mixtures, in the four series of simulations, have a solids concentration of respectively 40%, 30%, 29% and 28%.

Figure 47. Pipeline displacement as function of the length of the applied backfill. This graph shows the results of four series of tests. With a solids concentration of respectively 40%, 30%, 29% and 28%.

The previously observed difference in behaviour between the mixture with a solids concentration of 29% and 28% percent is also present in these simulation. The 28% mixture does not experience pipeline floatation regardless of the fluidised length. However, the 29% percent mixture does experience floatation.

Notable is also the difference in behaviour between the 40%, 30% and 29% mixtures. A larger concentration of solids provides a larger weight of the mixture, which will make the pipeline more prone to floatation. A larger weight of the soil-water mixture provides a larger force acting on the pipeline. The bending stiffness of the pipeline is in this case sooner insufficient to prevent floatation compared to lighter mixtures.
Using only the properties of the pipeline, there are two related approaches to make pipelines less prone to pipeline floatation during dredge-based backfilling. The first approach is increasing the pipeline weight. If the pipeline is heavier than the surrounding soil-water mixture, there is no risk of floatation. The second approach is increasing the bending stiffness of the pipeline. The bending stiffness is related to the elasticity modulus of the material used to construct the pipeline and the moment of inertia. The elasticity modulus can be increased by using high strength steel. However, this material is usually not economical feasible.

5.5 Backfilling offshore pipeline

In this section the beam-model is used to simulate the backfilling of an offshore pipeline. It must be noted that this model has no time component. This implies that only a certain moment of the backfilling process can be observed and not the total dynamical process. The results of the 2DV model (‘multiple fixed inflow points’ method) are used as input for the beam-model.

5.5.1 Model setup

The model setup for these simulations is equal to the setup described in section 5.2.1, except for the soil-water density.

Soil-water density
The results of the ‘multiple fixed inflow points’ simulations (Section 4.4.2) are used to generate input for the beam-model. The density of the soil-water mixture is not constant over the entire modelled area. For instance, the slurry density will be higher close to the bed surface and lower close to the inflow point. The results from the ‘multiple fixed inflow points’ simulations are converted to a grid as displayed in Figure 48. With this approach, it is possible to incorporate the different densities as a function of the horizontal distance x(i) and the vertical pipe displacement w(i).
Figure 48. Example of a typical density distribution of a soil-water mixture over the modelled area. This density distribution is based on the results displayed in Figure 33. A concentration of 0% corresponds with a density of 1025 kg/m$^3$ (seawater). A concentration of 20% and 45% corresponds with a soil-water mixture density of 1350 kg/m$^3$ and 1756 kg/m$^3$ respectively.

Figure 49 visualises the grid from Figure 48 combined with the results from Figure 33. The used grid seems very coarse. However, this is just an example to explain the used approach. The grid can be refined if this is necessary.

Figure 49. Example of a typical grid displayed on the result of the corresponding 2DV simulation (Figure 33).

### 5.5.2 Results

Figure 50 shows a result of the beam-model. The input for this simulation is acquired from a typical 2DV sedimentation test using the ‘multiple fixed inflow points’ method. The results from this 2DV sedimentation test are converted to a grid similar to Figure 48. This grid distributes the density of the soil-water mixture over the length and height of the modelled area.
Figure 50. Example of a typical result of the beam-model. The distribution displayed in Figure 48 is used as input for this calculation. The seabed level is located at 0 m.

Next, the simulated pipeline displacement is verified. This verification is done by combining the results of Figure 50 and Figure 33. A visualisation of this verification is shown in Figure 51. This figure shows that upward pipeline displacement is present at places of high solid concentration. At locations with low solid concentration, upward pipeline displacement is minimal. Locations which are not covered with backfill, experience a downward deflection.

Figure 51. Visualisation of the result displayed in Figure 50 combined with the result of the ‘multiple fixed inflow points’ analysis in Figure 33. The seabed level in this figure is 0.5 meters higher compared to the seabed level in Figure 50.

It can be concluded that the beam-model provides a tool which can predict pipeline floatation. Before using the beam-model, it is necessary to acquire information on the distribution and evolution of the soil-water mixture. The beam-model simplifies the backfilling process to a quasi-static process. However, in reality, the backfilling processes is more than just a series of snapshots. It is an evolving process in which the pipeline has a history. The pipeline can already be partly buried or floating. The beam-model cannot be used as a tool to predict pipeline floatation, if the history of the pipeline is not included.
6 Conclusions and recommendations

In this thesis pipeline floatation during dredge-based backfilling is subject of discussion. A subsea pipeline should remain stable in all circumstances. This includes the backfilling operation of a subsea pipeline. It is not always clear if the in situ soil is suitable to be used as backfill. In the past this lack of knowledge sometimes resulted in pipeline floatation during backfilling. Also material was disposed, which would have been perfectly suitable as backfill. In this case material is supplied from elsewhere, which is an economic disadvantage. These two risks indicate the importance of accurately predicting the displacement of a pipeline during and short after backfilling. The main objective of this study was stated as:

How can pipeline floatation during dredge-based backfilling be prevented?

The research questions that have been investigated in this thesis are presented below (see also Section 1.5).

- What is the sedimentation length of a particular soil-water mixture? To achieve this, it is necessary to know the density of the slurry, the gradation of the soil, the initial mixture speed and trailing speed of the Trailing Suction Hopper Dredger (TSHD).
- How does the density of a soil-water mixture develop in time?
- How does the strength of the deposited material develop? In case of coarse sand or gravel the strength development will be instantaneous. Strength development of a clayey soil-water mixture may take a few days.
- The properties of the pipeline in combination with the three points above will provide an estimation whether the pipeline will remain stable during backfilling.

The conclusions of this study are presented in Section 6.1. These conclusions are evaluated using the previously stated research questions. Section 6.2 concludes with a list of recommendations. This section explains what is still missing and what topics need further research.
6.1 Conclusions

This section elaborates on the various research questions. These questions have been presented in the previous section. The results can be used to draw conclusions about pipeline floatation during dredge-based backfilling.

First, the conclusions on the sub questions are presented. Secondly, the conclusions on the two dimensional sedimentation (2DV) model and the beam-model are presented. Finally, the conclusions regarding the main objective are drawn.

What is the sedimentation length of a particular soil-water mixture?

The sedimentation length can be defined as the distance covered by a soil-water mixture flow before 90% of the sand from this mixture is settled (Mastbergen, 1988). In this thesis is referred to this definition as the 90% sedimentation length. This definition is difficult to use when you consider a moving Trailing Suction Hopper Dredger (TSHD) with continues discharge. In that case it is impossible to know at what moment 90% of the soil-water mixture has settled.

Another definition of the sedimentation length is the Total Sedimentation Length (TSL). This definition is illustrated in Figure 52. The TSL of various mixtures can easier be compared and relevant parameters can be obtained. These parameters, in addition to the TSL, are the maximum sedimentation height and the density of the soil-water mixture.

The TSL and the 90% sedimentation length of a particular soil-water mixture depend on a number of factors. It is a function of the particle size, the particle shape, the initial mixture concentration and the initial mixture velocity. The 90% sedimentation length can be calculated using standard formulas (Section 2.2). The TSL can be analysed using the 2DV model.

The 90% sedimentation length of soil-water mixture is a measure to determine the sedimentation length more easily. However, when the sedimentation length of soil-water mixture is determined using the 2DV model, this length is not useful. In that case, the TSL together with the build-up height generates the most useful information.

Figure 52. Visualisation of the backfilling process. The Total Sedimentation Length (TSL) and the build-up height of the soil body are specified.
The 2DV model can be used to make a static analysis (Section 4.3). This will provide information on the TSL of a certain soil-water mixture when the inflow point remains fixed at one place. It is also possible to use the 2DV model to simulate the backfilling process of a sailing TSHD. This simulation is made using a moving inflow point. There are two approaches to achieve this within the model environment. The first one is the ‘multiple fixed inflow points’ approach (Section 4.4.2). In this approach a number of inflow points are defined next to each other. Material is discharged through these inflow points. These inflow points are used consecutively. The other approach is the ‘moving boundary’ method (Section 4.4.3). This method is more user-friendly and more elegant, because substantially less input is required. This method uses only one inflow point. This inflow point moves from a starting point to an end point with a certain speed. Appendix B provides more information about moving inflow points. However, at the time of writing this thesis, the ‘moving boundary’ method does not function properly. This method should therefore be improved.

**How does the density of a soil-water mixture develop in time?**

The density development of a soil-water mixture depends mostly on the grain size distribution of the considered soil. Soils with a relatively large fraction of fine material will settle slowly compared to soils with a smaller fraction of fine material. This is confirmed with one dimensional sedimentation tests using the 2DV model in Section 4.2. The static analysis (Section 4.3) suggests that soil-water mixtures with larger particles and lower concentrations settle out faster. Also the mixing layer will be smaller. This implies that the transition from suspension to soil will be faster or sometimes even instantaneous.

**How does the strength of the deposited material develop?**

Been & Sills (1981) suggested that the basic difference between a suspension (sedimentation phase) and a solid skeleton consisting of the same particles is the presence of effective stresses. This implies that during the sedimentation phase, the measured pore water pressures are equal to the total vertical stress. The maximum density at which a soil-water mixture can exist as a suspension without the presence of effective stresses, is termed the structural density. At higher density values, there is a soil structure that supports some or all of the sediment weight through non-zero effective stresses. There is a corresponding structural void ratio, at which the effective stress is zero. The fluid pressure is equal to the total load, because it behaves as a fluid. This is no longer true when an extended structure is being formed, because this structure can carry at least part of its own weight. As a result the pore water pressure will be lower than the total weight of the mixture.

Experimental and theoretical studies of settling processes by Pane (1985) and Toorman (1996) have been restricted to the solution of vertical deposition problems. Nevertheless, the study of backfilled trenches should not be limited to the modelling of sedimentation and the formation of soil only. Horizontal material transportation
through the trench should also be included. Transportation of sediments, sedimentation and strength development are simultaneous parts of the settling process.

The properties of the pipeline in combination with the three points above will give an estimation whether the pipeline will remain stable during backfilling.

Data from the different reference projects are used to simulate the backfilling operation in the 2DV model. These simulations produce several plots of the sedimentation process. These plots can be used to simulate the TSL and the density of the soil-water mixture. The results of the 2DV model are used as input for the beam-model. The beam-model assumes that pipeline displacement caused by liquefied soil can be modelled using the beam on elastic foundation theory.

This model presents an approach to predict pipeline displacement. To make a convincing assessment of the pipeline behaviour during backfilling, a number of parameters need to be determined. Firstly, it is necessary to obtain the grain size distribution of the in situ soil. This grain size distribution, together with the initial mixture discharge velocity and the sailing velocity of the TSHD, provides information of the TSL, the mixture height and the mixture density. The dimension of the pipeline and the bedding constant $k_v$, together with the flow field obtained from the 2DV model, are used as input for the beam-model. This model estimates the displacement of the pipeline at a particular moment. When the predicted displacement is too large the local material should not be used as backfill and material should be supplied from elsewhere.

The 2DV model is tested to its limits in this thesis. The simulation of the backfilling operation of a moving hopper requires a model area with a great length and a relatively small height. This results in a model composed of stretched grid cells (in longitudinal direction). The 2DV model is at his numerical boundaries due to these stretched grid cells. The results of the ‘moving hopper’ simulations are therefore questionable. The results of the static simulations can be considered as reliable, since in these simulations the 2DV model remains well within the boundaries.

2DV model

The 2DV model is developed to simulate the sedimentation of sand or silt. These materials develop strength directly after sedimentation. The 2DV model cannot be used to provide a proper simulation, if a material exhibits consolidation behaviour.

The 2DV model is well suited to simulate sedimentation in relatively small spaces. The results of the 2DV model may become inaccurate when the model space becomes larger. For instance, when a relatively long model area is used to model the backfilling operation of an underwater trench. In that case, the model has a large length and a small height. The grid cells are stretched in horizontal direction. These stretched grid cells may cause numerical instabilities. The sediment and momentum transport in horizontal direction is in this case questionably. The results of the sailing TSHD analysis consists of relatively high solids concentrations. It is doubtful that these high concentrations are observed in reality. The pick-up functions applied in the 2DV model have difficulties processing high concentrated soil-water mixtures.
The results of the sailing TSHD are unreliable due to the previously mentioned reasons. The application of the 2DV model for large model areas should therefore be validated using experimental data.

**Simulation of one dimensional sedimentation column**

The sedimentation time of different soil-water mixtures is compared in the one dimensional sedimentation tests. In these tests all parameters are kept constant, except for the grain size distribution. Seven grain size distributions are analysed in this study. Four of these distributions are acquired from case studies and three are artificial monosized distributions.

The sedimentation time of the different soil-water mixtures are compared with the D50 values of these mixtures. The derived relation is shown in Figure 19. Using this relation it is possible to estimate the sedimentation time of a soil-water mixture. This empirical formula is only valid for a mixture with an initial mixture density of 20% solids. The relatively coarse soil-water mixtures (Markermeer, North sea, 500 µm, and 250 µm) have short sedimentation time. According to these simulations, these tests are therefore suitable to be used as backfill material. The particles in the finer soil-water mixtures (Varandey, Kuwait, and 125 µm) need a relatively long time to settle out of suspension. These soils are therefore not appropriate or questionable to be used as backfill material. Sailing velocity, initial mixture velocity, mixture density and the trench profile are not taken into account in these simulations and are therefore not a part of this estimation.

The 90% sedimentation time of soil-water mixture depends, according to these simulations, on the D50 value. The influence of the shape of the grain size distribution is very small. However, in this study only grain size distributions are analysed, with a fines content of less than 35% percent. Mixtures with a larger percentage of fines may behave differently.

**Analysed case studies:**
- Markermeer, D50: 324 µm
- North sea, D50: 218 µm
- Varandey, D50: 96 µm
- Kuwait, D50: 23 µm
- Artificial monosized grain size distributions, D50: 500 µm, 250 µm, 125 µm

**Simulation of backfilling process using static TSHD**

The results of the simulations of the backfilling process of a static TSHD compare the build-up height and sedimentation length of different soil-water mixtures. In these tests all parameters are kept constant, except for the grain size distribution and the mixture density.

The results of the analysis in which the grain size distribution is varied (Section 4.3.2), reveals that a mixture with a larger D50 value settles out faster compared to a soil-water mixture with a smaller D50. This results in a larger build-up height and sedimentation length for the mixture with the large D50 value.
The comparison of four soil-water mixtures with different initial density (10%, 20%, 30% and 40% soil content), shows that soil-water mixtures with a larger soil content have a larger sedimentation length. However, the build-up height is indifferent of the soil content and depends on the D50 value (Figure 29).

Based on these simulations, it is not possible to determine if a pipeline will float or not during the backfilling operation. However, the sedimentation length and build-up height reveal the behaviour of a soil-water mixture. A long sedimentation length and slow progress of the build-up height indicate that a pipeline is susceptible for pipeline floatation during installation. According to these simulations, the Kuwait, Varandey, and 125 μm mixtures have a potential risk of pipeline floatation during backfilling. The Markermeer, North sea, 500 μm, and 250 μm soil-water mixtures have a low risk of pipeline floatation during backfilling, according to these simulations.

Analysed case studies:
Markermeer, D50: 324 μm
North sea, D50: 218 μm
Varandey, D50: 96 μm
Kuwait, D50: 23 μm
Artificial monosized grain size distributions, D50: 500 μm, 250 μm, 125 μm

Simulation of backfilling process using sailing TSHD

The results of the simulations of the backfilling process of a sailing TSHD can be divided in three different categories. The plots of these simulations can be found in Appendix F. The coarse soils (Markermeer and 500 μm) settle out of suspension fast. These soils are therefore appropriate to use as backfill material. The relatively fine soils (125 μm and Varandey) remain in suspension for a relatively long time and are therefore not useful as backfill material. The third category is a soil with a distinct coarse and a distinct fine part (North Sea). The fine particles remain in suspension for a relatively long period, however the coarse particles settle out almost immediately. A soil with these characteristics can therefore be an appropriate backfill material, when the coarse particles settle out of suspension fast and will add weight to the pipeline. These coarse particles will protect the pipeline from floatation, while the finer particles will settle out over time.

The results of the sailing TSHD analysis consists of relatively high solids concentrations. It is doubtful that these high concentrations are observed in reality. It is therefore possible that the 2DV model will predict pipeline floatation, while this will not happen in reality.

Analysed case studies:
Markermeer, D50: 324 μm
North sea, D50: 218 μm
Varandey, D50: 96 μm
Artificial monosized grain size distributions, D50: 500 μm, 125 μm
**Beam-model**

The beam-model is used to model pipeline displacement. In the model, a pipeline will tend to float, if it is lighter than the surrounding soil-water mixture (Section 5.3). However, the pipeline will only float if this soil-water mixture extends over a certain length (Section 5.4). This length depends on the pipeline stiffness and on the weight of the soil-water mixture in relation to the weight of the pipeline (Section 5.4).

The beam-model overestimates the stiffness of the pipeline. Underwater pipelines are usually covered with a concrete casing. This casing is not evaluated in this model. However, the specific gravity of the pipeline is corrected for this concrete casing. This leads to relatively large pipeline thickness. The simulated pipeline will therefore not float as easily as in reality. The used pipeline thickness is roughly two times higher as the used thickness in the industry. This results in a moment of inertia which is roughly two times higher as in reality. This means that the pipeline will act two times stiffer as in reality.

The beam-model does not consist of the possibility to fixate the pipeline locally. This possibility should be implemented to simulate already sedimented soil on the top of the pipeline. The absence of this possibility results in a too conservative model. Since, this already settled material adds weight to the pipeline and will reduce the area of the pipeline surrounded by the soil-water mixture.

Strength development of the soil-water mixture is not included in the 2DV model. This causes the highest buoyancy when the soil-water mixture concentration is highest. However, the soil-water mixture already develops strength when this soil-water concentration is reached.

The beam-model is too conservative and does not consist of all relevant processes. The pipeline stiffness is overestimated and the concrete coating is not evaluated. Strength development of the soil-water mixture is not included in the model, and already sedimented soil cannot be used to add weight to the pipeline. These options should be included to create a more useful model.
How can pipeline floatation during dredge-based backfilling be prevented?

This thesis elaborates on the prevention of pipeline floatation during dredge-based backfilling. The 2DV sedimentation model is used to simulate the backfilling of an underwater trench with a pipeline placed in it. This model generates a density distribution, which is used as input for the beam-model. The beam-model calculates the pipeline displacement due to the backfilling operation. These models simulate the backfilling process and can be used to estimate if a certain pipeline is susceptible for floatation during backfilling. However, these models do not provide a direct solution to the main objective of this thesis: How can pipeline floatation during dredge-based backfilling be prevented.

When pipeline floatation during dredge-based backfilling is expected, generally three methods can be used to prevent this:

1. Adding weight to the pipeline.
2. Locally fixating the pipeline using gravel, sand, or rock.
3. Altering the backfilling process: different sailing speed TSDH, different initial mixture velocity, different mixture density or backfilling in layers.

These methods are not evaluated in this thesis. The 2DV model should be validated for large model spaces and the beam-model should be improved, before these methods are evaluated.

In future research, first the 2DV model should be validated for large model spaces. Secondly, the beam-model should be improved. When both of the models are considered suitable, they can be used to analyse the different methods of pipeline floatation prevention.
6.2 Recommendations

This research about pipeline floatation during dredge-based backfilling closes with a list of recommendations. Most of these recommendations follow directly from the conclusions and require only a little further explanation. The recommendations are based on actual practical strategies and on the numerical model.

- The 2DV sedimentation model can be used to model the backfilling operation of using a sailing TSHD. In the case of sailing TSHD, the model space is relatively long and the grid cells are therefore stretched. The influence of these stretched grid cells on the results of the model simulations are unknown. The 2DV model should be validated for model spaces with relatively large lengths. This validation should be carried out using experimental data or data from an actual backfilling operation.

- The 2DV model applies the Van Rijn and Van Rhee pick-up functions. These pick-up functions have difficulties to model mixtures with high solids concentrations. The pick-up functions applied in the 2DV model should be validated and probably modified for high concentrated soil-water mixtures.

- The 2DV sedimentation model is used to simulate the backfilling process of a pipeline placed in a subsea trench. As the name suggests, only two dimensions (horizontal and vertical) are taken into account in the 2DV model. This is explained in Figure 30. When a moving inflow point is used in a two dimensional model, this may lead to effects that are not seen in reality. It is not possible for material to move past the inflow jet. This is caused by the two dimensional nature of the model. In reality this inflow jet is three dimensional. Al sorts of complex mixture flows close to the inflow jet, are possible. This aspect of the simulation should be experimentally investigated.

- The beam-model is too conservative and not all aspects are included. The pipeline should be divided in clear components: steel pipeline base, concrete coating, etc. The strength development of the soil-water mixture should be included in the model. Already sedimented soil should be applied to fixate and add weight to the pipeline.

- To gain some information on the sedimentation process and to predict the probability of pipeline floatation, some simple sedimentation tests could be performed. To perform these tests, the same approach can be followed as showed in Section 4.2. If the actual in situ material is available, it is also possible to carry out a sedimentation test using a sedimentation column. This can give an indication of the sedimentation velocity. These tests are a simplification of reality. Mixture flows and other effects are not taken into account. However, it could give an indication of the sedimentation time.
• The 2DV model is not suitable to model soils which contain a large fraction of cohesive material. Also consolidation is not included in the model. To analyse this type of material, it is recommended to use another model. A model capable of modelling cohesive sediment and consolidation is the 1DV point model (Winterwerp, 2004). This model was developed on the basis of Delft3D-Flow. This model includes a momentum module, a sediment transport module and a turbulence module. These modules are comparable with the modules in the 2DV model. However, the 1DV point model is a one dimensional model. Furthermore, the 1DV point model consists of a flocculation model, a consolidation model, and a module which models surface waves.

In spite of the significant number of recommendations, it is believed that the results in this thesis have improved the insights into the behaviour of pipeline floatation during dredge-based backfilling. This is needed to increase the insights whether a soil can or cannot be used as backfill. Hopefully, this thesis has reduced the gap a bit that is often observed between the demanded soil characteristics and the in situ soil.
Bibliography


Appendix A: 2DV model discharge pipe

To correctly model the backfilling process it may be necessary to include the discharge pipe in the model. To verify if it is required to model the discharge pipe a comparison is made between a model with and a model without the discharge pipe in it. Placing the discharge pipe in the 2DV model has one major disadvantage. In that case it is not possible to use a moving inflow point. A moving inflow point is necessary to realistic model the backfilling process using a TSHD.

The next series of plots show the two different approaches. The plots at the left side show the contour of the concentration when the mixture inflow point is placed at the top of the model. The model used to make the plots on the right is identical to the first one. Except that an extra four meters is added on the top of the model. This four meters in used to place the discharge pipe in the model. The width of the discharge pipe is the same as the width of the inflow point: one meter. The discharge pipe is modelled by placing two obstacles next to inflow point. The obstacles are four meters long.

The plots at the right are stretched to make it possible to compare the plots with and the plots without the discharge pipe. The parameters used are in both models the same. The plots are each compared after the same moment in time.

Used parameters:
- Initial volume concentration 20%
- Mixture in flow velocity 4 m/s
- Sand bed porosity 0.4
- Grain density 2650 kg/m$^3$
- Water density 1025 kg/m$^3$
- Grain size 500 μm

![Model without discharge pipe](image1)

![Model with discharge pipe](image2)
Conclusion

The input for both models is exactly the same. It is tried to give both model representations the same grid. However, the models do not have the same height. That is why there can be slight differences in the grid in vertical direction.

The flow of the sand-water mixture is in both models roughly the same. There are small differences, but these are negligible. The shapes of the two formed sand hills are the same. Also the height and length of these hills are almost the same. The results of both model representations show small differences, but are overall comparable. The two representations are so much alike that they can be interchanged without losing valuable information. Excluding the discharge pipe from the model makes it possible to model a moving TSHD.
Appendix B: 2DV model, moving inflow point

Starting point for both ‘the multiple inflow points’ and the ‘moving boundary’ method is that the behaviour of the soil-water mixture flow discharge from a TSHD is modelled using the 2DV model. The 2DV model has the disadvantage that normally the inflow points only can be described as static points. To use a moving inflow point in the model two approaches are used. These approaches are listed below.

Multiple fixed inflow points

The method of the multiple fixed inflow points defines a maximum of fifty inflow points next to each other. Defining more inflow points leads to instability of the model.

When one wants to model the sedimentation of a substantial length of a trench, it is necessary to use a wider inflow point than one would use in practice. In this study a total inflow area of five hundred meters is simulated. This leads to fifty inflow points of ten meters wide each. Some values need to be converted to achieve this. The inflow rate per second should remain constant compared to an inflow point of one meter. This leads to a discharge velocity of 0.4 meters per second instead of 4 meters per second. A difficulty for this approach is that for each inflow point values need to be assigned individually. This is tedious work and it might lead to errors if one does not stay focussed.

An important aspect is the trailing speed of the TSHD. The trailing speed is simulated with the one by one use of the fifty inflow points. The total sediment influx should remain constant during the simulation. The sediment influx is a combination of the sediment concentration and the inflow velocity. The concentration is assumed to have a constant value of 20%. The summed inflow velocity is at all times 0.4 meters per second. It should be stated that for the first eight seconds this requirement is not applicable, because in this stage the model is starting up. An inflow point is not used when the inflow velocity is set at zero m/s. The used inflow velocities are showed in table 1.

The grid cells in these simulations have a horizontal width of one meter. This is about a factor five larger compared to the size of grid cells in vertical direction. These stretched grid cells increase the change of numerical instability of the model. Perfectly square grid cells would lead to a calculation time that would be at least five times longer. This would very undesirable, especially when one takes the current calculation time into account.
Table 1. The first fifty seconds of the inflow velocity input txt. file.

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<td>0</td>
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<tr>
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<td>0</td>
<td>0</td>
<td>0.32</td>
<td>0.08</td>
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<td>0.08</td>
<td>0.32</td>
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<tr>
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<td>0</td>
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<td>0</td>
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<tr>
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<td>50</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.32</td>
</tr>
</tbody>
</table>
Overflow level Left = 8.8

Rans Module

nr cells: nx = 560  nz = 26

initial time step = 0.001 [s]
end time = 3600 [s]
i_bodem= 0
kappa = 0.41
Free water surface = False
Wall on Top = False
Fluxlimiter used = False

k-eps module

Buoyancy terms ON
Coefficients used in k-eps model
c_mu = 0.09
c1eps = 1.44
c2eps = 1.92
c3eps = 0
sigmaT = 0.7
sigmak = 1
sigmaeps = 1.3

Sediment Transport

n0 = 0.4

number of fractions: 7
diam  [-]
3.55E-5  0.015
9.4E-5  0.021
0.0001525  0.187
0.000215  0.435
0.000375  0.342

ks : 0.003

Fluxlimiter used = True
Fluxlimiter type = Superbee
Influence flow velocity on sedimentation: Pickup Function Van Rhee
Theta 0 = 4

Fall velocity calc method = 1
Moving boundary

The setup of the moving boundary model is basically the same as the setup of the multiple inflow points method. The main difference is that in this approach only one inflow point is used. This inflow point has the same width as the inflow point in the previous method. The same grid as in the multiple inflow points method is used. Therefore it is not possible to use an inflow of one meter wide (an inflow point should always be larger than one grid cell). Also the inflow velocity of 0.4 meters is maintained. In this way both methods are comparable, because the model set up is comparable.

The moving boundary method has one main difference compared to the multiple inflow points method. This is the moving inflow point. The moving inflow point moves at trailing speed over the top of the model. It is believed that this approach provides a more smooth moving inflow point compared to the more uneven method of the multiple inflow points. The moving inflow point is commanded by the moving boundary file. This file contains a start point, an inlet surface and an inlet velocity.

Input 2DV Model
---------------

General Info

Input File : D:\NEW\TESTvar27update.xml

length = 1700
height = 4.5

rhow = 1025
rhos = 2650

Overflow level Right Fixed
Overflow level Right = 8.8
Overflow level Left Fixed
Overflow level Left = 8.8

Rans Module

nr cells: nx = 580 nz = 26

initial time step = 0.001 [s]
end time = 3600 [s]
i_bodem= 0
kappa = 0.41
Free water surface = False
Wall on Top = False
Fluxlimiter used = False
k-eps module

Buoyancy terms OFF
Coefficients used in k-eps model
\[ \begin{align*}
    c_\mu &= 0.09 \\
    c_{1\epsilon} &= 1.44 \\
    c_{2\epsilon} &= 1.92 \\
    c_{3\epsilon} &= 0 \\
    \sigma_T &= 0.7 \\
    \sigma_k &= 1 \\
    \sigma_{\epsilon\epsilon} &= 1.3
\end{align*} \]

Sediment Transport

\[ n_0 = 0.4 \]

number of fractions: 7
diam \([-]\)
\[ \begin{align*}
    4.65E-5 &\quad 0.078 \\
    7.65E-5 &\quad 0.242 \\
    0.0001075 &\quad 0.156 \\
    0.0001525 &\quad 0.177 \\
    0.000215 &\quad 0.202 \\
    0.0003025 &\quad 0.105 \\
    0.0011775 &\quad 0.04
\end{align*} \]

\[ k_s = 0.003 \]

Fluxlimiter used = True
Fluxlimiter type = Superbee
Influence flow velocity on sedimentation: Pickup Function Van Rhee
Theta 0 = 4

Fall velocity calc method = 1
Appendix C: Beam-model, Matlab

clc;
close all;
clear all;

%% Constant parameters
L = 2000; % Length [m]
alpha = 90; % Stinger angle [-]

n = 200; % Number of elements [-]
k = 35000; % Spring coefficient of soil [N/m]
E = 210000*10^6; % Elasticity modulus of steel [N/m^2]
D = 1/0.0254; % Outer diameter [inch]
t = 50; % Wall thickness [mm]
g = 9.81; % Gravitational acceleration [m/s^2]
T = 0; % Tension [tonnes]
Ds = 76930; % density steel N/m^3

At = pi*((D*0.0254)^2)/4; %surface total
As = pi*((D*0.0254)^2-((D*0.0254)-2*t/1000)^2)/4; %surface steel

q1 = (Ds*At-1025*g*At)*(1/(0.25*pi)); %1.25 Weight of pipeline [N/m]

I = pi/4*(10^-3*25.4*D/2)^4-... pi/4*(10^-3*25.4*D/2-t*10^-3)^4; % Moment of inertia [m^4]
a_sting = alpha*pi/180; % Rewrites the stinger angle to radians
T_max = T*10^3*g; % Rewrites the maximum tension to Newtons
F_hor = T*cos(alpha)*1000*g; % Horizontal component of the tension [N]

niter=100000; % Number of iterations

%% Boundary Conditions
x = linspace(0,L,n);
w = x*0;
dx = x(2)-x(1);

w(1) = q1/k; % Deflection on i = 1
w(2) = w(1); % Deflection on i = 2
w(n) = q1/k; % Deflection on i = n
w(n-1) = w(n); % Deflection on i = n-1
w(n-2) = w(n); % Deflection on i = n-2

%Buoyancy
Buoy=x*0;
%100 sec
%             a=68;
%             b=80;
%             c=90;
%             d=92;
%200 sec
a=80;
b=99;
c=100;
d=130;
%300 sec
%             a=39;
%             b=98;
%             c=109;
%             d=122;
%400 sec
%             a=25;
%             b=100;
%             c=120;
%             d=137;

conc_ab=0.30;
conc_bc=0.30;
conc_cd=0.30;

h1=6.0;
h2=6.0;

Buoy(a:b)=(conc_ab*2650+(1-conc_ab)*1025-1025)*g*At;
Buoy(b+1:c)=(conc_bc*2650+(1-conc_bc)*1025-1025)*g*At;
Buoy(c+1:d)=(conc_cd*2650+(1-conc_cd)*1025-1025)*g*At;

%% catenary combined with beam:
date
tic
for j=1:niter;
    for i=3:n-2,
        w(i) = (q1-Buoy(i)*(w(i)>-6)-k*w(i)*(w(i)>0)-E*I/dx^4*(w(i-2)-4*w(i-1)-4*w(i+1)+w(i+2))...)/(6*E*I/dx^4);
        if w(i)<-h1 && w(i)>-h2;
            Buoy(i)=(conc_bc*2650+(1-conc_bc)*1025-1025)*g;
        elseif w(i)<-h2;
            Buoy(i)=0;
        elseif w(i) > -h1
            if i>=0 && i<=a-1;
                Buoy(i)=0;
            elseif i==a && i<=b
                Buoy(i)=(conc_ab*2650+(1-conc_ab)*1025-1025)*g;
            elseif i==b+1 && i<=c
                Buoy(i)=(conc_bc*2650+(1-conc_bc)*1025-1025)*g;
            elseif i==c+1 && i<=d
                Buoy(i)=(conc_cd*2650+(1-conc_cd)*1025-1025)*g;
            else
                Buoy(i)=(conc_cd*2650+(1-conc_cd)*1025-1025)*g;
        end
    end
end
elseif i>=d+1 && i<=n
    Buoy(i)=0;
end
end;
end;
end;
toc

%% Plot options
plot(x,-w,'black','LineWidth',5);
set(gca,'fontsize',14);
xlabel('horizontal distance along seabed [m]', 'Fontsize', 14);
ylabel('height above seafloor [m]', 'Fontsize', 14);
title('Pipeline displacement, North Sea Fixed inflow points, t = 0 s', 'Fontsize', 14);
Appendix D: Results static tests

Results:
Grain size: 500 µm, concentration 0.1

Grain size: 500 µm, concentration 0.2
Grain size: 500 μm, concentration 0.3

500 μm, conc. 0.4
Grain size: 250 μm, concentration 0.1

Grain size: 250 μm, concentration 0.2
Grain size: 250 μm, concentration 0.3

Grain size: 250 μm, concentration 0.4
Grain size: 125 μm, concentration 0.1

Grain size: 125 μm, concentration 0.2

Grain size 125 μm, concentration 0.3
Appendix E: Results 1D tests

500 μm

250 μm

125 μm
Kuwait

After 3600 seconds the sandbed reached a height of 0.78 meter. Only forty-seven percent of the sediment is settled out.

Varandey

After 2000 seconds almost all material is settled out. Already after 1400 seconds 90 percent of the sediment is settled out. At this time a lot of fine material in low concentration is still in suspension.

North Sea

After 480 seconds all material except some fine particles is settled out. After 360 seconds already 90 percent of the material was settled out of suspension.
After 300 seconds all material except some fine particles is settled out. After 230 seconds already 90 percent of the material was settled out of suspension.
Appendix F: Results moving inflow point

Results ‘multiple inflow points’

Varandey
North Sea sand
Markermeer sand
D = 125 μm

C.C.P. Biemans

Boskalis – TU/Delft

111
D = 500 μm
Results ‘moving boundary’

Varandey
North Sea sand
Markermeer sand
Appendix G: Results beam-model