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Migration of Gravel Barriers over a Consolidating Substrate: Implications for Coastal Management.

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Declaration

This thesis was submitted for examination in December, 2016. It does not necessarily represent the final form of the thesis as deposited in the University after examination.

I, Lauren Burt declare that this thesis and the work presented in it are my own and has been generated by me as the result of my own original research.

I confirm that:

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2 Where any part of this thesis has previously been submitted for any other qualification at this University or any other institution, this has been clearly stated;

3 Where I have consulted the published work of others, this is always clearly attributed;

4 Where I have quoted the work of others, the source is always given. With the exception of such quotations, this thesis is entirely my own work;

5 I have acknowledged all main sources of help;

6 Where this thesis is based on work done by myself jointly with others, I have made clear exactly what was done by others and what I have contributed myself;

7 None of this work has been published before submission.

Abstract

Gravel barrier beaches are important geomorphological features that provide a buffer to low-lying coastal areas around the world from wave attack. This sheltering effect also promotes the formation of extensive intertidal habitat and lagoons composed of poorly consolidated sediment substrates of varied stratigraphy in the lee area with silt, clay and peat layers. Gravel barriers naturally respond to increases in sea level and wave overtopping of beach material by migrating landwards over the poorly consolidated substrate in their lee. This application of load causes the substrate to consolidate, causing the barrier crest to reduce over time. The magnitude of consolidation and consequent crest lowering will make the barrier vulnerable to further sea level rise and wave overtopping, resulting in acceleration of landward migration and increases in flood risk. Very few global or local studies exist to explore this problem and there is a lack of qualitative data, despite the implications for coastal management. This thesis aims to address this knowledge gap, utilising Hurst Spit as an interesting and important case study of a natural gravel barrier system, maintained in response to storm events in order to preserve its major flood defence purpose. The gravel barrier has migrated landward by 100 metres over the last 60 years in response to a reduction in sediment supply and storm damage. The next phase of sediment recharge is due within the next 5 years, with material due to be placed on the back slope, to attain a wide crest that meets design requirements. This realignment of the back slope will extend onto the poorly consolidated material, causing it to consolidate. Sediment coring confirmed that the substrate material was predominately marine muds, with high water content, but low permeability. The greater the thickness of the poorly consolidated material, the higher the magnitude of consolidation, and the thickness of the substrate was found to be less than the height of the beach overburden. The magnitude of consolidation at Hurst Spit is varied, making some areas more vulnerable to crest lowering. Vulnerability to consolidation was caused by increased substrate thickness, presence of peat and beach aspect in relation to incoming predominate storm waves which cause enhanced overtopping and landward migration of the barrier. The results highlight the need for further understanding of the consolidation of barrier beaches, especially when conducting beach maintenance which essentially realigns barrier beaches over poorly consolidated materials. The process of consolidation coupled with future sea level rise is of great interest for those involved in the management of gravel barriers in the future, especially where large areas of low-lying land and assets are protected.

Preface

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Special thanks goes to Rob Ainsworth at Soils Ltd., for the generous provision of a drill rig and talented crew for a day earlier this year. Such support for academic work has been priceless in enabling me to meet the aims of a project, which could have been unachievable due to cost.

My wonderful Mother, deserves an honour for her ability to keep me focused and drive me forward for the entirety of this masters and beyond.

To Ryan. Thank you for donning waders and helping me in my quest to find peat. The endless weekends and evenings that I have spent finishing this masters are almost complete. You're a star for standing by me throughout.

Dedication

Ultimately, this thesis is dedicated in memory Professor Andrew Bradbury. A truly brilliant and talented man who guided and inspired me at the very beginning, for he is the reason that I am writing this today.

I was honoured and extremely proud to win the 2016 Bradbury Bursary set up by SCOPAC. This was awarded upon review of the scoping study prepared for this thesis, and supported both fieldwork and data analysis.

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Abbreviations

ACF	Automatic Consolidation Frame
BP	Before Present
CCO	Channel Coastal Observatory
CSA	Cross-sectional Area
GIS	Geographical Information Systems
GPS	Global Positioning System
MHWS	Mean High Water Springs
MLWS	Mean Low Water Springs
MMO	Marine Monitoring Organisation
MSL	Mean Sea Level
NE	Natural England
NFDC	New Forest District Council
NNR	National Nature Reserve
OD	Ordinance Datum
RTK	Real Time Kinematic
SAC	Special Area of Conservation
SPA	Special Protected Area
SSSI	Site of Special Scientific Interest
UK	United Kingdom

Symbols

A	Area
D	Diameter (mm)
E'_0	One-dimensional stiffness modulus
G_s	Relative density (ρs/ρw) of soil grains (grain specific gravity)
kPa	Kilopascal
N	Newton
C_v	Consolidation coefficient for vertical compression due to vertical flow
d	Maximum drainage path length (half-height of oedometer test specimen h_0)
e	Void ratio
g	Acceleration due to Earth's gravity (9.81m/s^2)
h_0	Initial specimen height (mm)
k	Soil permeability
t	Time
t_x	Reference point on time axis
v	Specific volume
w	Water content
z	Depth coordinate
γ	Unit weight (ρg)
ε'_v	Vertical strain
Ln	Natural logarithm
ρ	Mass density (subscript s is of soil, subscript w is of water)
ρ	Settlement
σ'_v	Vertical effective stress
ϕ	Phi
µm	Micrometre
0	Subscript to denote initial state

1. Introduction

1.1 Motivation of Study

In the UK, gravel barriers provide widespread natural protection to the coastline and are of major ecological and environmental importance. Barriers tend to migrate landwards due to overtopping storm waves, which push sediment over the crest to the back slope. This tendency has been exacerbated due to increases in relative sea level over the 20th and early 21st Century, further enhanced by anthropogenic climate change in the future. Barriers underlain by poorly consolidated sediments are especially vulnerable as these sediments consolidate under the load applied by a migrating barrier, causing the barrier crest to lower and making the barrier vulnerable to further overwash and overtopping. Barrier formations provide a wealth of benefits as they reduce coastal flood risk and shelter low-lying land in their lee. Coastal managers looking to maintain this flood protection need to understand the magnitude of consolidation to predict barrier dynamics and ensure that the design level is maintained. Managed realignment of gravel barriers may become a preferred shoreline management policy for coastal managers in the future, as the 'hold the line' may be increasingly unsustainable in light of sea level rise. This may involve material placed on the lee slope of existing barriers as an artificial roll back, loading previously unconsolidated material. The magnitude of consolidation in this context and implications for coastal management are not widely understood, and this is due to the lack of understanding of the stratigraphic and geotechnical properties of the substrate beneath migrating gravel barriers that make it vulnerable to consolidation processes. This thesis addresses this problem by providing an analysis of consolidation behind a significant barrier beach at Hurst Spit, Hampshire, UK. Previous collection and analysis of such data is very limited, both in the UK and worldwide.

1.2 Aims and Objectives

The primary aim of this thesis is to investigate the stratigraphic and geotechnical properties of the back barrier sediments at Hurst Spit, an important example of a migrating gravel barrier.

The following objectives were considered:

Objective 1	Conduct representative sediment sampling of the back barrier sediments at Hurst Spit, using coring equipment.
Objective 2	Establish the physical and geotechnical properties of the sediment.
Objective 3	Explore the implications these results for the management of Hurst Spit, including a proposed replenishment.
Objective 4	Discuss the wider implications of these results for coastal management of barrier beaches.

1.3 Structure of Thesis

The structure of this thesis aims to clearly present the steps taken in order to meet the aim of this thesis, to investigate the stratigraphic and geotechnical properties of the back barrier sediments at Hurst Spit.

Having introduced the study motivation, aims and objectives, a major section to draw together relevant literature is presented. This literature review explores the overarching concepts to enable a detailed understanding of the problem that this thesis aims to solve. The methodology section then demonstrates the methods used to solve the problem, and the results section then aims to present findings of the field and laboratory work. A discussion of the results follows in the next section, and will link the findings back to the aims and objectives of this thesis. In the final section, a concluding statement aims to draw together the findings and clarify important key findings and make recommendations for future research.

2. Literature Review

A review of available relevant literature was undertaken and is presented within this section. The first section looks to explore main feature of this study; gravel barrier beach systems. A detailed overview of barrier beach system configuration and stratigraphy, in addition to the forcing factors which influence the morphological evolution over a range of time scales is provided. Consolidation is then introduced as the main process of this study, firstly through an overview of consolidation theory and then applied to the coastal context. Methods for prediction of consolidation are provided. The next session then draws together a range of case studies of consolidation in the coastal environment, to set this study into context and discuss its importance. Finally, Hurst Spit, the study site used to investigate the stratigraphic and geotechnical properties of the substrate beneath migrating gravel barriers that make it vulnerable to consolidation processes is presented. A summary of the literature review draws together the main points.

2.1 Barrier Beach Systems

This section aims to clarify the form and configuration of barrier beaches, forcing factors and morphological evolution over a range of time scales.

2.1.1 Beach Nomenclature

In the first instance, beaches may be categorised into type dependent on sediment composition. This can be defined with use of a particle size distribution study, where the dominant sediment size or sizes (bimodal distribution) can be identified. Traditionally, the Udden-Wentworth scale (Wentworth, 1922) is used (Packham *et al.,* 2001) (Appendix A). This classifies gravel as having a mean diameter of 2 to 256mm (-1ϕ to -8ϕ), sand as 63µm to 2mm (4ϕ to -1ϕ) and mud <63µm (<4ϕ). Mud can be composed of silt and clay, with clay as any sediment less than 3.9µm in diameter. The composition of beach sediments can vary between locations and is a function of local sediment supply (Pye, 2001; Stripling *et al.*, 2008; Sutherland and Thomas, 2011).

With an understanding of the particle size distribution of the beach, it may be further categorised into fine (sand), coarse (gravel) and mixed (sand and gravel) grained beach types. The beach profile of each type varies due to the particle size. Coarse sediment is able to maintain steeper slope angles, and it is often found that gravel beaches have a steep, reflective shoreface (Nicholls, 1985; Pye, 2001; Anthony, 2008). The permeability is also relatively high, allowing for dissipation of incoming wave energy (Anthony, 2008). Beaches of fine sediment composition are not able to maintain such a steep slope, resulting in dissipative beaches of a gentler slope, and a lower permeability. It is worth mentioning that beaches are often a mixture of a range of coarse and fine particle sizes, *i.e*. a mixture of sand and gravel but may have a dominant sediment size (Nicholls, 1985; Pye, 2001; Dornbusch and Ferguson, 2016). Understanding of the detailed dynamics of wave interaction with coarse and mixed grained beaches is regarded to be narrower in scope than for fine-grained beaches (Pye, 2001; Jennings and Shulmeister, 2002; Neal *et al.,* 2002; Buscombe and Masselink, 2006; Anthony, 2008).

Coarse-grained beaches are often referred to as gravel or shingle beaches (Packham *et al.,* 2001; Van Rijn and Sutherland, 2011) with a sediment size of 2-64mm according to the Wentworth Scale. There is a tendency to use 'shingle' to describe rounded and sub-rounded gravel (Packham *et al.,* 2001; Pye, 2001; Nicholls and Webber, 1987) and therefore the terminology is interchangeable, with reference to both in the literature. Coarse-grained sediment will be referred to as 'gravel' for the purposes of this study as it includes both angular and rounded sediment above 2mm in diameter.

Beaches can be further categorised based on their configuration. Fringing beaches are wholly joined to the mainland at the landward side of the beach and remain in this location. Free-standing beaches may be partially attached to the mainland in the form of a spit, or tombolo, or detached in the form of a barrier beach (Anthony, 2008; Stripling *et al*., 2008). Beaches may also be placed into the 'swash aligned' or 'drift aligned' subcategory. 'Swash aligned' beaches are orientated perpendicular to the dominant wave direction, and are therefore subject to cross shore transport, whereas 'drift aligned' beaches are controlled by longshore sediment transport due to waves due to orientation at an angle to the dominant wave direction (Davidson-Arnott, 2010; Masselink and Russell, 2013).

The gravel barrier beach is of greatest interest to this study. The reflective, wave energy dissipating features of gravel in the form of barrier serves as an important natural coastal defence for areas of vulnerable low-lying land. Gravel barriers are considered as more resilient to change on a larger temporal and spatial scale than barriers composed of sand (Anthony, 2008). Barrier beaches are considered as narrow elongated features with a distinct crest, which separates seaward and landward beach slopes (Stripling *et al.*, 2008). Figure 1 shows a typical cross sectional profile of a gravel barrier beach, which distinct features such as a steep back slope, crest, and steep foreshore with berm features.

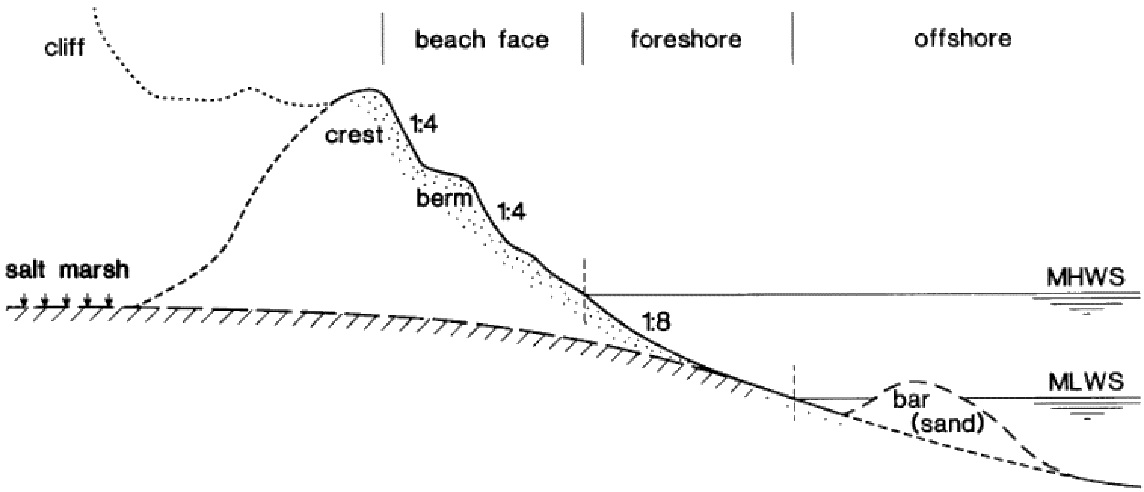

Figure 1: Typical cross sectional profile of a gravel barrier beach (Van Rijn and Sutherland, 2011).

Fine-grained barrier beaches are also common features around the world, however their form and morphological evolution vary to those of gravel composition. There is a growing consensus that gravel beaches are excellent facets to coastal managers, as features with vast geomorphological, ecological and engineering importance to the natural and human environment (Pye, 2001).

2.1.2 Morphological Evolution of Gravel Barrier Systems

Gravel barriers are dynamic features, influenced by a variety of different environmental forcing factors in the coastal zone. The benefits of gravel barriers are under threat due to changes in environmental forcing factors (Bradbury, 2000; Rosati *et al.*, 2010). Their evolution occurs over a range of temporal scales, from response to storm events to sea level rise over millennia (Rosati *et al.*, 2010).

One forcing factor that influences the morphological evolution of gravel barriers is the sediment supply (Bradbury, 2000; Davidson-Arnott, 2010). Gravel barriers can be found in many mid to high latitude coastlines around the world, the majority of which were previously impacted by Quaternary glaciation (Nicholls, 1985; Pye, 2001; Anthony, 2008; Van Rijn and Sutherland, 2011). The last glaciation yielded a source of glaciofluvial sediment with a full spectrum of sizes for beach, dune, barrier and estuarine growth, however it is a finite source of limited offshore supply (Masselink and Russell, 2013). Gravel beaches may also be found at lower latitudes adjacent to coral reef systems, eroding cliffs or adjacent to mouths of high energy rivers (Pye, 2001).

Wave action is thought to be the exclusive driver of sediment transport on gravel beaches, with tidal action relatively ineffective in sediment entrainment (Lewis, 1931, 1938; Pye, 2001; Van Rijn and Sutherland, 2011). Incoming waves push gravel up the beach to the run-up limit during the uprush phase, and during the backwash (weaker due to percolation through the gravel) the gravel moves down the beach due to gravity and wave retreat (Van Rijn and Sutherland, 2011). During prolonged periods of exceptional swell wave conditions, the morphological evolution of gravel barriers is accelerated resulting from overtopping and overwashing, especially where a site has experienced extreme wave heights and water levels during a storm event (Nicholls and Webber, 1989; Bradbury, 2000).

The beach is naturally driven landwards, as material is removed from the seaward front of the beach and transported over the crest to the landwards face, in a cycle of rollover (Masselink and Russell, 2013). During periods of energetic wave energy, this process is exacerbated, and the crest of the beach may translate many metres during an individual storm event. It is likely that barriers have naturally been migrating landwards over longer timescales during the Holocene transgression, a 10,000-year period of rising sea level after the last glaciation. Swash aligned gravel barriers respond to sea level rise through transgression or 'rolling back' (Orford *et al*., 1995; Dornbusch and Ferguson, 2016).

Increases in sea level over the last millennia have resulted in landward migration of coastal systems. Global mean sea level has risen at a rate of 1.7mm per year during the 20th century, increasing to 3.2mm per year in the period 1993 to 2003 (Horsburgh and Lowe, 2013). If a barrier is unable to migrate at the same rate as relative sea level rise into a suitable accommodation space, this leads to coastal squeeze (Orford and Pethick, 2006; Rosati *et al.*, 2010; Masselink and Russell, 2013). On the other hand, if the barrier is able to migrate over a gently rising, solid geology, then the barrier is able to maintain pace with sea level rise (Bradbury, 2000).

As the barrier elongates and accumulates, the barrier provides a sheltering effect from waves, storm surges and wind in its lee. This often promotes formation of extensive intertidal habitat and lagoons composed of poorly consolidated sediment substrates of varied stratigraphy in the lee area with silt, clay and peat layers (Davidson-Arnott, 2010; Rosati *et al.*, 2010). The presence of poorly consolidated sediment within the accommodation space of a landward migrating barrier can make the barrier vulnerable to subsidence as the load applied causes the substrate to consolidate. Peat is highly compressible, and is more compressible than silts and clays. The magnitude of consolidation of peat depends on its thickness (Rosati *et al.*, 2010).

The use of ground penetrating radar (GPR) to investigate the internal stratigraphy of barrier beaches has been explored at a variety of locations (Neal *et al.,* 2002) as the gravel and sand sediment is of low electrical conductivity (Bennett *et al.,* 2009). Bennett *et al.,* (2009) continue to highlight the need to investigate the use of GPR to gain insight into how barriers evolve over a range of spatial and temporal scales.

For those barrier beaches undergoing a net loss of sediment, this may be mitigated through artificial beach nourishment techniques (Dean, 1983; Coates *et al.,* 2001). Compatible material may recharge the beach during a replenishment, or be recycled from adjacent beaches undergoing a net gain of material. This technique is becoming more favourable to coastal managers as the 'soft' engineering technique is seen to have a higher benefit to cost ratio and provides a sustainable solution. Another coastal management technique that accommodates for the morphological evolution of gravel barrier systems is that of managed realignment, which is becoming a viable option for a variety of sites. It is clear that gravel barriers naturally migrate and adapt to changes in forcing factors and that in some locations, holding them back is not considered sustainable in the future (Cooper *et al.,* 2004). Managed retreat provides additional accommodation space for gravel barriers under long-term sea level rise and can provide habitat creation (Cooper *et al.,* 2004). In some locations, the 'hold the line' principle is maintained for gravel beaches to maintain the beach as primary flood and erosion risk defences. Pevensey barrier beach is an example of an intensive management plan to maintain the level of protection; however this method is expensive and requires ongoing commitment (Sutherland and Thomas, 2011).

2.1.3 Future of Gravel Barriers

By referring to the contemporary morphological evolution of gravel barriers, an insight into the future of gravel barriers can be inferred. In brief, potential future changes in environmental conditions including sediment availability, wave action, water level and accommodation space are likely to impact gravel barrier morphology and evolution (Bradbury, 2000; Dornbusch and Ferguson, 2016), with variations in response dependent on local conditions (Masselink and Russell, 2013).

Future sediment availability is of concern, as the major contemporary glaciofluvial sediment source is of limited supply, and therefore supply via longshore transport and wave driven offshore to onshore transport is reducing. In England and Wales, it is calculated that 30% of the coastline is fringed by gravel beaches, with many sites undergoing a net loss of material (Jones *et* al., 2013). Other sources such as cliff input will vary in the future, as sections prone to erosion are stabilised, often blocking direct supply of material to the beach. For maintained beaches, artificial nourishment of sediment from a range of different land based and marine sources is an increasingly preferred beach maintenance method to increase beach volumes, with recycling of beach material from local areas of natural surplus to areas of erosion as an additional method. Of interest is the emerging increased frequency for material to be placed on the lee slope of barriers during renourishment, as opposed to the front slope to increase beach volume and consequent defence height. This is a relatively new technique to stabilise and widen the barrier crest, in a form of managed realignment as opposed to holding the line, and accepts that the barrier would naturally evolve in this way (Dornbusch and Ferguson, 2016). This method will increase the landward footprint of the barrier, extending into areas that may be internationally designated for their habitat, nature conservation or geological value, and therefore the benefits of making the barrier more resilient to storms will raise habitat management issues. It must be highlighted that, in the absence of human intervention, the barrier would naturally roll back over these designated habitats and so they would be engulfed naturally (Bradbury, 2000; Dornbusch and Ferguson, 2016). There has also been a shift in acceptance of using dredged marine material for beneficial use in beach renourishment schemes if it is of suitable particle size distribution and quality for the intended use. The gravel base layer of Cowes Breakwater was supplied with beneficial use material won from dredging in Southampton Water. Beneficial use can serve as a new source of locally won material, reducing the cost of importing from further afield. The apparent paradigm shift in coastal management from hard defence schemes to soft defence schemes and managed realignment is likely to continue into the future (Cooper *et al.,* 2004), with some management schemes abandoned entirely with the policy of no active intervention. This has occurred most recently at Medmerry (West Sussex) and Cley (North Norfolk) where the barrier beaches were previously maintained and reprofiled annually to main the crest height for flood defence.

Sea level rise is a major global contributory forcing factor causing coastal erosion at a local scale over a greater temporal scale, with concern mounting in rise of predicted rates of sea level rise due to climate change in the future (Zhang *et* al., 2004; Masselink and Russell, 2013; Masselink *et al*., 2015). Predictions of future rates of sea level rise are dependent on isostatic and eustatic changes, and will vary locally due to local conditions. Sea level for the London region is predicted to rise by 18cm by 2040 and 36cm by 2080 based on probabilistic projections for a medium emissions scenario, and includes land movement (UKCP, 2009). Predicted future sea level rise will reduce the available freeboard, the distance between mean water level and the crest of the barrier, resulting in increased water depths, which allow higher waves to arrive at the barrier (Masselink *et al.,* 2015; Dornbusch and Ferguson, 2016). If the accommodation space in the lee of the barrier is suited to roll back of the barrier, then the barrier will be able to maintain its form, especially if the gradient of the land in the lee is of a gradually increasing slope (Dornbusch and Ferguson, 2016).

A further predicted consequence of future climate change is increased wave climate severity due to increased storm frequency and duration, and changes in the prevailing wave direction (Masselink and Russell, 2013). This is predicted to increase coastal flooding and erosion risk (Orford and Pethick, 2006; Masselink *et al*., 2015). The magnitude of increase is more difficult to predict, especially at a local scale. The projected future trends in storm surge by 2100 are <9cm above the current average storm surge levels, but may fall within the expected natural range (UKCP, 2009). Increased storminess would increase the risk of overtopping, and therefore increase the rate at which barriers migrate landwards over time.

It is clear that a mixture of localised factors are important in deciding the future of gravel barrier morphology. Human intervention in the form of coastal protection works aims to stabilise barrier migration in a sustainable way, to keep pace with sea level rise.

2.2 Consolidation

This section aims to clarify the theory of consolidation. Gravel barriers often promote the formation of poorly consolidated materials in their lee, over which they rollback, causing this substrate to consolidate. The theory of consolidation is introduced, and then placed in the context this study. Experimental techniques to calculate consolidation potential are also explored.

2.2.1 Consolidation Theory

Consolidation theory is an important element of soil mechanics for engineering purposes. The geotechnical properties of the ground beneath a structure must be determined before the load is applied to ensure that subsidence does not occur. Settlement within a sand or gravel substrate often occurs instantaneously or over a short period of time. In low permeability soils such as clay or silt, the settlement occurs over a larger temporal scale, from months to years, decades or even centuries after construction due to consolidation (Head and Epps, 2011).

Terzaghi (1925) derived the original theoretical relationship for calculating soil consolidation, and this is reproduced in further works (Terzaghi, 1943; Terzaghi and Peck 1996). Terzaghi (1925) distinguishes between primary and secondary consolidation in order to define the process of consolidation. Primary consolidation is the dissipation of excess pore water pressure from the soil matrix based on fundamental hydraulic principles. Secondary consolidation is the consequent shifting and deformation of the soil grains as they fill the voids left by the pore water. Therefore the term 'consolidation' refers to the process of soil deformation during expulsion of pore water under load (Head and Epps, 2011; Powrie, 2014). The rate at which soil deformation occurs is controlled by the rate of drainage and therefore soil permeability (k) and length of maximum drainage path (d) influences the rate of consolidation. Silts and clays are relatively slow draining but have high consolidation potential (Powrie, 2014). Further detailed information on the derivation of consolidation can be found in Powrie (2014).

2.2.2 Consolidation of Barrier Beaches

The sheltering effect of the barrier from wave attack often promotes formation of intertidal habitat and lagoons composed of poorly consolidated sediments in the lee area such as silt, clay and peat (Rosati *et al.*, 2010). During the deposition of material onto the poorly consolidated substrate, the natural loading that takes place is considered to cause one-dimensional compression, as the surrounding soil prevents lateral strains (Powrie, 2014). Due to the low permeability of the substrate (silty clay), when a load is added it causes an increase in pore water pressure. Pore water is expelled from the soil due to the formation of a hydraulic gradient, and the remaining soil deforms. The pore water pressure gradually obtains an equilibrium, and soil deformation no longer takes place (Powrie, 2014). In this time, the barrier crest has lowered, increasing overtopping and overwashing.

Figure 2 shows a schematic of the landward migration of a barrier over time, and the resulted substrate consolidation and crest lowering adapted from Rosati (2009).

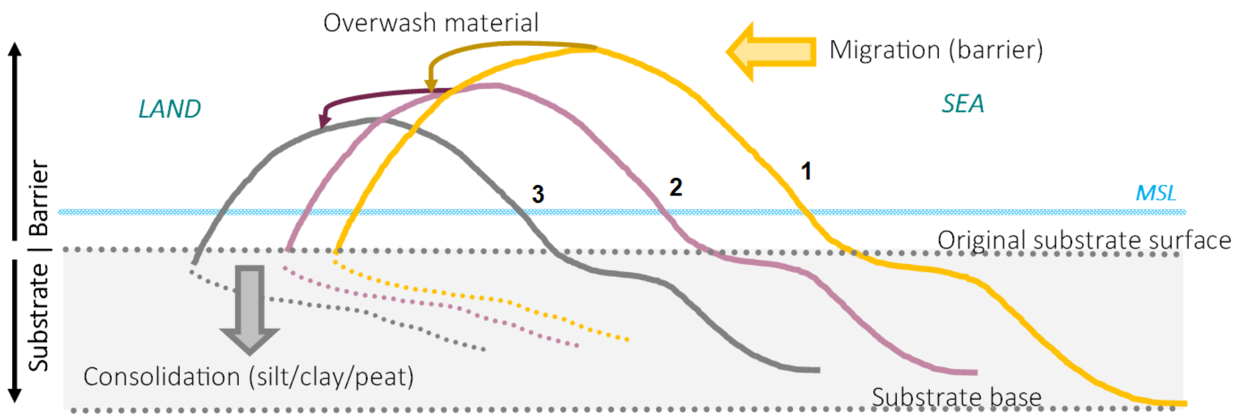

Figure 2: Schematic of the process of landward barrier migration over time and resultant substrate consolidation. Adapted from Rosati (2009).

This thesis is focused on the physical and geotechnical properties of the substrate beneath a gravel barrier, which make the substrate liable to consolidation, and therefore the barrier liable to subsidence. There is a notable lack of other studies within the UK that focus on this overarching theme, with various studies conducted on the fine sandy coastlines of the United States. The lack of studies is often attributed to the lack of quantitative core data to demonstrate the physical and geotechnical properties or thickness of the substrate. The core extraction is complex, and requires technical equipment, time and expenses to conduct. Furthermore, the study sites are often protected by an array of environmental protection designations, which discourage sediment removal. To enable predictions of consolidation to be conducted, an understanding of the physical and geotechnical properties of the substrate are required. This is even more difficult in the coastal zone due to tidal constraints and the soft surface causes access issues. At least, an understanding of the thickness of poorly consolidated substrate is required.

Dean (1983) briefly discusses the presence of marsh deposits beneath barrier islands, and how this leads to amplified landward migration by lowering of the crest elevation and increased vulnerability to overwash. Dean (1983) continues to suggest that if these barriers were to be artificially stabilised through beach nourishment, that the subsidence would only occur to a limit, and so that over time a stabilised beach would be possible. The need to understand where consolidation of barriers is a potential issue, and the consequent requirements when conducting an artificial nourishment is highlighted (Dean, 1983). Rosati *et al.,* (2010) further studied the process of consolidation beneath barrier islands, where the substrate was poorly consolidated deltaic, estuarine, peat or bay sediments and made the barrier vulnerable to accelerated landward migration. Rosati *et al.,* (2010) present a two dimensional (2D) model for cross-shore migration of a barrier island to represent consolidation of the underlying substrate, and barrier migration over time.

Cooper (2015) developed a numerical model of substrate consolidation beneath a retreating barrier beach. Assumptions of the substrate stratigraphy were made, however findings indicated that higher magnitudes of consolidation were expected with increased substrate depth. In addition, it was inferred that substrates of higher permeability and lower stiffness also resulted in greater subsidence. Areas with a layer of peat within the poorly consolidated substrate were also vulnerable to greater consolidation magnitude.

2.2.3 Consolidation Measurement and Prediction

The one-dimensional oedometer test is one of the simplest and most traditional methods for understanding the behaviour of soil during consolidation (Atkinson, 2007; Knappett and Craig, 2012; Powrie, 2014). The standard oedometer test is conducted on samples of low-permeability such as silt or clay to investigate the stress strain relationship in the vertical direction (Head and Epps, 2011; Powrie, 2014). There are various methods and device set-ups available (Head and Epps, 2011). In this case, the 'fixed ring' oedometer type was available to meet the needs of the study, and the operation of the one-dimensional oedometer test follows the British Standard Institution BS 1377, Part 5, currently standard in the UK (British Standards Institution, 1990; Head and Epps, 2011; Knappett and Craig, 2012).

A cylindrical sample of the soil is prepared within a ring with porous disks placed on the top and bottom to drain the sample in the vertical dimension, ensuring that consolidation is one-dimensional (Atkinson, 2007; Powrie, 2014). Axial stress is applied through adding loads in increments to control the loading stress, and the resultant axial strain is measured with a dial gauge at intervals for a period of at least 24 hours (Atkinson, 2007; Head and Epps, 2011; Knappett and Craig, 2012). The methodology used for oedometer operation is described in further detail in Section 3.3.

In a saturated soil such as the substrate used in this study, it does not compress instantaneously after an applied load, however will settle for some time as the void ratio decreases and the soil matrix deforms. This rate of deformation is controlled by factors such as soil permeability (k) and maximum drainage path length (d) (equal to half the specimen thickness) (Powrie, 2014).

Powrie (2014) highlights that as the reduction in void ratio is not instantaneous, neither is the change in effective stress. As the load increments are added, the increase in total vertical stress results in a preliminary increase in pore water pressure. Then, over time as the water is expelled from the pores causing soil consolidation, the excess pore water pressure dissipates (Powrie, 2014). At the end of each increment period, the applied total stress will equal the effective vertical stress in the specimen once the excess pore pressure has dissipated (Knappett and Craig, 2012). To enable an estimation of the consolidation, the relationship between vertical effective stress (σ'_v) and strain (ε'_v) is required. To understand the time over which the settlement occurs, the geotechnical characteristics of the soil need to be tested (Powrie, 2014).

An oedometer test provides:

1) Plots of specific volume (*v*) against the natural logarithm of vertical effective stress ($\ln\sigma'_v$). These plots are used to investigate the behaviour of the soil under load at different depths and can indicate how soil stiffness varies under load.

2) Plots of settlement (ρ) against the square root of time ($\sqrt{t}$) for each load increment (where load is added). These plots are then used to estimate the magnitude of consolidation that will be observed in the field when subjected to an increase in vertical load, and can indicate the theoretical time for 90% of consolidation to occur.

This information will be collected and presented for the purposes of this thesis.

2.2.4 Relevant Case Studies of Consolidation

A study of global and local examples of barriers was conducted to set the wider context of the issue of consolidation into the coastal domain. Other structures such as breakwaters built onto a poorly consolidated substrate are also considered.

There is a distinct lack of case studies within the available literature that specifically study the process of consolidation beneath gravel barriers, at both a global and local scale. Furthermore, very little quantitative data explores the physical and geotechnical properties of the substrate over which the barrier migrates and therefore how vulnerable they are to consolidation processes. For the studies that do exist, the majority are based on sandy barrier islands. The overarching concept is still relevant, as these sandy barrier are still a 'load' and the poorly consolidated substrate is generally still silty clay marine sediments. Kramer (2016) investigated the evolution of the West Belle Pass Barrier (Louisiana, United States of America), in addition to the primary consolidation of its back barrier sediments. This information fed into a conceptual model of back barrier loading originally presented by Rosati *et al*., (2010). It was concluded that consolidation of the barrier is likely due to the presence of poorly consolidated materials in the lee of the barrier, and the likely requirement of future beach renourishment to maintain the protective nature of the barrier beach to large areas of low-lying deltaic land behind the barrier (Kramer, 2016).

Within the UK, there are many examples of gravel barrier beaches, where consolidation of back barrier sediments due to landward migration of the barrier is likely.

Chesil Beach is an iconic gravel barrier beach located on the (central-southern UK) coastline, separating a tidal lagoon 'The Fleet' from the sea (Bennett *et al.*, 2009). The thickness of gravel barrier is likely to be relatively higher than the substrate over which it has migrated, and the poorly consolidated lagoonal substrate is likely to be vulnerable to consolidation under increased overburden. Attempts to use GPR to identify the stratigraphy of the barrier sediments could not be applied to the substrate beneath the barrier as these materials are considered to hinder the radar signal due to increased salinity by saltwater intrusion (Bennett *et al.*, 2009). Few records of the substrate stratigraphy beneath the Chesil Beach are available (Bennett *et al.*, 2009) so it is difficult to make predictions about consolidation at this location. Consolidation of the barrier over time is of importance due to the protective nature of Chesil Beach. The present management policy for Chesil is 'hold the line' (Halcrow, 2011), however the future management of Chesil Beach will account for continuing natural landward barrier migration in light of sea level rise and may include an increase in the cross sectional area, and landward migration of the crest in places.

Slapton Sands is located within Start Bay, on the central-southern UK coastline, separating a freshwater lagoon 'Slapton Ley' from the sea. Slapton Sands is another example of a gravel barrier beach undergoing natural landward migration over poorly consolidated materials, with increased rates of rollback during large storms. In 2001, a severe storm resulted in major damage to the barrier, so that the road that runs along it had to be realigned further landward. (Chadwick *et* al., 2005; Masselink and Buscombe, 2008). Cores abstracted down through the barrier, and in the Ley demonstrate that beneath the gravel barrier layer there is an underlying substrate of muddy saltmarsh sediments. The presence of these sediments highlights that the barrier must have migrated landwards over longer timescales (Chadwick *et al*., 2005), as the formation of muddy sediments requires a sheltered environment. The future management of Slapton Sands is focussed on an important access road which runs almost its entire length. It is concluded that it is not likely to be sustainable to maintain this access road in the future, due to the natural landward migration of the gravel barrier (Masselink and Buscombe, 2008).

During the large-scale replenishment works to the gravel barrier of Hurst Spit, Hampshire in 1996, predictions of expected consolidation due to loading of replenishment material were made, and monitored using displacement beacons. An expected subsidence of 1m proved to be an overestimate with a range of 0.3-0.7m observed during the first 10 years after construction (Brampton *et al.,* 2007). This data were provided by the NFDC Coastal Group in the form of measured displacement from settlement beacons installed after the 1996 recharge scheme, with total settlement to 1999 labelled (Figure 3). It is clear that settlement magnitude varied spatially, with an area at the 'hinge point' the most vulnerable to consolidation (<0.5m in 3 years).

Figure 3: Post-1996 scheme settlement beacon data (1996-1999) courtesy of NFDC Coastal Group, with settlement in metres.

To enable the understanding of consolidation in the coastal context to be studied further, the scope of this section was extended to examples of other structures such as breakwaters, a form of coastal defence, sometimes built on top of poorly consolidated marine sediments in the coastal zone. In preparation for construction, ground testing is often carried out by the engineering company to enable predictions of consolidation to be calculated. Often the design of the breakwater details a specific crest height and run-up to meet the coastal defence requirements of the structure. If the sediment beneath the structure consolidates, and the structure subsides then this design height is not maintained.

To the east of Hurst Spit, within the Western Solent is Lymington Estuary. Lymington Marina is a popular yacht haven located within the Estuary, which is fringed on both sides at its entrance with saltmarsh. This saltmarsh is undergoing erosion, and therefore its protective wave attenuating properties are in decline. In response to this increased risk of higher wave heights propagating up the estuary into the marina, the Lymington Harbour Commissioners developed a multi-phase coastal defence programme that commenced in 2010. The first phase involved the construction of a 100m rock breakwater that spanned an area of saltmarsh and mudflat on the western approaches to the Marina. The second phase involved the construction of a 135m rock breakwater that spanned an area of saltmarsh and mudflat on the eastern approaches of the marina in 2014. Further phases will extend these in length away from the channel on an as-required basis, with an extension to the western breakwater as phase 3 projected to occur between 2024 and 2028 (Lymington Harbour Commissioners, 2015). Appendix B shows the layout and location of the Phase I and II breakwaters. Analysis of lidar data (Channel Coastal Observatory) (2011-2013) demonstrates that consolidation of up to 0.5m occurred along the span of the breakwater, which was in excess of predictions. An additional volume was required during the Phase II breakwater's construction to allow for predicted settlement so that the new design factored in this additional subsidence (Black and Veatch, 2013).

Also located within the Solent area on the southern coast of England is the newly installed offshore breakwater at the entrance to Cowes on the Isle of Wight. The substrate beneath the breakwater was poorly consolidated, and therefore consolidation was factored into the design with use of a geotextile and drainage layer, and installation of settlement beacons in the first stage of construction of the gravel core. Results from the initial monitoring of subsidence were factored in to the final stage of construction when the rock armouring was added to maintain the desired crest height (Cowes Harbour Commissioner, 2016).

It is clear that consolidation is a key process influencing the management of coastal protection at a variety of locations. Not only does consolidation occur beneath gravel barriers, but also beneath any structure, that overlies poorly consolidated marine sediments. Generally, this consolidation is factored into the design, however an understanding of the stratigraphic and geotechnical properties of the substrate is required to make adequate predictions of consolidation magnitude, to maintain the required design crest height of these defence structures.

2.3 Introduction to Hurst Spit

This thesis focuses on Hurst Spit as a study site as this is an example of a barrier beach undergoing landward migration over a poorly consolidated substrate at an accelerated rate. The following sections describe the site, and its past, present and future configuration and management. The substrate underlying Hurst Spit consists of poorly consolidated saltmarsh sediments, silt-filled buried channels and interstratified relict beach gravels with interstitial sand. The barrier was subject to a significant stabilisation scheme in 1996, and it was considered that the overburden of recharge material would result in settlement of the crest of about 1m over 10 years, with rates varying spatially. Measurements up to 1999 indicated up to 0.5m of settlement, most of it in the first year (1996-7) with negligible consolidation in the period 1997-2009 Bradbury et al., 2009).

In response to a series of severe storms, the spit has experienced a net-loss of sediment since the last major scheme in 1996, and requires an imminent recharge of similar proportions and extent to the 1996 management scheme to maintain its future standard of protection. The design requires additional material to be loaded to the landward slope of the Spit, increasing the crest width by more than 20 metres. This would again create a significant new overburden that will result in consolidation of the substrate material, and settlement of the crest. The potential for settlement is of interest to NFDC Coastal Group, who are co-ordinating the design of the future recharge scheme, to ensure that this is factored into the design.

This section discusses the location and description of Hurst Spit, and its past, present and future evolution.

2.4.1 Hurst Spit Site Location and Description

Hurst Spit is gravel barrier located in Christchurch Bay on the Southern central coastline of the UK (Figure 4). It is often referred to as the "Guardian of the Western Solent" (Bradbury and Kidd, 1998) due the protection that it affords to large areas of low-lying coastal land within the Solent.

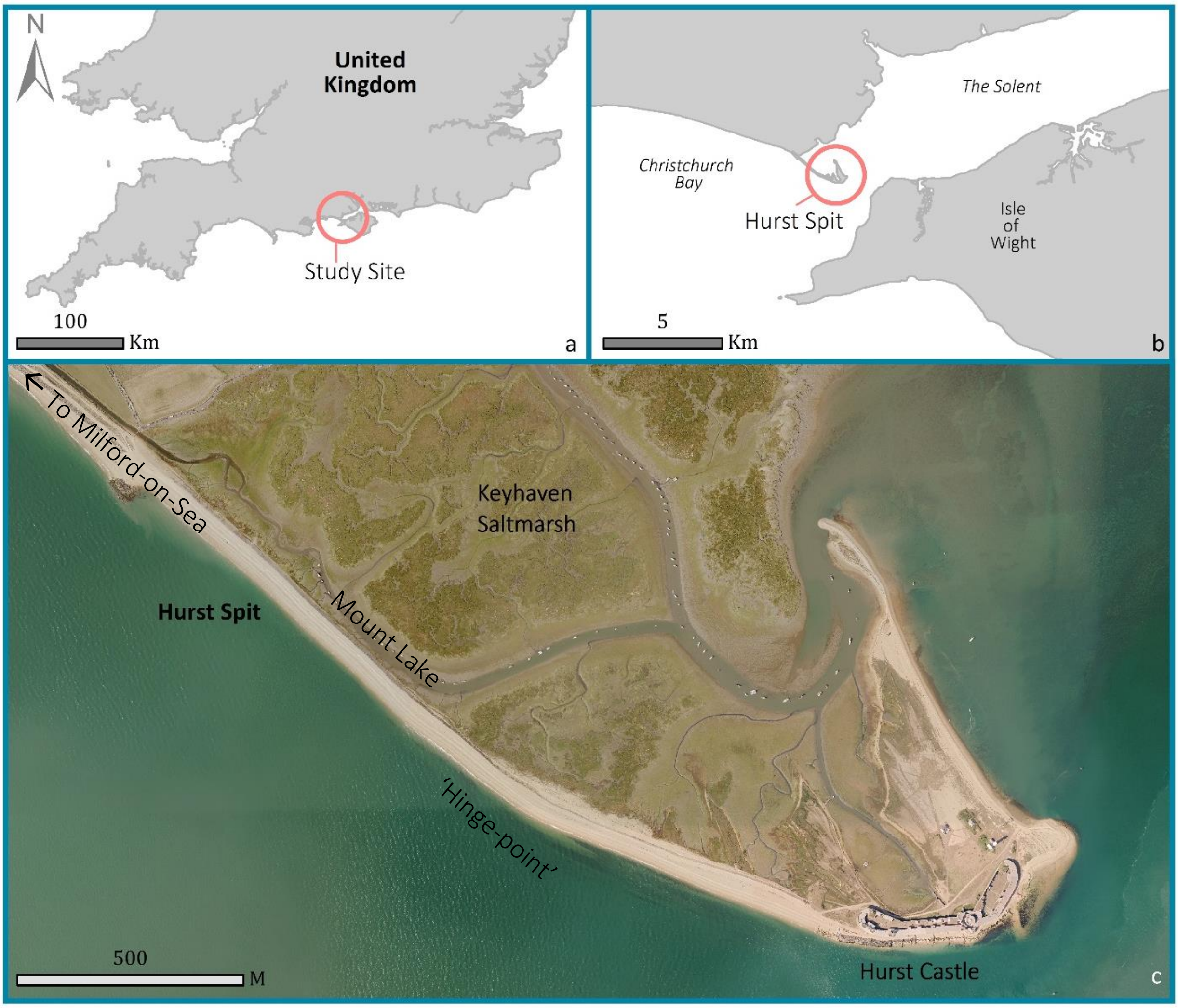

Figure 4: Location of Hurst Spit (a) on the Southern Central coast of the UK, (b) as the 'Guardian of the Solent' and (c) as a gravel Spit, providing sheltering to Keyhaven Saltmarsh and Western Solent. Aerial photography (2016) (C) NFDC, courtesy of the Channel Coastal Observatory.

The Spit It is attached to the mainland at Milford-on-Sea and its configuration can be split into three sections depending on aspect. The first section can be described as extending southeast from the fortified proximal end by 1km to the 'hinge point', which marks the second section where there is a slight deflection in angle for another 1km towards Hurst Castle Point. Hurst Castle marks where the 1km recurve bends and extends to the North West making the Spit approximately 3km in total. Figure 5 shows an image with cross section profile locations to represent these three sections.

Figure 5: Map to show the locations of three key coastal monitoring profiles, to demonstrate the variability in cross section along the length of Hurst Spit. Aerial photography (2016) (C) NFDC, courtesy of the Channel Coastal Observatory.

The first section (profile 5f00045) has much in common with barrier features as it is the most vulnerable to transgression due to overwash as it is orientated to the dominant south westerly wave direction (Nicholls and Webber, 1987). The second (profile 5f00020) and third (profile 5c00584) sections are more typical of recurve features with lower elevations. The typical cross sections of each of the three sections are shown in Figure 6 to demonstrate how the cross sectional area and crest height decreases with distance along the Spit.

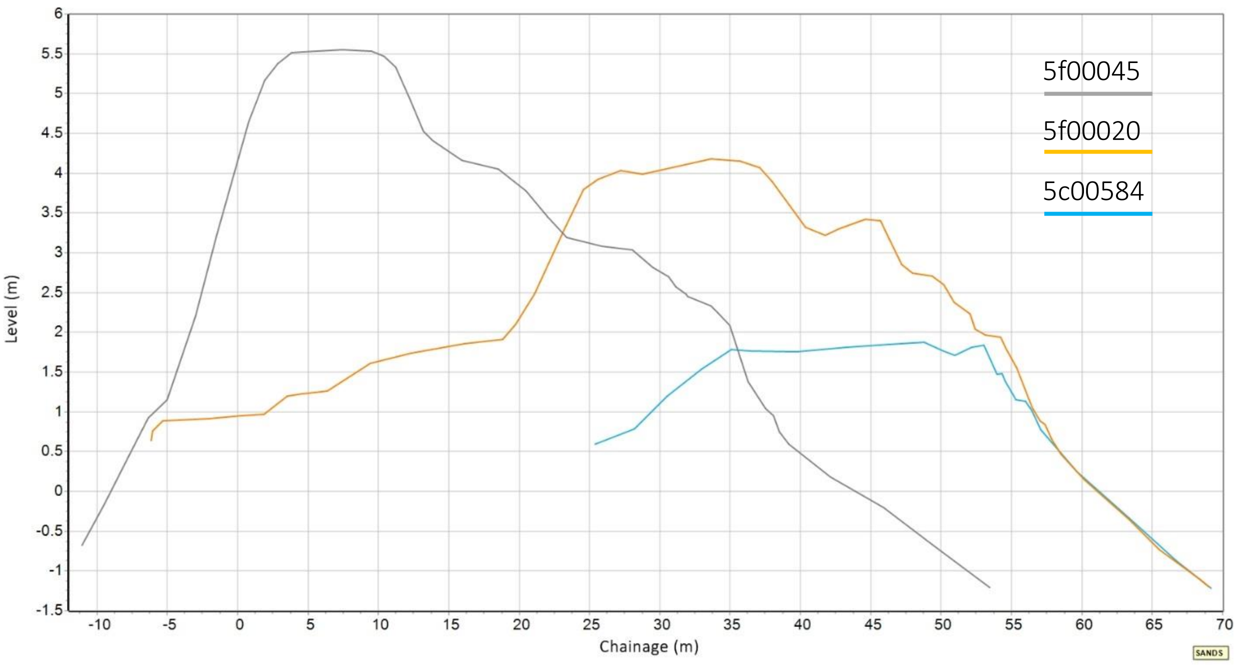

Figure 6: Typical cross sections of the three sections of Hurst Spit (created using SANDS software at CCO).

The dominant sediment type is gravel with a mean diameter (d_{50}) of 15mm (Bradbury 1998; Stripling *et al.* 2008) but is currently a mixture of sub-angular and sub-rounded sand and gravel. This is due to replenishment from a mixture of sources over the past century (inland quarry material, import from the Shingles Bank offshore, and recycling from the distal end at North Point). New sediment sometimes varied in characteristics to the natural sediment (Stripling *et al.,* 2008). There is poor sorting of sediments due to human activity, with some areas of pure gravel, and other areas with a well-defined proportion of sand (Stripling *et al.,* 2008). The crest height has been artificially increased over time, from 2.4 to 4.2m pre-1996; it is currently 4.2 to 5.6 along the main section. The 1996 scheme increased the crest to 8mOD, but has since been reduced through crest trimming.

The tidal range at Hurst Spit during spring tides is small, at 2.2m (Nicholls and Webber, 1987; Stripling *et al.,* 2008); however there are strong tidal currents in the vicinity of Hurst Castle which can reach 2.3m/s (Nicholls and Webber, 1987).

The Spit and saltmarsh are internationally designated as a Ramsar Site, with European designation as a Special Area of Conservation (SAC) and Special Protected area (SPA) and national designation as a Site of Special Scientific Interest (SSSI) (NFDC, 1995; Stripling *et al.,* 2008). The SSSI is designated for the Spit as a site of geomorphological interest, and the saltmarsh is part of the Keyhaven to Lymington saltmarsh SSSI and National Nature Reserve (NNR) for ornithological interest (NFDC, 1995). Hurst Spit provides coastal protection to an extensive area of land in the Western Solent, which is low-lying and vulnerable to coastal flooding such as residential areas, Hurst Castle historic monument and sites of national and international conservation value (NFDC, 1995; Stripling *et al.,* 2008).

In the lee of Hurst Spit lies the Keyhaven Saltmarsh, with areas of saltmarsh vegetation separated by a well-developed creek system and mudflats including the channel at Mount Lake. Hurst Spit has been rolling back over these sediments, and therefore the substrate beneath Hurst Spit is likely to be consolidated saltmarsh and mudflat deposits (Figure 7) overlying a gravel bed of 'Plateau gravels' (Nicholls, 1985).

Figure 7: A view southeastwards along Mount Lake towards Hurst Castle, from the lee of Hurst Spit, showing the poorly consolidated material of the Keyhaven intertidal mudflats.

There is a lack of understanding of the configuration and stratigraphy beneath Hurst Spit. A survey undertaken by British Geological Survey at Hurst Spit to enable an understanding of beach thickness using a Tromino passive seismic device at two locations. It concluded that the sand and gravel proportion of beach sediments varied, and that this is possibly due to remediation works. Attempts were made to establish the thickness of poorly consolidated material beneath Hurst Spit, however discussion of results implies that further work is required, including verification with boreholes through the gravel barrier and substrate beneath (Raines and Morgan, 2016).

2.4.2 History of Hurst Spit and Management

The history of Hurst Spit may be categorised into two phases: millennial, long-term natural evolution, and its relatively recent management and human intervention on a centennial scale. Historically, Hurst Spit has been vulnerable to breaching during severe storm events and a net loss in sediment supply. A detailed chronology of the millennial scale evolution is provided by Nicholls (1985) and Nicholls and Webber (1987). In essence, Hurst Spit is thought to have commenced formation as a result of a series of marine transgressions which eroded material and formed the Christchurch Embayment at least 7000BP (Nicholls, 1985; Nicholls and Webber, 1987).

Human intervention commenced in the late 18th Century, originally through mineral extraction and beach mining. The extracted materials were used to construct buildings and supply industry, often enhanced coastal erosion. The construction of coastal defence structures originated in the 1900's in attempt to reduce soft cliff erosion, with large-scale groyne construction in the 1940's along the Bournemouth and Christchurch coastline dramatically reducing the longshore drift rate further east towards Hurst Spit. Volumes of sediment reaching this furthest downdrift frontage were massively reduced, and beach volumes loss accelerated (NFDC, 1995). Notable storm damage occurred during events in 1954, 1962, 1981-82, 1989-90, where overtopping and crest lowering occurred, sometimes leading to breaching. The rate of transgression increased during these storm events with predictions of 1.5m per year in the preceding period 1867 to 1968 increasing to 3.5m per year from 1968 to 1982 (NFDC, 1995; Nicholls and Webber, 1987).

The need for a large-scale coastal protection scheme was further emphasised by storm damage in 1989, and was implemented in 1996 by New Forest District Council, involving a major gravel renourishment and construction of a rock revetment and breakwater (NFDC, 1995; Brampton *et al.*, 2007). The recharge almost doubled the volume of the pre-scheme Spit and increased the crest width and height to between 5 and 7m. Allowance for settlement due to compaction of the substrate was factored into the design (Brampton *et al.*, 2007). Evidence of failure of the poorly consolidated material due to excessive loading was reported through observations of mud squirting out from beneath as the recharge material as it was added, and these localised areas of rapid failure and consolidation made loading of gravel difficult, so the method of loading was adjusted to smaller stages (Brampton *et al.*, 2007).

Recent History of Hurst Spit

A series of severe storms during winter 2013-14 impacted Hurst Spit, with wave heights of 4.5m measured at Milford on Sea generated during the highest magnitude event on the 14th February 2014 (Bradbury and Mason, 2014). Seven of the 15 highest storms (exceeding a 1 in 1 year return period) since 2003 at Milford-on-Sea were recorded between October 2013 and February 2014, with the 14th of February storm reaching a 1 in 50 year return period. The many sections of the Spit from the proximal point to Hurst Castle saw reduced crest width and height, with prominent overwash fans (Dornbusch and Ferguson, 2015). Approximately 47,000m^3 was lost between March 2013 and February 2014 (Bradbury and Mason, 2014). In response to the severe storm damage, repairs were conducted to reinstate the desired profile cross section, using sediment recycled from the overwash fans and locally from North Point, with sediment reprofiled on the back slope. The back slope is now steeper, but the crest lower as can be seen in Figure 8.

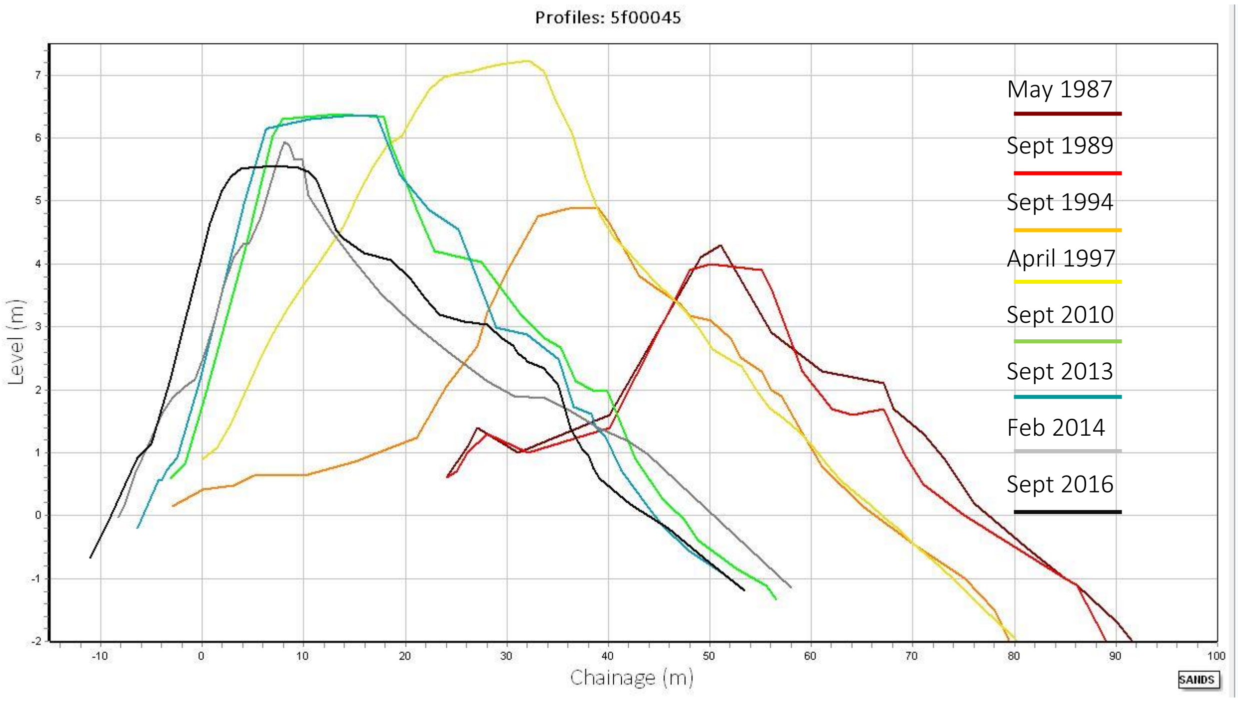

Figure 8: Cross section at profile 5f00045 to show change over time.

Figure 8 also demonstrates the long-term rollback of the barrier at 5f00045 1987 to 2016. It clearly shows the change in profile after the 1996 recharge, the highest that the profile was built. It has since lowered due to subsidence, and crest reprofiling and trimming. In almost 30 years, the crest has moved landward by almost 50 meters.

Recycling of sediment from North Point to localised areas of erosion is still undertaken every 3-4 years of about 5,000m^3, nevertheless Hurst Spit continues to decline in volume, loss of sediment from the vicinity of Hurst Castle estimated at 2,500m^3 per year (NFDC Coastal Group, 2016 pers.comm.). The need for a large replenishment is apparent to maintain the desired crest height and width along the crest.

2.4.3 Future Hurst Spit and Management

This section provides further information on the future management of Hurst Spit, based on discussions with New Forest District Council Coastal Group (2016 pers.comm.). Since the last major repairs to winter 2013/14 storms, only minor maintenance has been required as no significant storms have occurred. The NFDC Coastal Group recognise that this grace period has been advantageous in allowing natural recovery of the barrier foreshore, however are aware that the majority of the Spit lies below the desired design level, with some stretches vulnerable to future storms (greater than 1:10 years return period) due to narrow crest widths. Large-scale beach recharge using externally sourced material has been possible during the past 20 years, but has been deferred due to the ability to recycle material within the local area from stockpiles and North Point, and through trimming of crest material. The finite nature of these sources of sediment for recycling have prompted plans for the next phase of major replenishment. The preliminary design anticipates that a wider crest will be essential to maintain the design level required. Figure 9 demonstrates an example profile (5f00045) with current cross sectional profile, and potential post recharge profile. The current crest width along the first kilometre is 8m, but it is proposed to increase the crest width landwards by up to 24m, to 32m. The design crest elevation is 5.6mOD with a 1:2 back slope. This new cross section with wider crest is to account for future erosion over a 20-year period, and will meet the design standard.

The current shoreline management plan policy for Hurst Spit is to 'hold the line' (NFDC, 2010), however it is often difficult to maintain a set line, when the barrier beach migrates naturally in response to forcing factors such as storm events, sediment supply and water level. The next phase of recharge may involve placement of large volumes of sediment on the lee slope of the barrier to maintain the desired crest width, with the back barrier advancing landward, over the poorly consolidated back barrier sediments. Figure 9 demonstrates the preliminary plans whereby material will be added to the barrier so that the back slope of the barrier may advance over 20m landwards, resulting in a larger footprint of saltmarsh engulfed by the barrier. This landward translation of the barrier over saltmarsh has however been occurring naturally over time (100m in 60 years). Adding or reworking material to the seaward face of Hurst Spit is likely to be unsustainable as the material is likely to be transported along shore, and not retained in the cross-shore profile. Managed realignment is emerging as a more sustainable option for the future in light of sea level rise.

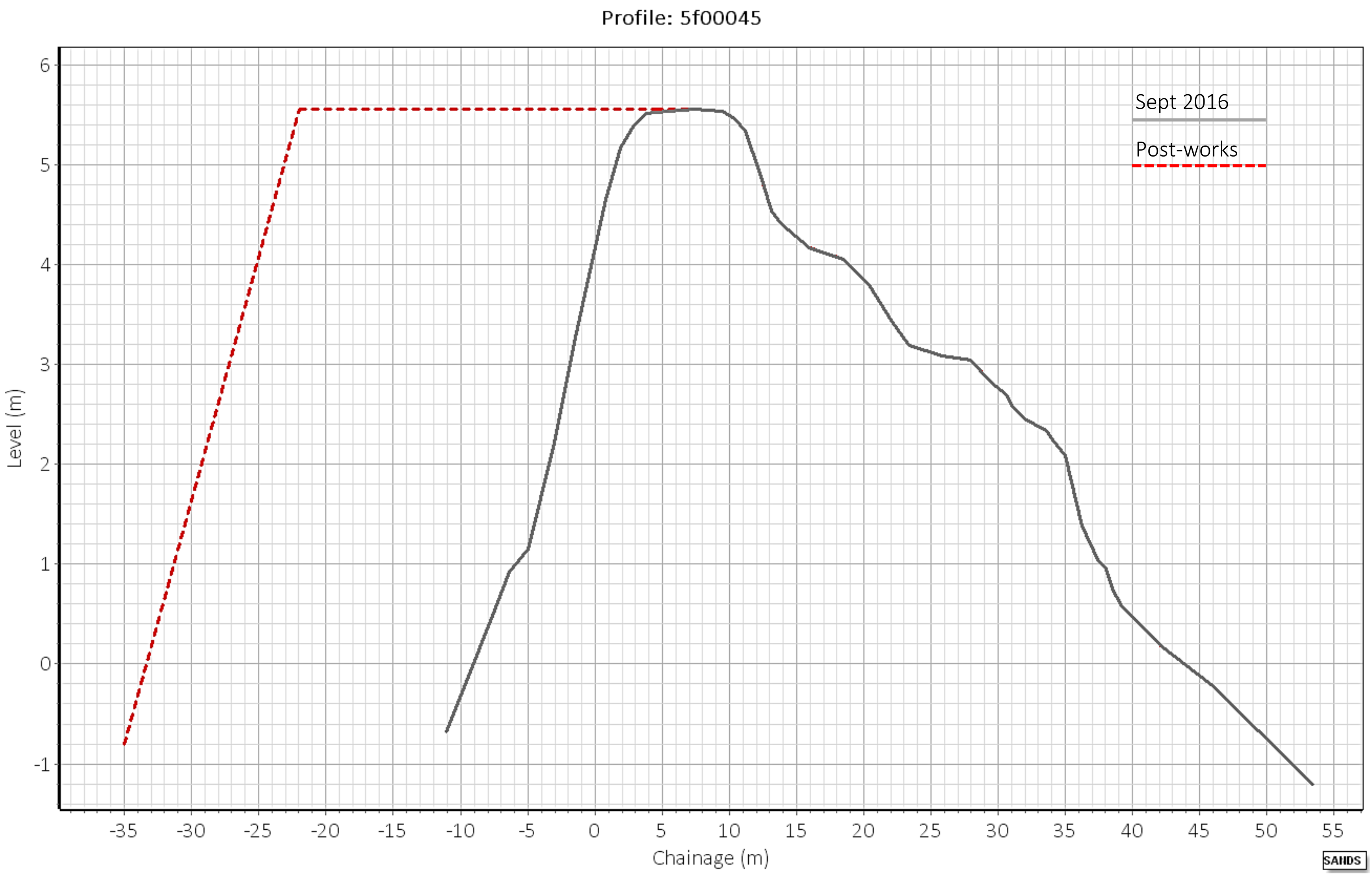

Figure 9: Preliminary design CSA of Hurst Spit at profile 5f00054 for the next major replenishment compared to most recent CSA.

2.5 Summary of Literature Review

A literature review was undertaken to source information relevant to this study, and covered a wide range of literature sources. Through a detailed overview of beach nomenclature relating to classification of gravel beach barriers, an understanding of their form, morphological evolution and future evolution is gained. By then focusing on the main process of consolidation, both in the general soil mechanics and engineering context, and applying it to the context of the coastal zone with a variety of case studies, the aims of this study are set into context. Hurst Spit is introduced as a case study, an important example of a landward migrating gravel barrier, vulnerable to subsidence due to the poorly consolidated back barrier sediments. A literature review, which originated from the selection of this study site explored the real world application of the theory of consolidation and will enable an understanding of the implications for coastal managers to be gained.

3. Methodology

This section introduces the methodology required to meet the objectives of this thesis, including historical shoreline analysis, sediment coring, sedimentary and geotechnical analysis. Limited work has been conducted on the recent shoreline dynamics and therefore a historical shoreline analysis was conducted to demonstrate the landward migration of the barrier. Sediment cores were conducted to enable a greater understanding of the stratigraphy and geotechnical properties of the back barrier sediments at Hurst Spit, with analysis conducted in the laboratory.

3.1 Historical Shoreline Analysis

This element was conducted to enable the identification of suitable locations for sediment sampling, to support this thesis' first objective to 'conduct representative sediment sampling'. The preliminary requirements were to acquire cores in the lee of Hurst Spit, in the area likely to be within the future footprint of the barrier as it migrated landwards. The first kilometre stretch originating from the end of the rock revetment was highlighted as a focus point of the study through meetings with the local authority coastal engineering team. It was thought that this area is vulnerable to washover during extreme storm events due to its south-westerly facing aspect.

Digitisation of the back barrier from historical aerial photography was conducted in ArcGIS software. The historical aerial photography was kindly provided by New Forest District Council and the Channel Coastal Observatory, dating back to 1947. Digitisation of the back barrier indicates the rate of landward migration of Hurst Spit. Areas that have undergone faster rates of landward retreat can be identified. Analysis of historical aerial photography over time can also identify the substrate over which the barrier has migrated, distinguishing between areas of saltmarsh, mudflat or creek system beneath the barrier. Core locations were selected from within the area likely to be within the footprint of the barrier as it migrates landward in the future. The latest (2016) aerial photography was used to identify suitable areas in the lee of the barrier to sample from. The substrate must be previously unconsolidated, as identified through the historical aerial photography.

3.2 Acquisition of Back Barrier Sediment Cores

This element was conducted to meet the first objective to 'conduct representative sediment sampling'. There is a lack of understanding on the depth and stratigraphy of the sediments in the lee of Hurst Spit. Due to the lack of information on the physical and geotechnical properties of the sediment substrate beneath the gravel barrier at Hurst Spit, a programme of sediment sample acquisition was conducted. In order to conduct fieldwork that included the removal of <1m^3 of sediment from a site protected under statutory environmental designations, permission was sought and granted through the Marine Monitoring Organisation, Natural England and NFDC.

A preliminary fieldwork trial tested the use of the extendible Dutch gouge, hand auger and Russian peat corer, to find the most appropriate technique. The findings of the trial concluded that the extendable Dutch gouge was the most appropriate method to produce 'disturbed' sediment samples down to the base of the sediment substrate for particle size analysis. It also highlighted the importance of conducting the main fieldwork campaign at low tide during spring periods to enable a low enough water table for the acquisition of intact sediment samples. This restricted the time frame that samples could be acquired.

The requirement of a method for obtaining 'undisturbed' sediment samples for geotechnical analysis was also identified. The correct coring technique can provide samples where the structure and water content are preserved as much as possible to try and represent in-situ conditions for geotechnical testing (Whitlow, 1995). During the main fieldwork campaign, a drill rig was kindly provided by Soils Ltd in May 2016 for one day (Figure 10). The drill rig was able to produce two undisturbed cores with the Dutch gouge providing the majority of the sediment samples.

Figure 10: Image of drill rig at Hurst Spit, kindly provided by Soils Ltd.

The desired core locations were first identified during the historical shoreline analysis, to ensure that the coverage was suitable. All sites were in the lee of the barrier within the anticipated footprint of the barrier in the future. Preferred locations were regularly spaced, and in line with monitored beach profiles. In the field, an RTK GPS system was used to navigate to the desired core locations, and then a dynamic assessment of that site was conducted in the field. Presence of gravel overwash on the lee slope often meant that the final site selection had to be more flexible to ensure that coring was successful. Core locations were named by the nearest cross-sectional profile number. The main aim of the sediment sampling was to meet the second objective of this study to 'establish the physical and geotechnical properties of the substrate'. This meant that cores needed to meet the substrate base geology, a relatively impermeable layer of 'plateau gravel'.

The core locations are shown in Figure 11 and further detailed in Table 1.

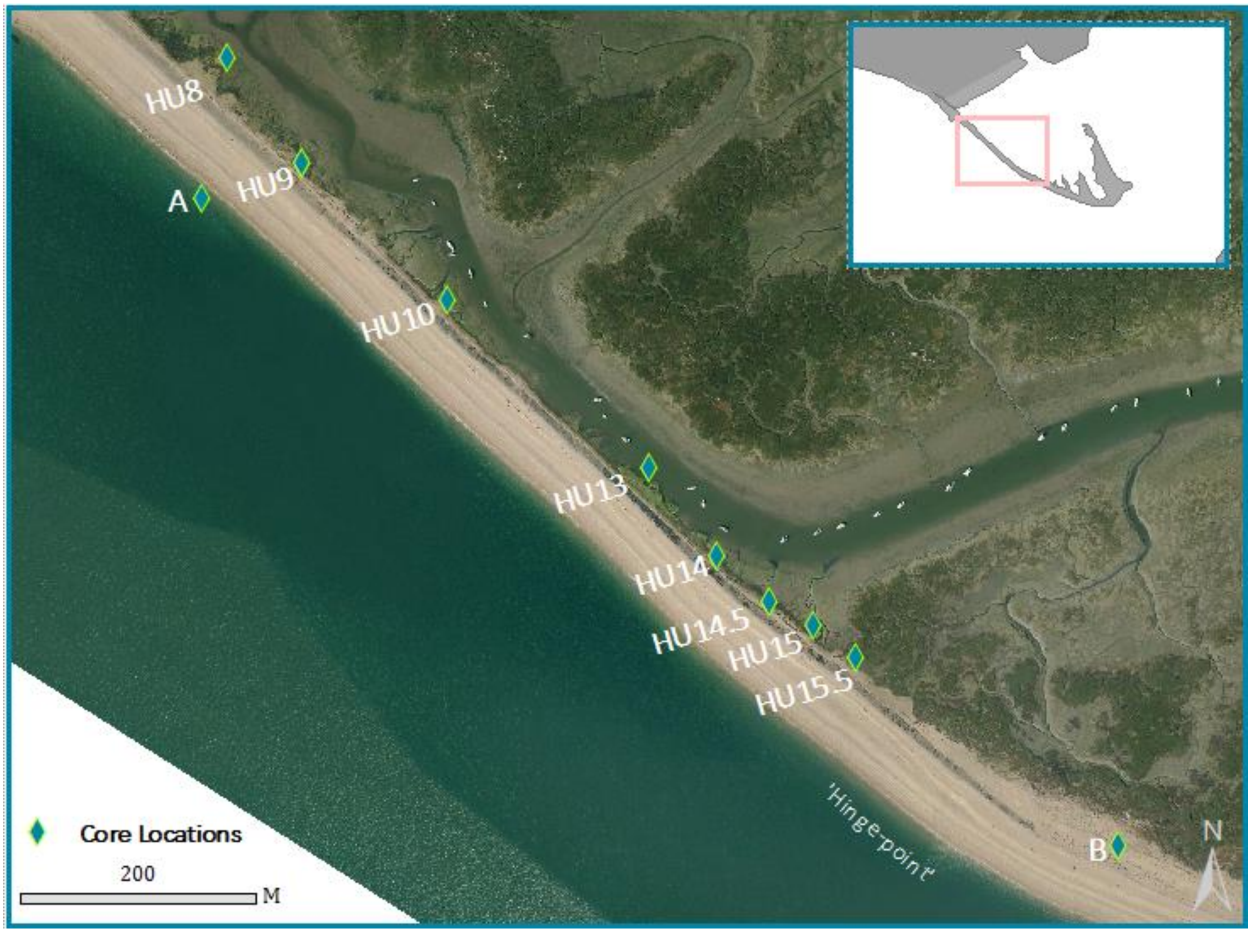

Figure 11: Map to show locations of cores. Aerial photography (2016) (C) NFDC, courtesy of the Channel Coastal Observatory.

Core Location	Profile Number	Easting	Northing	Core Type	Maximum depth achieved (mOD)
HU8	5f00052 (HU8)	430173.7	90688.5	Dutch Gouge Box Core	0.00 to -2.50
HU9	5f00049 (HU9)	430236.4	90598.4	Drill Rig	0.90 to -2.80
HU10	5f00045 (HU10)	430358.2	90481.1	Dutch Gouge	-0.40 to -2.15
HU13	5f00039 (HU13)	430526.3	90336.6	Dutch Gouge	-1.19 to -4.45
HU14	5f00037 (HU14)	430583.3	90260.4	Dutch Gouge	-0.48 to -1.28
HU14.5	5f00036	430626.7	90220.5	Drill Rig	0.43 to -4.57
HU15	5f00034 (HU15)	430663.	90200.6	Dutch Gouge	0.12 to -4.08
HU15.5	5f00033	430699.4	90172.2	Dutch Gouge	0.57 to -2.93
A	5f00049-51	430150.0	90570.0	Hand Auger	Assuming 0 to -3.6m
B	5f00026	430920.0	90010.0	Hand Auger	0.6 to -4.8

Table 1: Core locations.

Please note that location ‘A’ and ‘B’ are cores presented by Nicholls (1985) and Nicholls and Clarke (1986). At the time that core ‘A’ was extracted, it was landward of the barrier. The location has since emerged at the front of the barrier, and therefore the stratigraphy will have been altered through consolidation due to the loading of the landward migration of the barrier. Core ‘B’ was also clear of the back slope of Hurst Spit in 1985, but now lies underneath the overwash fans. The sediment within these cores is likely to have been altered geotechnically by the load imposed by the gravel barrier as it migrated landwards, and are no longer considered as ‘normally consolidated’. These locations have been included in Figure 11 for purposes of historical interest, and have not contributed to the understanding of the present day stratigraphy at these locations.

3.3 Sediment Analysis

This element was conducted to meet the second objective of this study to ‘establish the physical and geotechnical properties of the substrate’. The use of the sediment analysis laboratory at the National Oceanography Centre (NOC) was kindly provided to enable analysis of the physical properties of the substrate. Experiments conducted included particle size analysis, and tests for water and organic content. The British Ocean Sediment Core Research Facility (BOSCORF) kindly offered to refrigerate the sediment samples prior to analysis to ensure that they were preserved.

In the field, the cores were split into 10-30cm sections dependent on visual inspection of soil type and properties (sediment grain size, stiffness, composition, colour *etc*.). This enabled detail of variation in physical properties with depth to be captured, and ensured that the processing of sampling was appropriate to the time availability. Each subsample was sealed in a zip lock bag and logged separately.

In the sediment laboratory, each sample was further subsampled and processed using the following steps:

1. A small subsample (~30g) was retained as a reference sample
2. A further subsample (~15g) was dried at 50°c to remove any water, and then heated at 400°c in the furnace to enable organic content to be calculated.
3. The remainder of the main sample was first dried at 50°c to test for in-situ water content, then wet sieved to remove the fine fraction <63µm, dried and re-weighed to calculate the fine fraction proportion.
4. Any sediment that remained was dry sieved through 2mm to distinguish the proportion of sand and gravel in the sample. A full particle size analysis was not possible to complete within the timeframe available.

3.4 Geotechnical Analysis

This element was conducted to meet the second objective of this study to 'establish the physical and geotechnical properties of the substrate'. Geotechnical analysis was only conducted on 'undisturbed' samples. The samples were all moderately stiff marine mud similar to that in Figure 12 below.

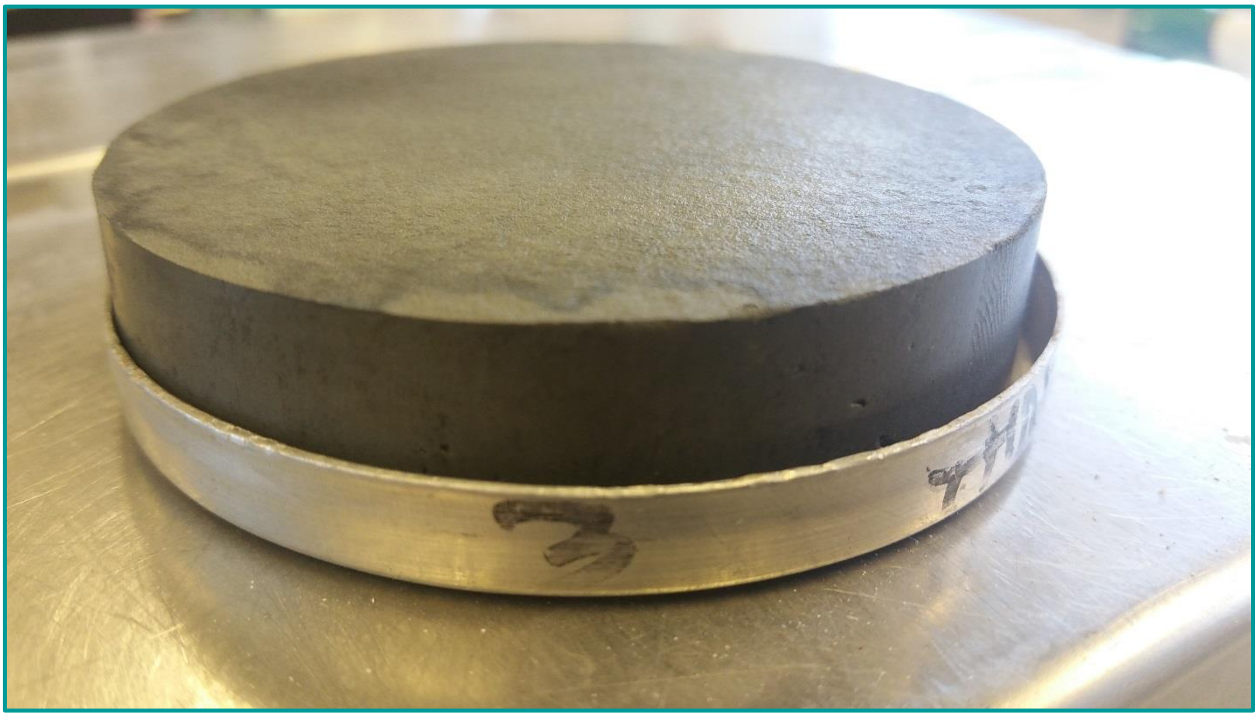

Figure 12: Example of the marine mud used for oedometer testing.

Oedometer equipment was used to test for geotechnical properties. The principle of an oedometer is described in Section 2.23. In this section, the methodology for operation of the oedometer is described. The methodology aimed to follow the standard UK procedure given in BS 1377: Part 5 1990. Two different one-dimensional consolidation devices were used during the testing phase of this study. The traditional oedometer press (Figure 13a) applies loads in increments using hanging weights. Two new Automatic Consolidation Frames (ACF) (Figure 13b) were made available for testing.

Figure 13a: Traditional Oedometer set-up.

Figure 13b: Automatic Consolidation Frame.

These apply load with a load transducer, which compresses air to the required set Newtons (N) with automatic logging available for use. The traditional oedometer required samples of 50mm diameter and 76mm, with the ACF only able to take 75mm diameter samples. Samples were prepared using the following steps:

1. Weigh the consolidation ring and measure dimensions such as diameter D (mm) and initial specimen height h_0 (mm) using internal Vernier callipers. Ensure smooth ring surfaces for cutting.
2. On a glass plate, place the core. Press the oedometer ring into the selected sample surface and use cutting tools to trim the excess so that a cylindrical sample is retained within the ring, with no air gaps. Retain the excess trimmings to determine initial moisture content w_0 (%) of the sample.

3. Weigh the consolidation ring containing the finally prepared sample. Add filter paper to the top and bottom.
4. Assemble the component parts (Figure 14a and 14b): retaining ring, consolidation ring porous discs, loading cap and set in place on the one dimensional consolidation device. Set up the displacement transducer and calibrate to zero.
5. Ensure the one dimensional consolidation device is calibrated for use.
6. Set up stopwatch, recording sheet and necessary loads required.

Figure 14a: Component parts of oedometer cell.

Figure 14b: Assembled component parts of oedometer cell.

The locations of the samples, accompanied by the depth of sample, description and equipment used are given in the Table 2 below. Suitable undisturbed samples were more difficult to obtain. Suitable samples were homogenous, fine-grained mud, deemed to be undisturbed and of suitable diameter. Any cores of <50mm diameter were regrettably unsuitable and included the peat cores. The samples are therefore from two locations, and aim to see if there is any difference in stiffness between locations, and with depth.

Location Reference	Depth (m)	Description	Equipment (cell diameter)
HU8 (box sample)	0.2-0.3	Mud	Traditional rig (50mm)
HU14.5 (drill core)	0.6-0.7	Mud	Automatic Consolidation Frame (75mm)
HU14.5 (drill core)	0.6-0.7	Mud	Automatic Consolidation Frame (75mm)
HU14.5 (drill core)	1.6-1.7	Mud	Traditional rig (76mm)
HU14.5 (drill core)	2.6-2.7	Stiff Mud	Traditional rig (50mm)

Table 2: Locations of samples, depth, description and equipment used.

An initial load for soils of very soft consistencies must be very low, less than 20kPa, and probably closer to 6kPa (Head and Epps, 2011).

To gauge the maximum real life overburden of the gravel barrier beach, the unit weight of the beach needed to be calculated. The unit weight (γ) can be described as the weight of a unit volume (kN/m^3). A sample of beach material was taken from Hurst Spit (sand and gravel mix), and the unit weight was calculated as 16kN/m^3. The crest height of Hurst Spit is a maximum of 6mOD, with the surface of the back barrier sediments around 0mOD, making the maximum future overburden approximately 6m. This would result in a maximum total vertical stress (σ'v) of up to 100kPa. The oedometer uses a sample at a smaller scale, and uses a lever to apply this equivalent load to the smaller sample and replicate the same stresses as the real life situation would do.

Due to the varied cell diameters (and therefore surface areas), the same load can be applied with the oedometer, but will result in differential vertical stresses. The traditional oedometer rigs required load to be set using hanging weights, whereas the ACF required loads to be set in Newton (N). Flexibility in weights available was restricted to multiples of 100g. The load applied in N to the ACF could be set as desired. Table 3a demonstrates the loads and equivalent total vertical stress on the samples for the 50mm and 76mm traditional oedometer rig. Table 3b demonstrates the equivalent load required for the 75mm ACF to maintain similar total vertical stresses. Due to the larger diameter of the 75 and 76mm cells, a higher load was required for the oedometer to reach an equivalent total vertical stress than for the 50mm cell. It was difficult to achieve exactly the same conditions of a 6m crest and beach overburden (100kPa), this was due to the restricted nature of the hangar weights, and therefore the closest increment of load was used. This is highlighted in blue in Table 3a and 3b. In all cases, the maximum vertical stress had to exceed 100kPa, due to the increments of load available. This was suitable to ensure that the desired load was reached, but may result in an overestimate of consolidation. This situation is also acceptable where material will be placed in the channel and other areas below 0mOD as the overburden will need to be increased to maintain a 6mOD crest height. The required load for the 50mm oedometer to reach an overburden of 6m would be 2150g, whereas for the 75 and 76mm oedometer samples, it would be 4800g (424N).

D (mm)	50		76	
A (m^2)	0.001963		0.004536	
Load on hangar (g)	Total Vertical Stress (kPa)	Equiv. load height (m)	Total Vertical Stress (kPa)	Equiv. load height (m)
100	5	0.3	2	0.1
200	9	0.6	4	0.2
400	18	1.1	8	0.5
900	40	2.5	18	1.1
1800	81	5.1	35	2.2
3600	162	10.1	70	4.4
7200	324	20.2	140	8.8
14400	648	40.5	280	17.5

Table 3a: Relationship between the load on hangar, total vertical stress and equivalent overburden height of gravel barrier for the traditional (50mm and 76mm) oedometer rigs. The maximum load required is highlighted in blue.

	D (mm)	75	
	A (m^2)	0.004418	
Load on hangar (g)	Equivalent (N) for ACF	Total Vertical Stress (kPa)	Equiv. load height (m)
100	8	2	0.1
200	17	4	0.3
400	35	8	0.5
900	80	18	1.1
1800	159	36	2.2
3600	318	72	4.5
7200	636	144	9.0
14400	1271	287	18.0

Table 3b: Relationship between the load on hangar, equivalent (N), total vertical stress and equivalent overburden height of gravel barrier for the ACF rig (75mm) oedometer rigs. The maximum load required is highlighted in blue.

3.5 Summary of Methods

Section 3 introduced the methodology required to meet the objectives of this thesis, including a description of the historical shoreline analysis, sediment coring and sedimentary and geotechnical laboratory analysis. The methodology was carefully selected to ensure that it could provide a suitable amount of information within the time and resource constraints. The methods used were successful in producing results of interest to this thesis, and will be presented in the next section.

4. Results

This section presents the results of the thesis, including results of the historical shoreline, sediment and geotechnical analyses.

4.1 Historical Shoreline Analysis

The back of the barrier beach was digitised where historic records of aerial photography were available. This aimed to demonstrate the landward migration of the barrier over the last 60 years. Figure 15a shows the digitised landward extents, overlaying 1946 aerial photography. This indicates the surface over which the barrier has migrated. This varies from saltmarsh vegetation, and channels. It can also demonstrate the spatial variation in retreat, and therefore the stretches that were more vulnerable to overtopping and overwash which led to landward migration. Figure 15b also shows the digitised landward extents, overlaying the latest 2016 aerial photography. This demonstrates the present configuration, relative to historic positions. The present landward extent has been relatively stable since the 1996 large-scale coastal protection works, with minor adjustments landward after the 2013/14 storm damage and consequent post storm works.

Figure 15a and 15b demonstrate that, in some areas, the barrier has migrated an average of 100m in 60 years, with an average rate of 1.67m per year. Areas close to the proximal end of the spit have undergone high rates of landward migration. Areas either side of the 'Hinge-point' have also undergone higher rates of landward migration.

The barrier has sequentially migrated landwards toward Mount Lake channel over time due to storm overtopping and overwashing, and consequent post-storm coastal protection works to stabilise and maintain the barrier. Mount Lake itself appears to have maintained the same position over time, however the barrier has slowly encroached and appears to now be directly on the channel banks. Future landward extension of the back barrier will result in recharge material being placed into this channel, which may prompt the channel to migrate in response, the first movement in more than 60 years.

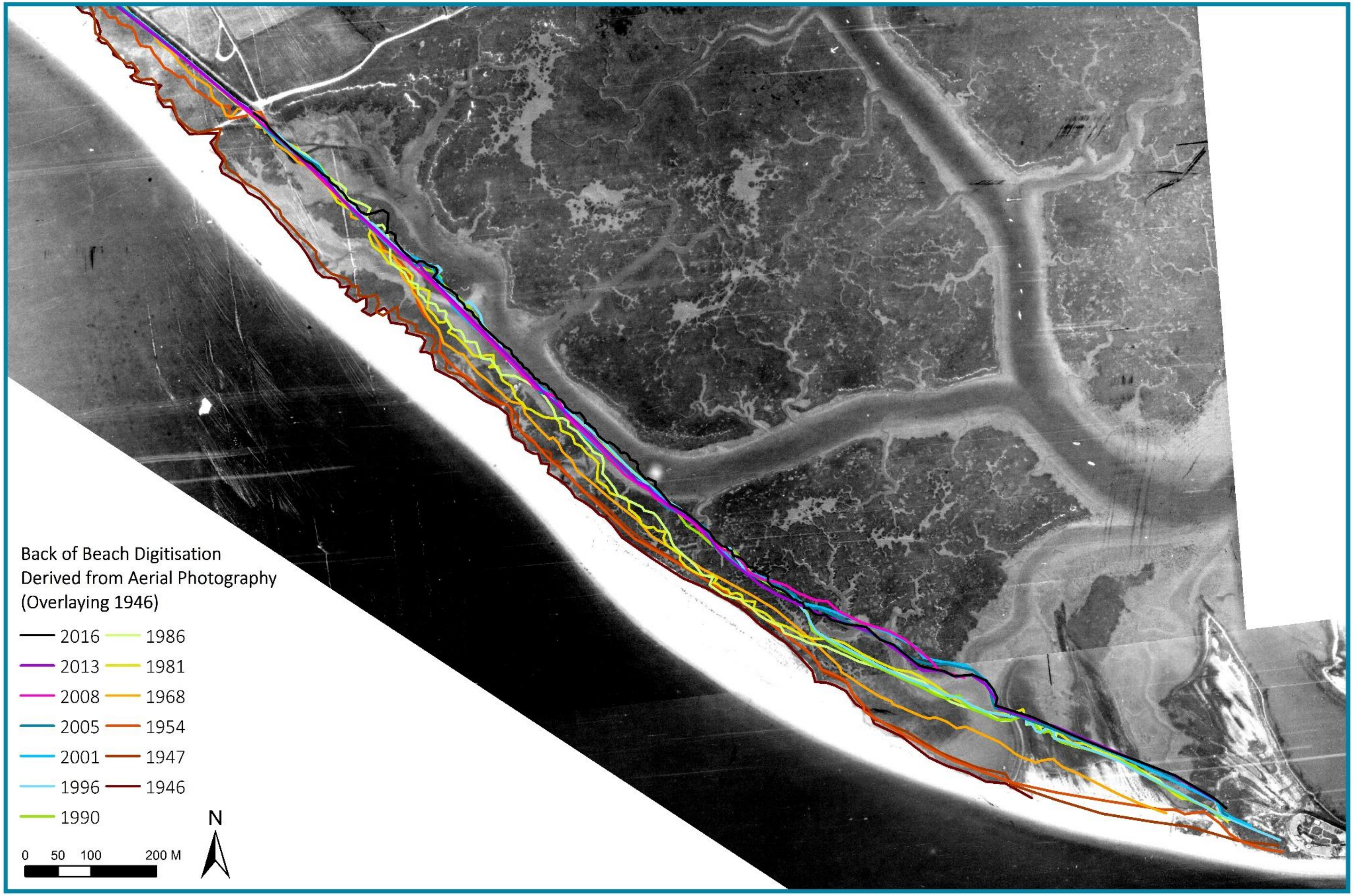

Figure 15a: Back of beach digitisation derived from historic aerial photography (1946 to 2016) overlaying 1946 aerial photography courtesy of the NFDC

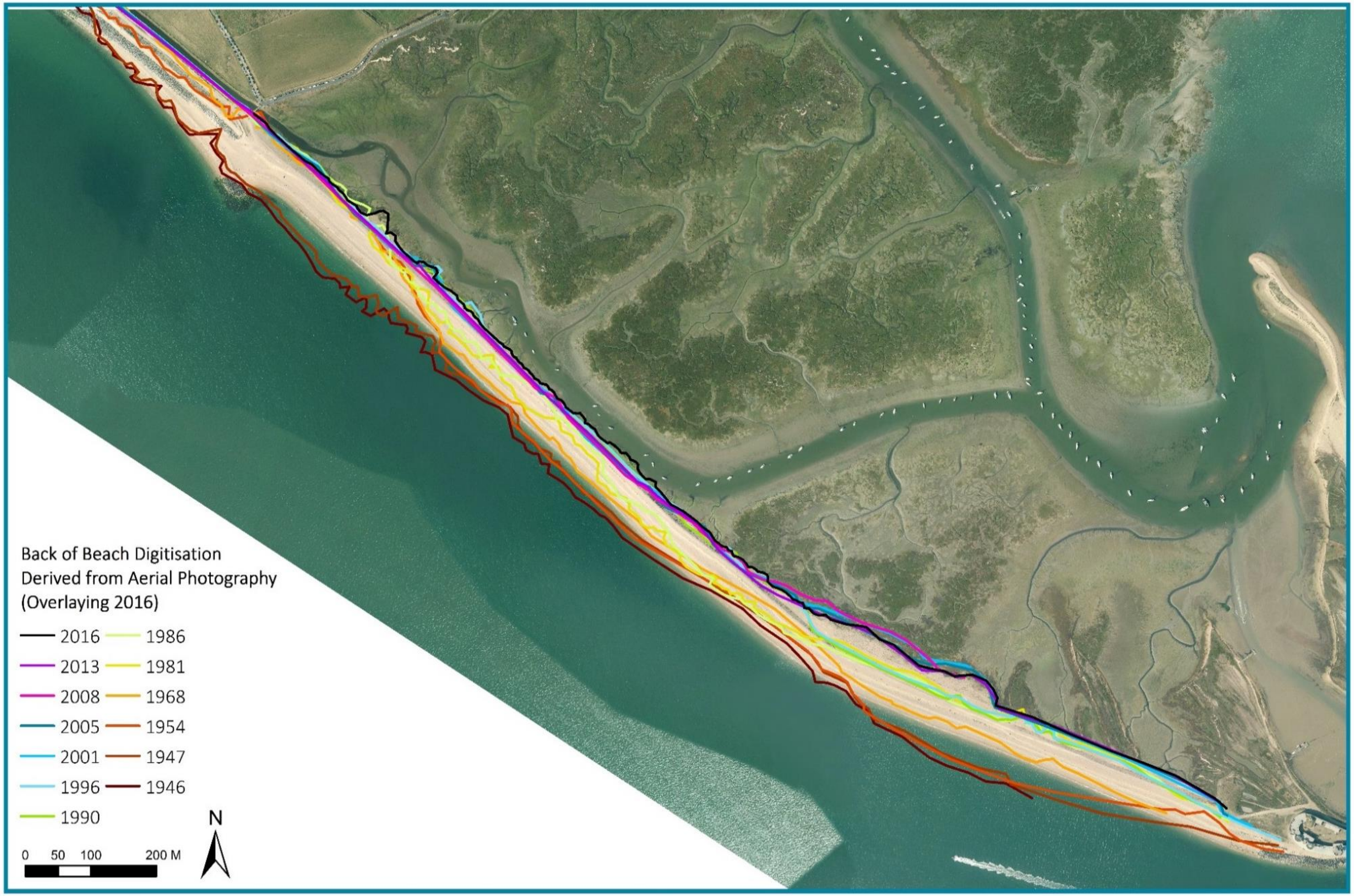

Figure 15b: Back of beach digitisation of aerial photography (1946 to 2016) Aerial photography (2016) (C) NFDC, courtesy of the Channel Coastal Observatory.

4.2 Sediment Analysis and Soil Classification

This section sets out the results from the sediment analysis, to demonstrate how an understanding of the stratigraphic properties of the back barrier sediments of Hurst Spit has been gained. Little work had been conducted previously to identify how the stratigraphy varies vertically and with distance along the spit. There was also uncertainty as to how the thickness of sediment varied spatially.

A total of eight cores were collected and analysed for their soil characteristics in the sediment analysis laboratory. Initially, the sediment samples were visually inspected for properties such as colour and composition. An indication of soil stiffness was also gained, in addition to general observations of sediment grain size. Logging of samples in the field had already enabled identification of any variation in sediment composition, so a high level of stratigraphy detail was maintained. Water content and organic content were calculated for each subsample. After removal of water and organic content by washing and re-drying, the proportions of sand, gravel and mud were also calculated to identify the dominant sediment type. The sediment analysis spreadsheets are available in Appendix C. Each table has a summary of observations.

The dominant sediment type was the dark-grey mud, which forms the Keyhaven mudflats and saltmarsh. Generally the water content of the mud is high (<50%) due to its low permeability and high water table, and organic content is low (<5%). Where there is gravel, the water content is low due to the high permeability of the sediment. The presence of peat is confined only towards and beyond the 'hinge-point', beneath current areas of saltmarsh vegetation. Peat is absent from any areas which are mudflat creek and therefore non-vegetated at the surface. Layers with peat presence see a marked increase in organic content (<65%), but the peat thickness rarely exceeds 1m, and is always in the bottom layer of the poorly consolidated material above the impenetrable gravel base.

A summary of the basic stratigraphy is presented in the form of a two dimensional plot in Figure 16. Figure 16 shows the dominant grain size for each core location, with locations presented with distance along the spit.

Through analysis of the core sediments, the core at HU9 was deemed unsuitable as a representative sample of back barrier sediments. This is mainly due to the requirement of the drill rig to have a supportive surface to set up on, at this location it needed to be the gravel overwash fan. The drill did not return any sediment for the first 2.1 metres, and then only produced samples predominately of sand and gravel. It appears that the drill had pushed the surface sediments down as it drilled through the sediment beneath, potentially resulting in false sediment samples. This was factored in to the second drilled core at HU14.5, where the drill rig was set up on a supportive surface but reached over to the poorly consolidated substrate.

Between HU10 and HU14, the storm overwashing from the 2013/14 storms had pushed gravel over the back barrier sediments, and it was difficult to find a suitable area to sample from which was free from gravel overwash material. This is why HU13 was located further landward from the others, from within the Mount Lake channel. Despite the channel location, this core still reached almost 3.5m below the surface, indicating that the channel had infilled with poorly consolidated material. It is also the reason for HU14 only achieving a core of 1.5m; the sediment had various layers of gravel, which made it difficult to penetrate down further.

Figure 16 demonstrates that the thickness of the poorly consolidated material increases with distance along Hurst Spit, from approximately 2.5m at HU8, to approximately 4m at HU14.5 and HU15. It is also interesting to see that the channel has also infilled with 3.5m of sediment at HU13. The level of the base of the poorly consolidated material also appears to vary. This base level is inferred from the maximum depth of the core reached, which was not limited by the equipment's depth range but rather by the impenetrable gravel base layer. The drill rig at HU14.5 was only able to penetrate a 0.2m into this gravel and yielded sub angular 'plateau' gravel.

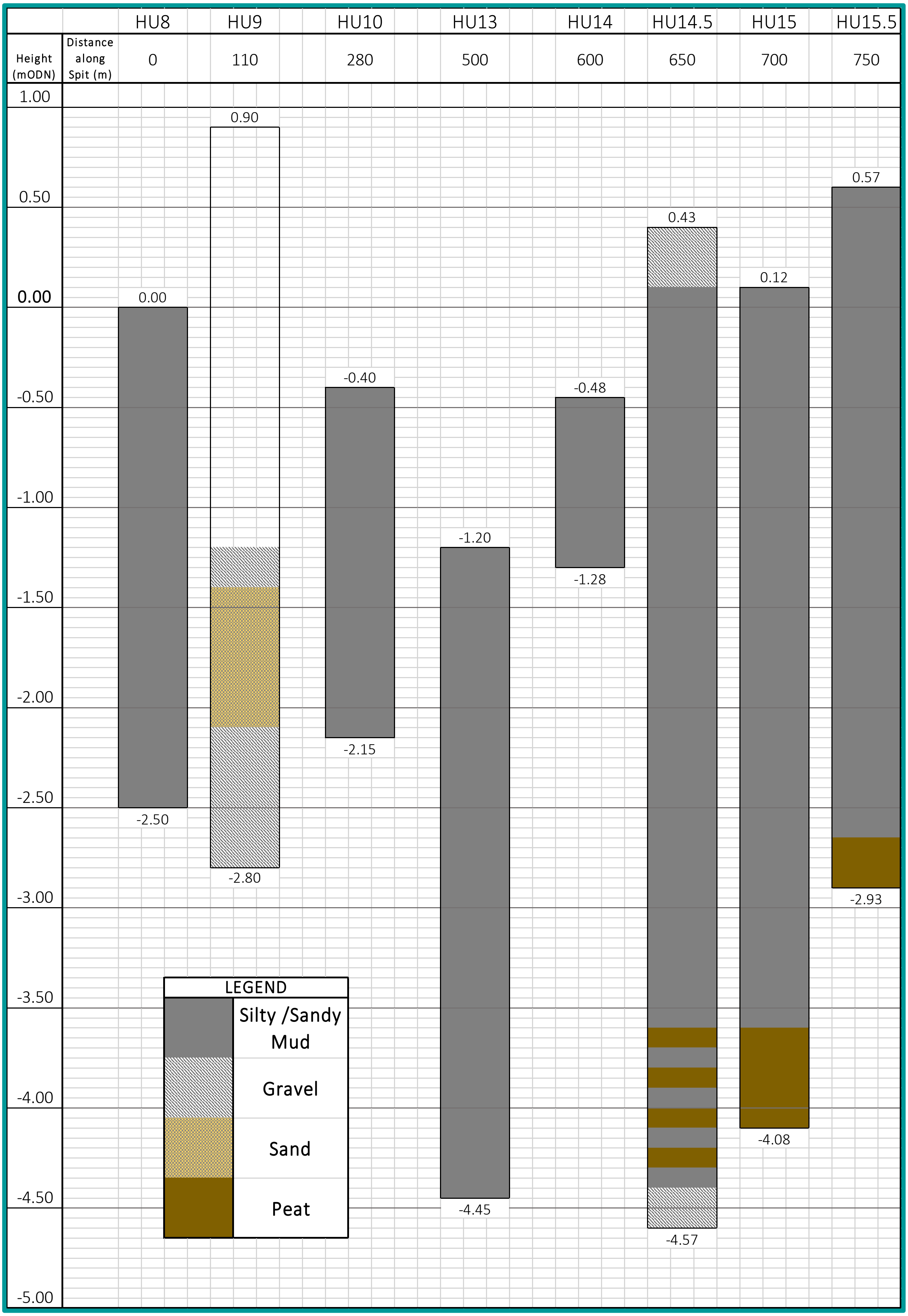

Figure 16: Two-dimensional plot to demonstrate dominant sediment composition, with depth and distance along Hurst Spit.

4.3 Geotechnical Analysis

This section sets out the results from the geotechnical (oedometer) analysis, to demonstrate how an understanding of the geotechnical properties of the poorly consolidated back barrier sediments of Hurst Spit has been gained. This section only refers to the poorly consolidated material. The oedometer test can provide information on sample permeability and stiffness, the magnitude of consolidation under load and the duration for 90% consolidation to occur. Samples from the surface (HU8) and at -0.6m, -1.6m and -2.6m below the surface (HU14.5) were tested with use of an oedometer. This was to ensure an understanding of how the geotechnical properties varied with depth.

The oedometer testing provided:

1) Plots of specific volume (v) against the natural logarithm of vertical effective stress ($\ln\sigma'_v$). These plots are used to investigate the behaviour of the soil under load at different depths and can indicate how soil stiffness varies under load. These results are presented in Section 4.3.1.

2) Plots of settlement (ρ) against the square root of time ($\sqrt{t}$) for each load increment (where load is added). These plots are then used to estimate the magnitude of consolidation that will be observed in the field when subjected to an increase in vertical load, and can indicate the theoretical time for 90% of consolidation to occur. These results are presented in Section 4.3.2

4.3.1 Changes in specific volume due to increased vertical stress.

The results of each oedometer test were used to calculate the change in specific volume with increased vertical stress. The following calculations were used for each loading stage:

$$v_f = 1 + w_f G_s$$

Where v_f is the final specific volume, w_f is the final moisture content and G_s is the relative density of the soil grains, roughly equal to 2.65. The water content of the sediment varied with depth as demonstrated by Table 4 below:

Depth (m) (location)	-0.15 (HU8)	-0.6 (HU14.5) LHS	-0.6 (HU14.5) RHS	-1.6 (HU14.5)	-2.6 (HU14.5)
W_f (%)	33	34	34	31	26

Table 4: Variation in final water content (w_f) with depth of oedometer sample.

Table 4 shows that final water content decreased with depth from 34% in the surface metre, to 26% at -2.6m below the surface. This is due to consolidation of the sediments at this depth, whereby the pore water has been expelled, and void space decreased.

To calculate specific volume:

$$v = h\left(\frac{v_f}{h_f}\right) = h\left(\frac{1 + w_{f G_s}}{h_f}\right)$$

Where h is the specimen height at the beginning of each load (mm), h_f is the final specimen height at the end of the test.

Figure 17 is the final plot which demonstrates the change in specific volume with increased vertical stress for each oedometer test. Figure 17 highlights that the sediment for the 50mm HU8 sample starts with a relatively higher specific volume (v=3.1). The increases in vertical stress exceeded that of the other tests, this resulted from an error in the early stages of calculation, which therefore overestimated the maximum stress required. The results from this specimen are interesting in that the loading stage is almost a straight line, indicating that there were no preconsolidation stresses for the surface material as is expected. The stiffness of the soil increases with increased load added. On unloading, the specimen is found to be much stiffer, not returning to the original state. When looking at the specimens for HU14.5, these are derived from much deeper in the soil matrix, and therefore originate from much lower specific volumes.

The deeper the specimen the lower the starting specific volume; v = 2.2-2.3 for -0.6m, (75mm, LHS and RHS HU14.5), 2.1 for -1.6m (76mm, HU14.5) and finally 1.95 for -2.6m (50mm, HU14.5). This is due to the decrease in final specimen water content with increased depth as demonstrated in Table 4. For all specimens the plastic deformation during the loading stages appears to exceed the elastic deformation of particles, so that the recovery during the unloading stage is not fully achieved.

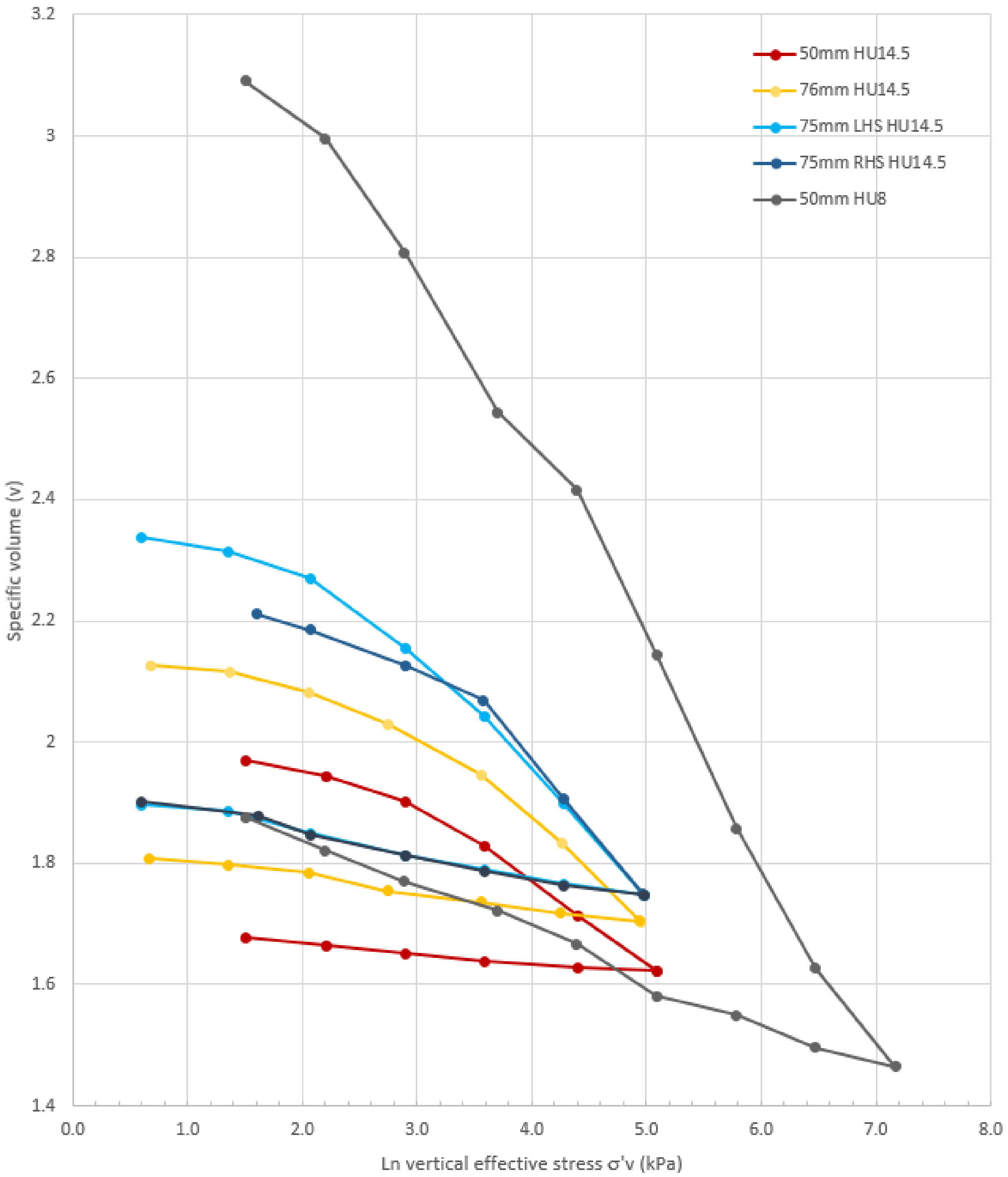

Figure 17: Changes in specific volume due to increased vertical effective stress for each oedometer specimen.

4.3.2 Calculating maximum consolidation.

For each loading stage, the settlement *p* (mm) was plotted against the square root of time $\sqrt{t}$ (min). These plots are available in Appendix D. Figure 18a demonstrates an example of a plot from HU14.5 (at 2.6m) depth, and shows settlement over time as the 1800g weight is added. The plot demonstrates that the settlement occurs in two phases: an initial phase where the majority of settlement occurs quickly, and then the slope flattens off to demonstrate the second phase where consolidation occurs at a slower rate. This trend can be seen in all plots. As the load increment is added there is a preliminary increase in pore water pressure, and pore water is expelled from the pores, resulting in rapid consolidation. As the pore water is expelled, the void ratio decreases and this results in soil matrix deformation. This marks the second phase of consolidation. Due to the low permeability of the soil, this rate of deformation is likely to occur over longer time periods.

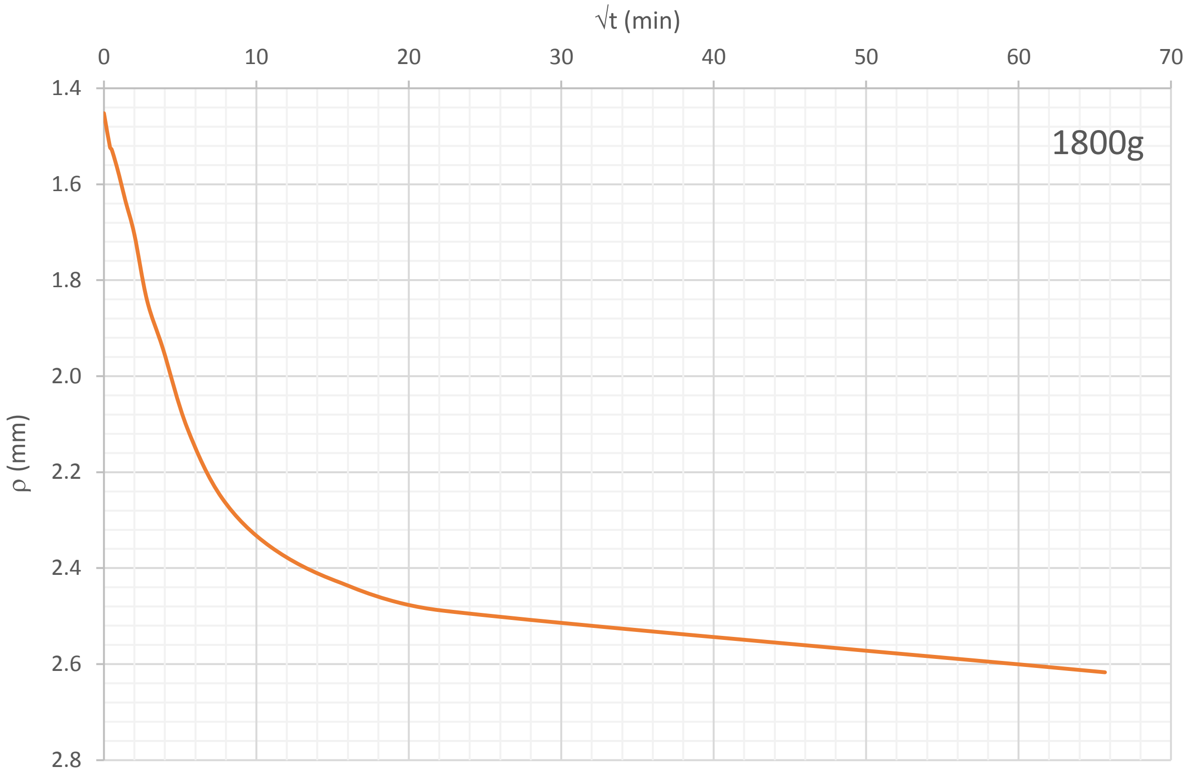

Figure 18a: Example plot of settlement over time for an oedometer specimen when the load is increased to 1800g.

These plots were then used to calculate the ultimate expected settlement (ρ_{ult}). Firstly the value of $\sqrt{t_x}$ is selected for each plot, using the method presented in Figure 18b.

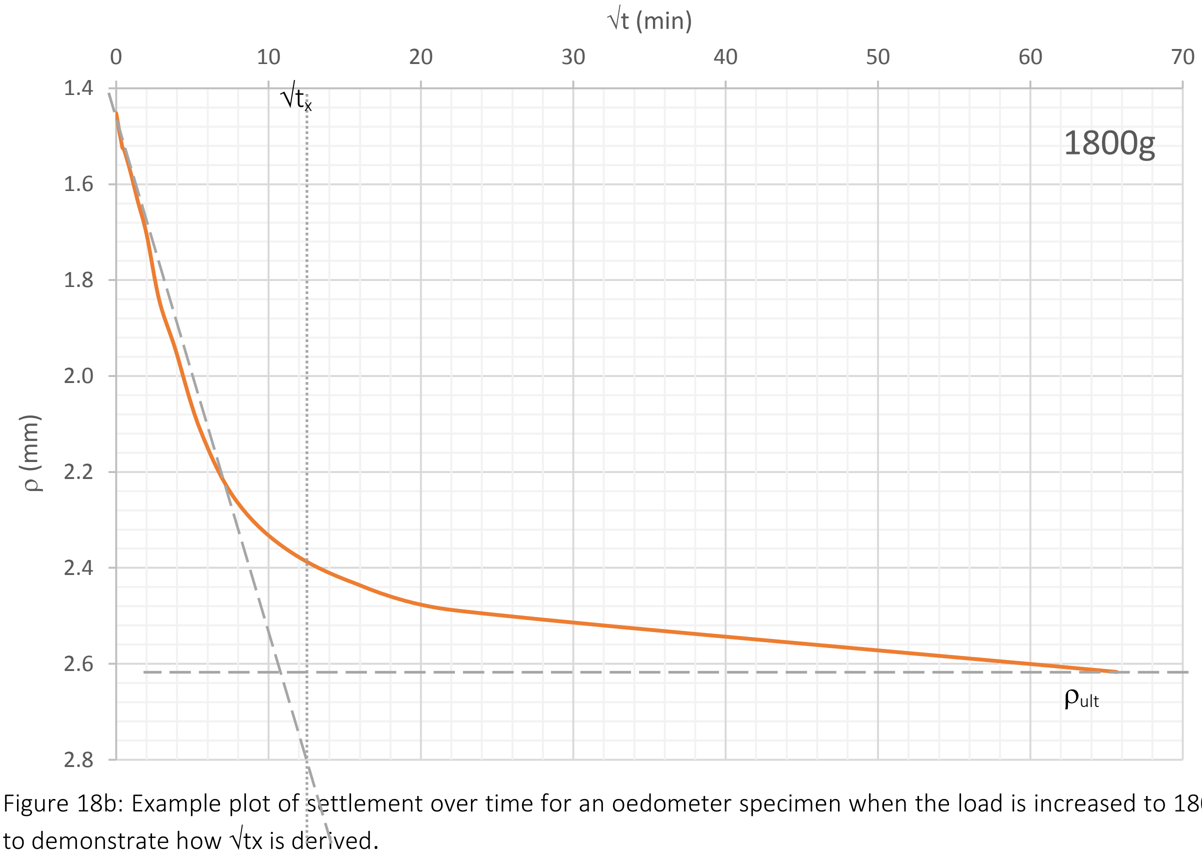

Figure 18b: Example plot of settlement over time for an oedometer specimen when the load is increased to 1800g, to demonstrate how √tx is derived.

To calculate the final settlement expected in the field, the following calculations were used:

$$t_x = \frac{3d^2}{4C_v} \; ; \; Cv = \frac{3d^2}{4t_x}$$

Where *d* is the drainage path length equal to the half the initial specimen height (h_0) (mm), and C_v is the consolidation coefficient for vertical compression due to vertical flow (mm^2/min). This is then converted to m^2/sec.

At the end of each load increment, the vertical strain ε_v is calculated:

$$\varepsilon_v = \frac{\rho_{max}}{h_0}$$

The one-dimensional modulus E'_0 is calculated:

$$E'_0 = \frac{\Delta\sigma'_v}{\varepsilon'_v}$$

The permeability *k* (m/s) is inferred:

$$k = \frac{\gamma_w C_v}{E'_0}$$

Finally, the real world equivalent maximum settlement ρ_{ult} (m) can then be calculated:

$$\rho_{ult} = \frac{2d\ \Delta\sigma_v}{E'_v}$$

Where *d* is the real world drainage path length equal to the half the thickness of the layer (m).

The time for 90% consolidation can also be calculated, this time using *d* as the real world drainage path length (m):

$$t_x = \frac{3d^2}{4C_v}$$

The results of these calculations are presented in Table 5. The thickness of poorly consolidated sediment varied between sites, and for Table 5, the thickness is set to 4.0m as this is the maximum thickness found during at site HU14.5. The results presented in Table 5 can be summarised in several key findings:

The first key finding from Table 5 is that the depth of the sample affects the overall magnitude of consolidation. This is shown in Figure 19a. For the expected vertical effective stress of a 6m thick gravel beach (100kPa) and substrate of 4m beneath, the range of expected maximum consolidation is between 0.58-1.08m. This is likely to be the maximum possible settlement for 6m of overburden. The range is caused by the depth of the sample taken used for the oedometer test. The deepest sample (HU14.5, -2.6m) results in the lowest magnitude of consolidation at 0.58m for 100kPa. This is because the sample has been previously consolidated by the load of mud above it, and it has therefore already undergoing an overburden and is likely to be consolidated making the sample stiffer. This is highlighted by the lack of consolidation during the first load stage, the soil has already undergone stresses at depth. The shallowest sample (HU8, -0.15m) results in the highest magnitude of consolidation at 1.08m for 100kPa, this is because these sediments have not undergone any previous consolidation under load, and are likely to have high levels of pore water and low stiffness. An average magnitude of consolidation expected for sediment 4m thick under an overburden of 100kPa is 0.83m.

Each depth of sediment sample represents a continuum of stiffness with depth. It is useful to see how the soil stiffness increases with depth, and therefore that surface samples are not wholly representative of the behaviour of the entire thickness of sediment. If surface sediments are used for calculations, an overestimation of total settlement may be derived.

	Load (g)	$\sigma'v$ (kPa)	$\sqrt{t_x}$ (min)	ρ_{max} (mm)	C_v (mm^2/min)	C_v (m^2/sec)	ε_v	E'_0 (kPa)	*k* (m/s)	ρ_{ult} (m)	t_x (sec)	t_x (years)
HU14.5 -2.6m	100	4.50	0	0.000	0	0	0.00	0	0	0	0	
	200	8.99	12	0.272	0.52	8.68×10^{-9}	0.01	661	1.29×10^{-10}	0.054	3.5×10^{8}	11
	400	17.99	10	0.703	0.75	1.25×10^{-8}	0.04	512	2.40×10^{-10}	0.141	2.4×10^{8}	8
	800	35.97	9	1.452	0.93	1.54×10^{-8}	0.07	496	3.06×10^{-10}	0.290	1.9×10^{8}	6
	1800	80.94	8	2.617	1.17	1.95×10^{-8}	0.13	618	3.10×10^{-10}	0.523	1.5×10^{8}	5
	3600	161.88	7	3.532	1.53	2.55×10^{-8}	0.18	916	2.73×10^{-10}	0.706	1.2×10^{8}	4
HU14.5 -1.6m	100	1.95	1	0.184	67.69	1.13×10^{-6}	0.01	201	5.51×10^{-8}	0.039	2.7×10^{6}	0.1
	200	3.89	9	0.285	0.84	1.39×10^{-8}	0.02	260	5.27×10^{-10}	0.060	2.2×10^{8}	7
	400	7.78	16	0.583	0.26	4.41×10^{-9}	0.03	254	1.70×10^{-10}	0.123	6.8×10^{8}	22
	800	15.57	10	1.040	0.68	1.13×10^{-8}	0.05	285	3.89×10^{-10}	0.219	2.7×10^{8}	8
	1800	35.03	10	1.796	0.68	1.13×10^{-8}	0.09	371	2.99×10^{-10}	0.378	2.7×10^{8}	8
	3600	70.06	9	2.779	0.84	1.39×10^{-8}	0.15	478	2.85×10^{-10}	0.585	2.2×10^{8}	7
	7200	140.13	9	3.923	0.84	1.39×10^{-8}	0.21	679	2.01×10^{-10}	0.826	2.2×10^{8}	7
HU14.5 LHS -0.6m	91	1.81	2.5	0.187	12.00	2.00×10^{-7}	0.01	194	1.01×10^{-8}	0.037	1.5×10^{7}	0.5
	193	3.85	2.5	0.389	12.00	2.00×10^{-7}	0.02	198	9.92×10^{-9}	0.078	1.5×10^{7}	0.5
	396	7.92	10	0.754	0.75	1.25×10^{-8}	0.04	210	5.84×10^{-10}	0.151	2.4×10^{8}	8
	906	18.11	11.5	1.732	0.57	9.45×10^{-9}	0.09	209	4.44×10^{-10}	0.346	3.2×10^{8}	10
	1790	35.76	12	2.686	0.52	8.68×10^{-9}	0.13	266	3.20×10^{-10}	0.537	3.5×10^{8}	11
	3590	71.75	11	3.896	0.62	1.03×10^{-8}	0.19	368	2.75×10^{-10}	0.779	2.9×10^{8}	9
	7192	143.73	9	5.176	0.96	1.54×10^{-8}	0.26	555	2.73×10^{-10}	1.035	1.9×10^{8}	6
HU14.5 RHS - 0.6m	249	4.98	1	0.564	75.00	1.25×10^{-6}	0.03	177	6.95×10^{-8}	0.113	2.4×10^{6}	0.1
	396	7.92	10	0.785	0.83	1.39×10^{-8}	0.04	202	9.73×10^{-10}	0.157	2.2×10^{8}	7
	906	18.11	8	1.302	1.17	1.95×10^{-8}	0.07	278	6.89×10^{-10}	0.260	1.5×10^{8}	5
	1790	35.99	9	1.814	0.93	1.54×10^{-8}	0.09	394	3.84×10^{-10}	0.363	1.9×10^{8}	6
	3590	71.75	14	3.234	0.38	6.38×10^{-9}	0.16	444	1.41×10^{-10}	0.647	4.7×10^{8}	15
	7192	143.73	10	4.617	0.75	1.25×10^{-8}	0.23	623	1.97×10^{-10}	0.923	2.4×10^{8}	8
HU8 -0.15m	100	4.50	17	0.856	0.26	4.32×10^{-9}	0.04	105	4.04×10^{-10}	0.171	6.9×10^{8}	22
	200	9.00	20	1.447	0.19	3.13×10^{-9}	0.07	124	2.41×10^{-10}	0.289	9.6×10^{8}	30
	400	17.99	22	2.615	0.15	2.58×10^{-9}	0.13	138	1.84×10^{-10}	0.523	1.2×10^{9}	37
	900	40.48	16	4.249	0.29	4.88×10^{-9}	0.21	191	2.51×10^{-10}	0.850	6.1×10^{8}	20
	1800	80.96	10	5.034	0.75	1.25×10^{-8}	0.25	322	3.81×10^{-10}	1.007	2.4×10^{8}	8
	3600	161.92	11	6.720	0.62	1.03×10^{-8}	0.34	482	2.10×10^{-10}	1.344	2.9×10^{8}	9
	7200	323.83	12	8.496	0.52	8.68×10^{-9}	0.42	762	1.12×10^{-10}	1.699	3.5×10^{8}	11
	14400	647.67	10	9.924	0.75	1.25×10^{-8}	0.50	1305	9.40×10^{-11}	1.985	2.4×10^{8}	8
	28800	1295.34	12	10.926	0.52	8.68×10^{-9}	0.55	2371	3.59×10^{-11}	2.185	3.5×10^{8}	11

Table 5: Parameters derived from the plots of consolidation with time, used to calculate maximum expected settlement, and time for settlement to occur.

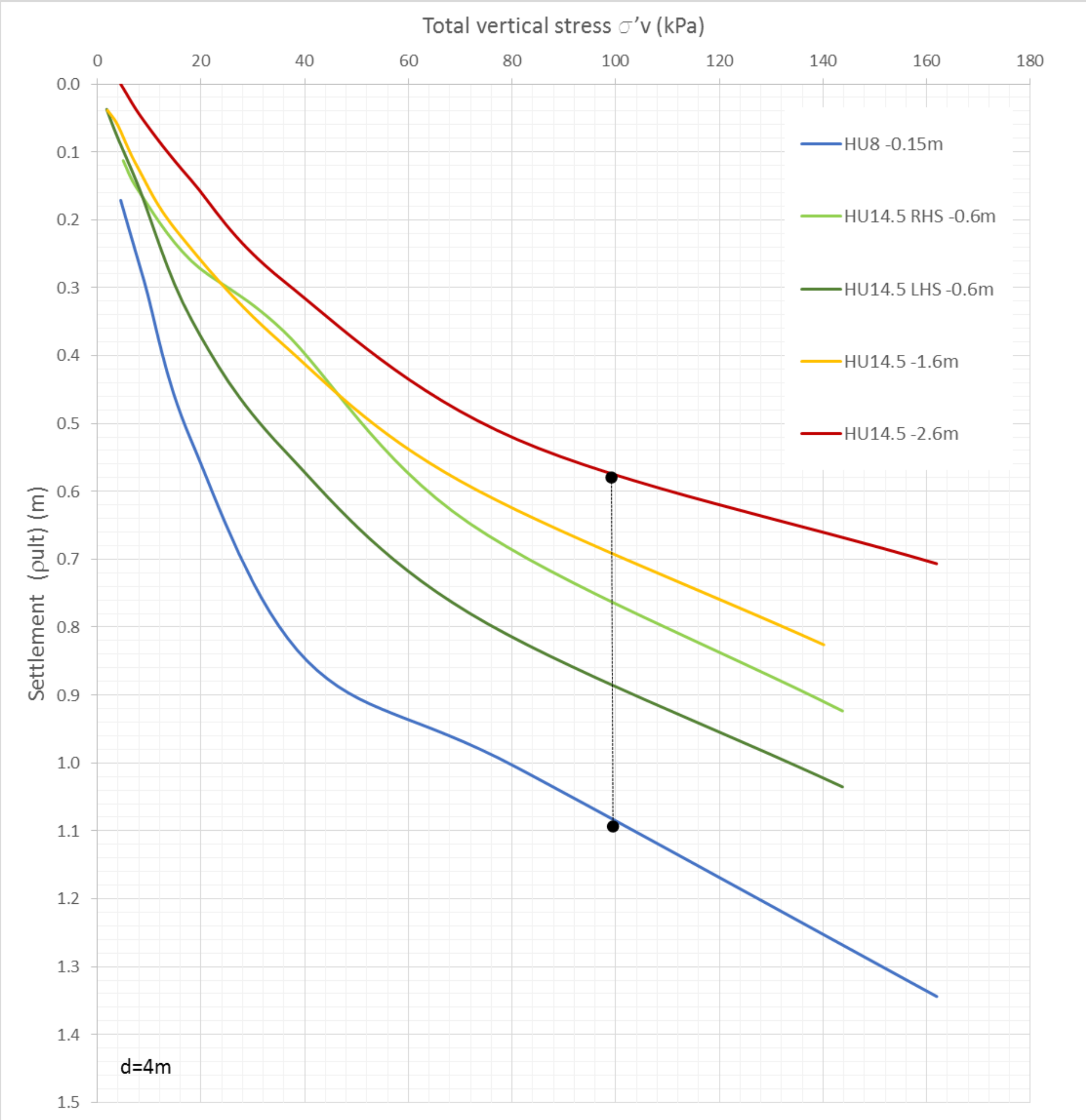

Figure 19a: Variation in maximum settlement expected due to increases in total vertical stress, for each oedometer sample (sediment thickness is 4m).

The second key finding is that permeability generally decreases with increased vertical stress. This is because the sediment is becoming more consolidated. The exceptions are found for the deeper sediment samples, where the sediment is already pre-consolidated under the overburden of mud above it, and therefore the permeability is already low.

The third key finding is that the time for 90% consolidation is high for all cases. This is interesting to see that consolidation can occur for decades after the load has been added. The surface sample at HU8 (0.15m) appears to take the longest duration to settle, even at low vertical stresses and this is due to the increased pore space which is full of pore water. All of this pore water must bleed out of the soil matrix to reach an equilibrium with the total vertical stress before the maximum consolidation occurs. Use of the surface sediments to predict overall settlement may result in an overestimation of time taken for full settlement to occur. The long timescales over which the consolidation occurs may influence the increments of loading, if the load is to be placed as such during a recharge. Loading in increments has the benefits of allowing some consolidation to occur between loads, and this will reduce the risk of slip failure of the sediments beneath the load, if the load is too high for one increment. Adding in increments and allowing consolidation to occur will increase the substrate stiffness and shear strength ready for the next load, and decrease the void ratio. Total consolidation between loads takes long periods and this is a negative aspect, however must not be ignored as slip failure of the substrate will result in lateral displacement of the load, and a loss in volume of recharge material that will need to be replaced.

Calculations were then made to understand the impact of the thickness of the poorly consolidated material on the overall magnitude of consolidation. Figure 19b demonstrates that the overall magnitude of final settlement is reduced, as the thickness of the substrate is reduced. For the expected vertical effective stress of a 6m thick gravel beach (100kPa) and substrate of 2.5m beneath (similar to at HU8), the range of expected maximum consolidation is between 0.36-0.68m. This is much lower than the range of 0.58-1.08m for 4m thickness of sediment. These findings highlight the need to understand the thickness of the poorly consolidated materials when predicting consolidation. A lack of understanding can result in inaccurate predictions of maximum consolidation. The time taken for maximum consolidation is also reduced because there is less sediment undergoing consolidation.

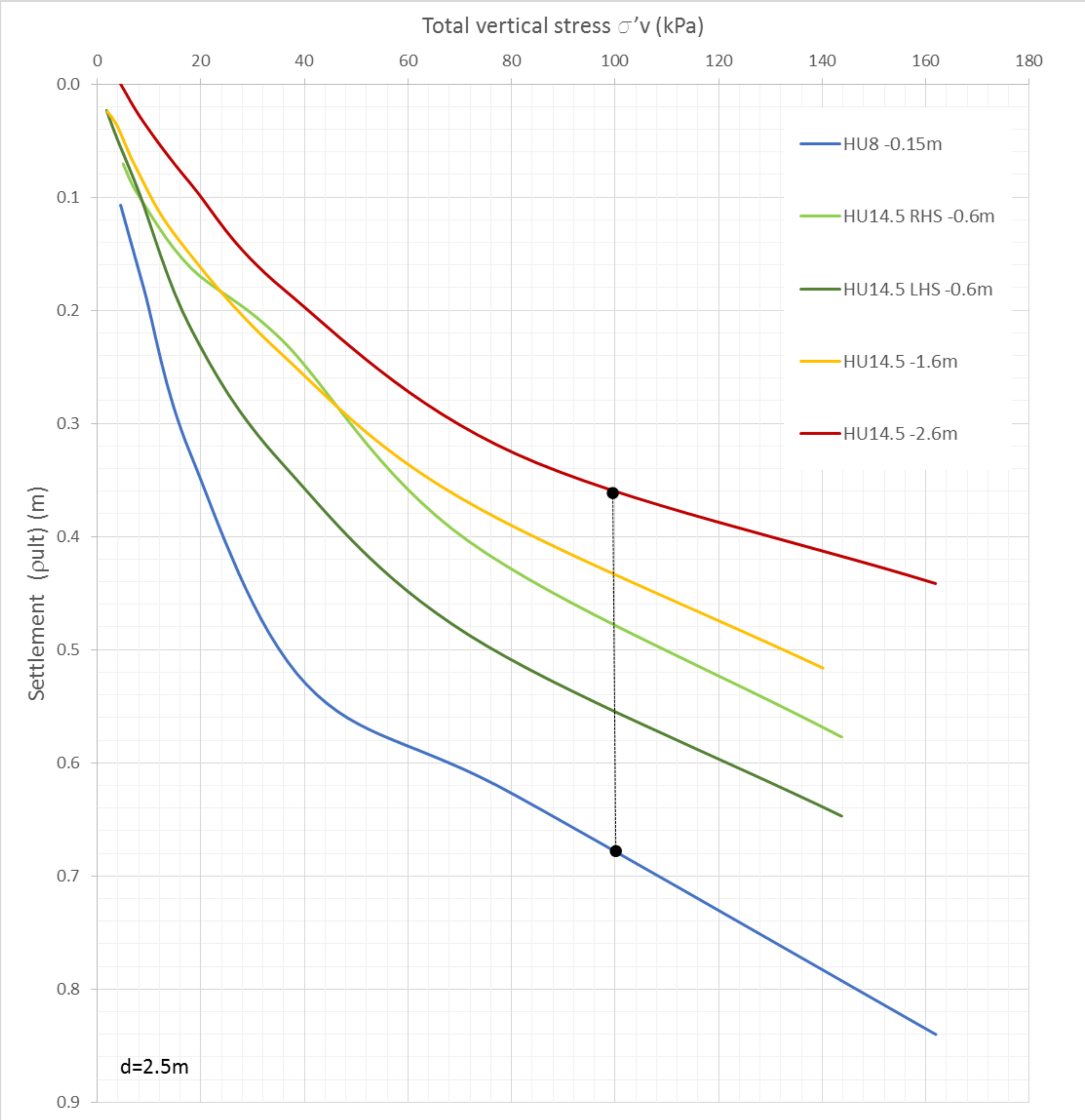

Figure 19b: Variation in maximum settlement expected due to increases in total vertical stress, for each oedometer sample (sediment thickness is 2.5m).

4.3.3 Calculating maximum consolidation.

This section aims to estimate potential consolidation of the back barrier sediments at Hurst Spit in the event of a future recharge. Three locations were selected based on the results from the sediment analysis. These locations were HU8, HU13 and HU15. These will be used to gauge the relative vulnerability to consolidation due to addition of beach recharge material. The beach profiles are shown in red, with the profile of additional recharge material in blue in Figures 20a, b and c. By using information of substrate thickness from the adjacent core, beach height above this substrate and the resultant vertical stress that will be imposed on the substrate, a calculation of subsidence due to consolidation is made. This consolidation is presented as a range (based on the oedometer findings) and then an average magnitude.

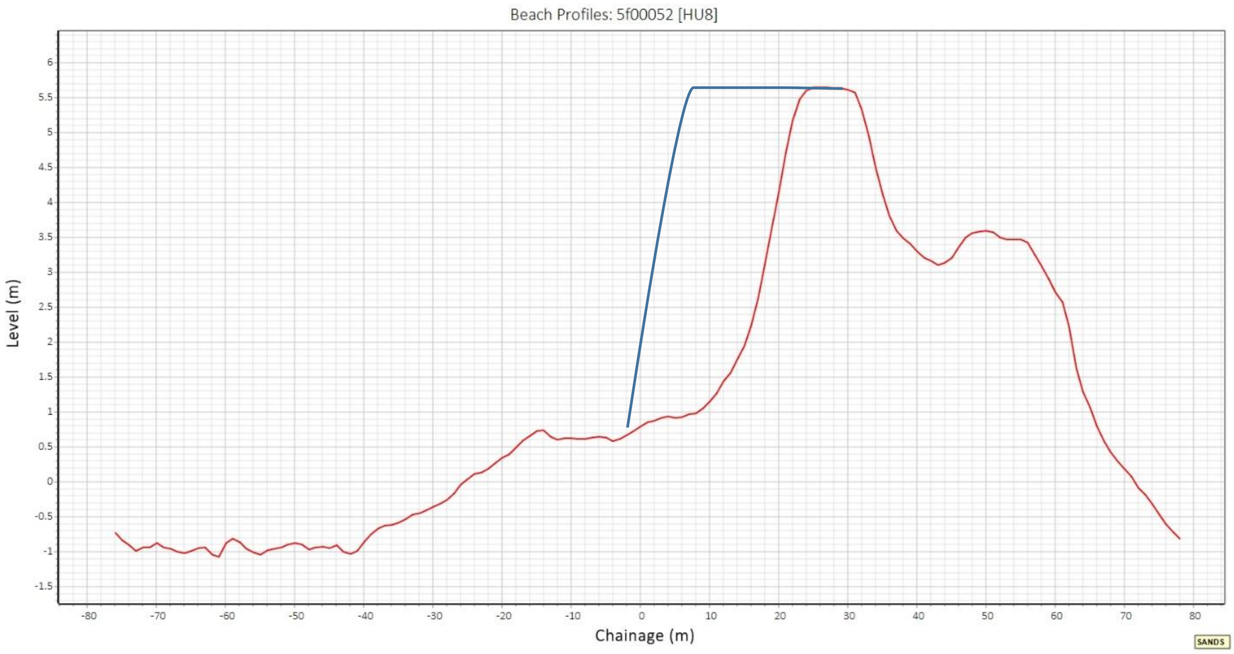

Figure 20a: Beach profile 5f00052 (HU8) (red) with potential profile of new recharge added in blue).

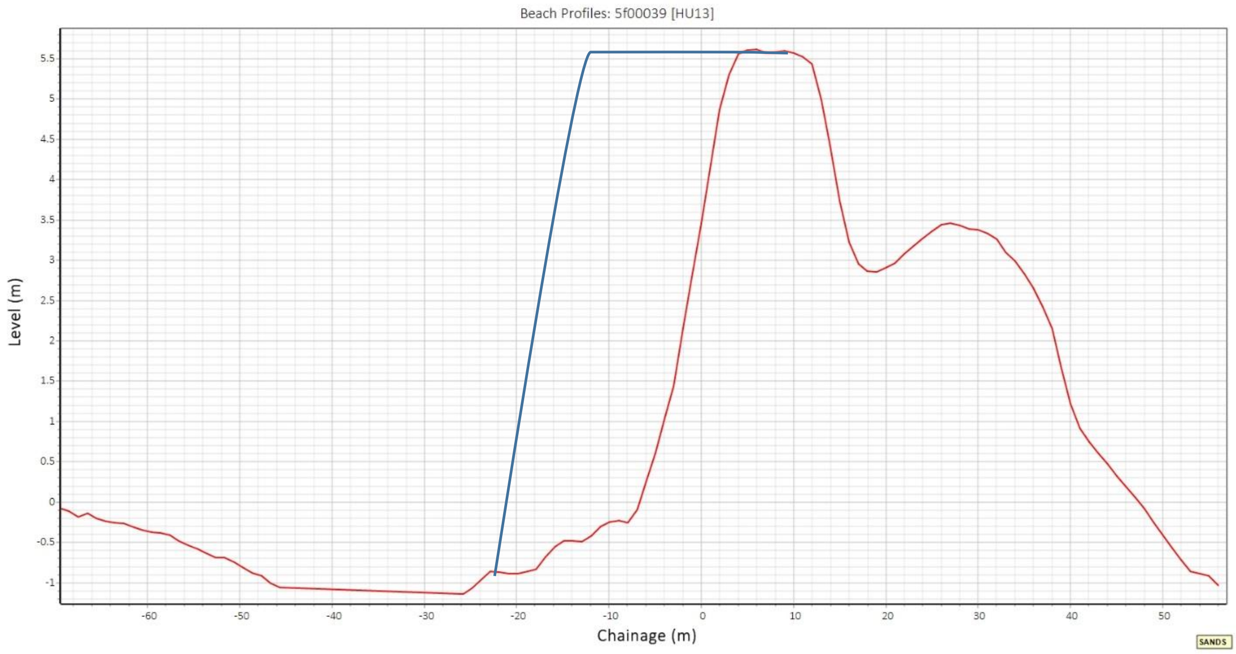

Figure 20b: Beach profile 5f00039 (HU13) (red) with potential profile of new recharge added in blue).

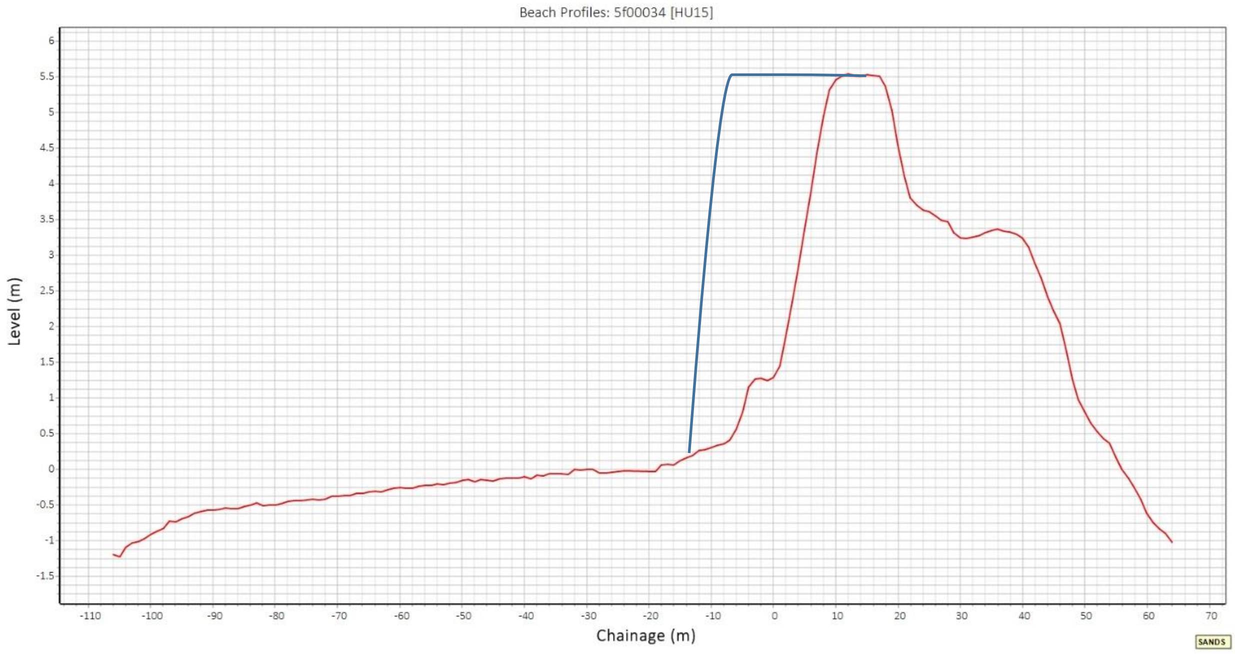

Figure 20c: Beach profile 5f00034 (HU15) (red) with potential profile of new recharge added in blue).

Table 6 presents the results of this exercise to determine which areas are more vulnerable to consolidation. Results show that consolidation magnitude is related to the thickness of the substrate, with the highest average consolidation predicted for HU15 where the substrate has the greatest thickness. This is closely followed by HU13, where the sediment thickness is slightly less than at HU15 (3.5m rather than 4.0m), but due to the increased height of beach required to gain a 5.6m crest, the load is higher, resulting in relatively high average consolidation. The least consolidation is expected at HU8 of the three locations. This is because the substrate thickness is reduced. The overburden required to reach a widened crest at 5.6m is also less than is required for the other two locations. Expected consolidation magnitude appears to increase with increased substrate thickness, which increases with distance along Hurst Spit.

Location	Thickness of substrate (m)	Height of beach (m)	Equivalent load (kPa)	Consolidation range (m)	Average consolidation (m)
HU8	2.5 (0 to -2.5m)	5.6m (0 to 5.6m)	90	0.34-0.64	0.49
HU13	3.5 (-1.0 to -4.5	6.6 (-1 to 5.6m)	106	0.52-1.00	0.76
HU15	4.0 (0 to -4.0)	5.5 (0 to 5.5m)	88	0.56-1.06	0.81

Table 6. Average maximum consolidation predicted for locations at Hurst Spit due to recharge.

4.4 Summary

This section aims to summarise the findings of the results section.

The historical shoreline analysis confirmed that Hurst Spit has been migrating landwards over the last century. Landward migration of 100 metres occurred during the last 60 years, with rates dependent on location. An average rate of 1.7m per year was calculated. Areas between the proximal end of the spit and the 'hinge-point' appear to have receded the most.

The acquisition of core samples of back barrier sediments in the lee of Hurst Spit have confirmed that the substrate over which the barrier has migrated is composed of poorly consolidated marine muds. The thickness of this substrate does vary, but is a maximum of 4m. The thickness appears to increase with distance along Hurst Spit. Peat is only located beneath current and previously vegetated saltmarsh east of HU14.5, and not beneath channel sediments. The peat layer usually forms the base layer of the substrate and does not exceed 1m in thickness. The Mount Lake channel has infilled with poorly consolidated material by 3.5m. As the barrier has migrated landward it has encroached on Mount Lake, and the next recharge may involve barrier sediments placed onto the channel banks.

The geotechnical analysis provided information on the stiffness of the substrate material and how it increases with depth. The substrate material is poorly consolidated, with a high water content and low permeability. This results in high potential for consolidation under load, however the duration of maximum consolidation may take years. It is clear that the thickness of poorly consolidated material affects the maximum potential consolidation magnitude, with greater thickness resulting in increased consolidation. This has implications for Hurst Spit, where the poorly consolidated sediment thickness varies. Implications of loading onto these sediments was explored, and highlighted that areas adjacent to HU13 and HU15 were more vulnerable to consolidation. Areas at the 'hinge point' could potentially consolidate by 0.8m under the future recharge, due to the high thickness of poorly consolidated materials of the back barrier sediments, and the presence of peat.

5. Discussion

The purpose of this section is to demonstrate and discuss how the objectives of this thesis have been met. These objectives were designed to support the overarching aim of this thesis, to investigate the stratigraphic and geotechnical properties of the back barrier sediments at Hurst Spit as an important example of a migrating gravel barrier.

To meet this aim, a campaign of sediment sampling was conducted through use of coring. These samples were found to be representative of the back barrier sediments at Hurst Spit. Little quantitive information existed to detail the thickness, composition and stratigraphy of the back barrier sediments, and these properties had previously been assumed. The sediment analysis of the core material provided a new insight into the thickness, dominant sediment type, and location of peat. The first objective “to conduct representative sediment sampling of the back barrier sediments at Hurst Spit, using coring equipment” has been successfully met.

Through analysis of the core sediments in the sediment analysis laboratory, the physical properties of the sediment were investigated. Information on the water and organic content were conducted and concluded that the water content was generally high (<60%) but generally decreased with depth, and that organic content was generally low apart from for peat layers. A basic particle size analysis distinguished the proportions of mud, sand and gravel to gauge the dominant sediment type. This analysis confirmed that the dominant sediment type was dark grey marine mud. The thickness of this mud varied spatially, generally increasing with distance along Hurst Spit. The thickness appeared to range from 2.5-4.0m, with some cores reaching peat deeper than -4.0mOD. Peat layers were confined to areas which were vegetated with saltmarsh at the surface, at the ‘hinge-point’ of Hurst Spit. The peat layer was towards the base of the substrate layer, beneath the mud and above the gravel base level. The thickness of the peat layer was <1m. The geotechnical properties of the sediment were explored using oedometer equipment. Undisturbed specimens acquired from drilled cores were prepared and tested under a range of loads which were equivalent to the overburden of a future beach recharge. Specimens from a range of depths below the surface were tested to establish whether stiffness varied with depth. It was clear that stiffness increased with depth, and that surface materials were not representative of the total sediment thickness, and would result in overestimation of consolidation. Investigations of the impact of substrate thickness on maximum expected consolidation revealed that thicker substrates resulted in higher magnitudes of consolidation. Therefore an understanding of the thickness is required to predict consolidation. The substrate permeability was generally high, but increased further with depth. Use of specimens from a range of depths and from two locations enabled an understanding of the geotechnical properties of the back barrier sediments to be understood. The methods selected ensured that the second objective to "establish the physical and geotechnical properties of the sediment" was met.

It is clear that Hurst Spit has migrated landwards during the last century in response to occasional extreme storm events. This landward migration is due to increase in the future as the current design and cross sectional profile is not likely to withstand overtopping from extreme storm events in the future. A replenishment is proposed in the future to provide an adequate standard of protection, however by increasing the crest width, material is likely to be placed on the back slope. This will increase the overburden for the poorly consolidated back barrier substrate, causing it to consolidate. Due to the varied thickness of back barrier sediments, the magnitude of consolidation is predicted to vary. Areas around the 'hinge point' are found to be most vulnerable to consolidation, due to the higher thickness of poorly consolidated material over which material will be placed. The area of barrier backing onto Mount Lake is also considered to be vulnerable to consolidation due to the surprisingly deep thickness of poorly consolidated material which has infilled the channel. The future management of Hurst Spit is likely to involve maintenance of the barrier as a flood defence, and the management must be sustainable. If the maintenance requires landward extension of the back barrier over poorly consolidated materials then more detailed ground investigations will provide further detailed information on the substrate thickness for predictions of consolidation to be made. Recommendations to load in increments are again highlighted due to the risk of slip failure, but the long duration for maximum settlement to occur may discourage this. Observations from the 1996 replenishment are of relevance as slip failure occurred when the load was not added in increments. Results from the settlement beacons observed that the area adjacent to the 'hinge point' underwent the greatest consolidation after the last recharge, and this supports the findings of this thesis. Results from the sediment and geotechnical analysis have informed predictions of settlement due to a future replenishment, and the spatial variation in consolidation vulnerability. Therefore, the third objective "to explore the implications of the results on the management of Hurst Spit, including a proposed replenishment" has been met.

Barrier beaches migrate landwards in response to a variety of forcing factors, however where this migration occurs over poorly consolidated materials is important consideration for coastal managers. Barriers often protect large areas of low-lying land and important assets, and coastal managers aim to maintain this level of protection. If the crest height lowers due to consolidation of the substrate beneath, then this level of protection reduces. Little work has been conducted to investigate the impact of consolidation beneath landward migrating barriers. In essence, this thesis has confirmed that an understanding of the thickness, stratigraphy and basic geotechnical properties of the back barrier sediments is required in order to predict consolidation under the overburden of a barrier beach. This thesis has also confirmed that assumptions of the thickness, stratigraphy and basic geotechnical properties may over or under estimate the magnitude of consolidation. Where a 'hold the line' management policy may prove unsustainable in the future, 'managed retreat' could prove to be more favourable. Where material is likely to be placed onto poorly consolidated back barrier sediments, then predictions of consolidation will be required. The methods used to support this thesis were relatively basic, and low cost, but the information provided has improved understanding of the stratigraphic and geotechnical properties of the back barrier sediments at Hurst Spit, and could be used at other locations where barriers are migrating landwards. It is hoped that this thesis will promote similar studies at other locations. Through exploring the wider implications of the findings of this thesis for coastal management, the final objective "to discuss the wider implications of the results of this thesis for coastal management of barrier beaches" is met.

6. Conclusions

6.1 Key Findings

This section will summarise the key findings of this thesis. In the first instance, this thesis has explored an interesting case study of a gravel barrier undergoing landward migration over poorly consolidated substrates, which are vulnerable to consolidation. In the case of Hurst Spit, the barrier thickness exceeds the thickness of the poorly consolidated substrate beneath it, and as such the relative subsidence is high.

A variety of techniques have been explored to identify the most suitable technique for core extraction, and the methods used were successful in providing samples which gave a better understanding of the poorly consolidated materials, over which Hurst Spit will migrate in the future. The analytical methods used for sediment and geotechnical analysis were appropriate for the time available, and provided suitable information to enable the main aim and objectives of this thesis to be met.

There is a clear spatial variability in poorly consolidated substrate thickness, and the coring confirmed that assumptions of sediment thickness may over or underestimate the actual thickness. The presence of peat within the stratigraphy also cannot be assumed. The presence of peat appears to be associated with areas of saltmarsh which are currently, or were previously vegetated. Use of surface materials to predict consolidation magnitude under load may result in an overestimated value, as the surface sediments are not representative of the total thickness.

Some sections of the barrier are more vulnerable to future consolidation than others, due to a variety of factors. Areas with a south-westerly aspect are more vulnerable to the highest wave impacts, and therefore more vulnerable to overwash of sediments and consequent landward migration of barrier sediments. Areas in the vicinity of the ‘hinge point’ are also more vulnerable as the sediment thickness is greater than at the proximal end of the spit. Furthermore, sediments in this area are underlain by peat which is highly vulnerable to consolidation under load. The section which is encroaching on the Mount Lake channel is also at a higher risk of consolidation, as the thickness of poorly consolidated substrate increases within the channel. These areas will require further investigation.

Recommendations for the future recharge at Hurst Spit are explored. It is evident that the recharge material should be added in stages, to minimise the risk of substrate slip failure. The methods used to support this thesis were relatively basic, and low cost, but the information provided has improved understanding of the stratigraphic and geotechnical properties of the back barrier sediments at Hurst Spit, to inform future plans for a potential recharge of Hurst Spit. The findings of this thesis could be applied to other locations where barriers are migrating landwards over poorly consolidated back barrier substrates, to gain an insight into the resultant consolidation and crest lowering. This is particularly important for locations where the barrier protects large areas of low-lying land and assets.

6.2 Recommendations for Future Research

In the first instance, the findings of this thesis could be used to inform the preliminary stages of the design for the next phase of recharge at Hurst Spit in the future. Areas at the most risk of consolidation are identified, and could be the subject of further research in the future. It would also be interesting to monitor settlement during the next phase of recharge to enable a comparison of predicted and observed consolidation magnitude over time.

The greater understanding of the substrate stratigraphy in the lee of Hurst Spit could be used to inform the next stage of the numerical model developed by Cooper (2015). This original model required assumptions of stratigraphy to be made. It is clear that the stratigraphy affects the magnitude of consolidation and that quantitive information needs to be factored in to the model to gain more reliable results.

Findings of this study could be applied to other local and national examples of barrier beaches, to inspire future research into the stratigraphic and geotechnical properties of the substrate beneath and in the lee of gravel barrier beaches undergoing landward migration. The subsidence of gravel barriers is clearly of importance to coastal managers, especially where the barrier provides a sheltering effect to areas of low-lying and populated land.

The sedimentary analysis was restricted due to time availability. A full particle size analysis was not possible for samples to detail the full particle size spectrum (including fine and coarse fractions). Small subsamples of the cores were retained and are available for further work, including proportion of silt and clay, and other sedimentary analysis. This would provide a detailed record of the sediment stratigraphy, and also inform an understanding of the drainage capability of the sediment. Both methods of core extraction (extendible Dutch gouge, and drill rig) proved efficient at yielding information on the thickness of poorly consolidated material, and provided samples suitable for particle size analysis. Further funding would allow use of the drill rig for more cores at a wider range of locations, and could even be used to establish the stratigraphy beneath Hurst Spit, currently unknown. A greater understanding of the proportions of clay and silt would be desirable, as these appeared to vary spatially, and will affect soil permeability and drainage. Furthermore, Hurst Spit appears to have great potential for further sedimentary and geotechnical studies.

Due to the coring methods available, the peat samples were of restricted diameter (<50mm) and therefore could not be tested for geotechnical properties. This study enabled an understanding of the presence, depth and thickness of the peat, however would benefit from further analysis to understand the impacts of peat layers on consolidation magnitude. Organisation of more substantial drills, to cover a larger area would yield undisturbed sediment samples for advanced sedimentary and geotechnical analysis. Undisturbed cores with a diameter >50mm are desired, and it is now known that the maximum substrate depth at Hurst Spit is -4.5mOD. The acquisition of suitable cores appears to be a limiting factor of studies in similar context to this thesis.

Calculations of the magnitude of consolidation in this thesis are based on one-dimensional consolidation theory. This is a simplification of real-world two-dimensional pore water flow, which is more complex to measure, predict and model. It is not impossible to measure but would provide an additional level of complexity to future studies of consolidation prediction beneath migrating barrier beaches. Use of triaxial equipment would also enhance the understanding of the mechanical properties of the substrate, and how it responds to addition of stress in perpendicular directions. This would also inform an understanding of the likelihood of failure of the sediment under load.

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8. Appendices

Appendix A. The Udden-Wentworth grain size scale (Wentworth, 1922).

Millimeters	µm	Phi (ϕ)	Wentworth size class	
		-20		
4096		-12	Boulder (-8 to -12ϕ)	
1024		-10		
256		-8		
64		-6	Pebble (-6 to -8ϕ)	
16		-4	Pebble (-2 to -6ϕ)	Gravel
4		-2		
3.36		-1.75		
2.83		-1.50	Gravel	
2.38		-1.25		
2.00		-1.00		
1.68		-0.75		
1.41		-0.50	Very coarse sand	
1.19		-0.25		
1.00		-0.00		
0.84		0.25		
0.71		0.50	Coarse sand	
0.59		0.75		
1/2 0.50	500	1.00		
0.42	420	1.25		
0.35	350	1.50	Medium sand	Sand
0.30	300	1.75		
1/4 0.25	250	2.00		
0.210	210	2.25		
0.177	177	2.50	Fine sand	
0.149	149	2.75		
1/8 0.125	125	3.00		
0.105	105	3.25		
0.088	88	3.50	Very fine sand	
0.074	74	3.75		
1/16 0.0625	63	4.00		
0.0530	53	4.25		
0.0440	44	4.50	Coarse silt	
0.0370	37	4.75		
1/32 0.0310	31	5		
1/64 0.0156	15.6	6	Medium silt	
1/128 0.0078	7.8	7	Fine silt	
1/256 0.0039	3.9	8	Very fine silt	Mud
0.0020	2.0	9		
0.00098	0.98	10		
0.00049	0.49	11	Clay	
0.00024	0.24	12		
0.00012	0.12	13		
0.00006	0.06	14		

Appendix B. 2016 Aerial photography image of Lymington Phase I and II breakwaters (courtesy of Channel Coastal Observatory, 2016).

Appendix C. Soil Description Spreadsheets

Type	Dutch gouge	Location	HU8	Date	09/04/2016

Depth (mOD) From	Depth (mOD) To	Colour	Stiffness	Description	Water content (%)	Organic content (%)	Gravel (%)	Sand (%)	Mud (%)
+0.00	-0.30	Dark grey	High	Silty MUD, grit/gravel, shell fragments	34	2	25	19	56
-0.30	-0.50	Dark grey	High	Silty MUD, grit/gravel	50	3	0	4	96
-0.50	-0.80	Dark grey	Moderate	Silty MUD, grit/gravel	43	1	29	4	67
-0.80	-1.00	Dark grey	Moderate	Silty MUD	54	3	2	2	96
-1.00	-1.30	Dark grey	High	Silty MUD, grit/gravel	53	1	2	2	96
-1.30	-1.50	Dark grey	High	Silty MUD	53	4	0	3	97
-1.50	-1.80	Dark grey	High	Silty MUD	47	2	0	13	86
-1.80	-2.00	Dark grey	High	Silty MUD, grit/gravel	29	0	11	38	51
-2.00	-2.30	Dark grey	Low	Silty MUD	42	2	5	22	73
-2.30	-2.50	Brown	Very High	Silty MUD	23	0	7	32	61

General observations:

Location was close to proximal end of Hurst Spit, 2.5m of moderate to very high stiff, silty mud samples extracted
Sand proportion increased at depth, some layers had gravel in.
Limit was impenetrable, gritty gravel at -2.5mOD
Mud has high water content, often exceeding 50% of sample
Mud has low organic content (<5%)
No layer of peat exists.

Type	Drill rig	Location	HU9	Date	04/05/2016

Depth (mOD)		Colour	Stiffness	Description	Water content (%)	Organic content (%)	Gravel (%)	Sand (%)	Mud (%)
From	To								
+0.90	-1.20	n/a	n/a	No recovery	n/a	n/a	n/a	n/a	n/a
-1.20	-1.40	Light orange/ grey	n/a	GRAVEL, fine to coarse	4	0	88	10	2
-1.40	-1.50	Light orange/ grey	n/a	SAND, gravel, clay/silt	13	4	70	14	3
-1.50	-1.60	Dark grey	n/a	SAND, gravel, clay/silt	7	1	84	15	1
-1.60	-1.70	Grey	n/a	SAND, gravel, clay/silt	8	1	85	12	3
-1.70	-1.90	Dark grey	n/a	SAND, gravel, clay/silt	6	1	86	10	4
-1.90	-2.10	Light orange/ grey	n/a	SAND, gravel, clay/silt	6	0	87	10	3
-2.10	-2.50	Dark grey	n/a	GRAVEL	7	1	92	5	3
-2.70	-2.70	Dark grey	n/a	GRAVEL	9	3	93	6	1
-2.70	-2.80	Dark grey	n/a	GRAVEL	7	1	93	6	1

General observations:

Location selected as it was necessary to set up drill rig on supportive ground. This was the first location of the drill rig, and set up on an overwash fan. The core did not recover any of the substrate material, but appeared to recover gravel material from the overwash Spit material- mostly gravel and sand. This is not representative of the back barrier beach sediments, which are generally poorly consolidated muds.

Water content was very low due to the high permeability of the gravel and sand particles (<15%)

Organic content was also very low (<5%)

Limit reached was impenetrable gravel. No peat layer exists.

Type	Dutch gouge	Location	HU10	Date	09/04/2016				
Depth (mOD) From	**To**	**Colour**	**Stiffness**	**Description**	**Water content (%)**	**Organic content (%)**	**Gravel (%)**	**Sand (%)**	**Mud (%)**
-0.40	-0.65	Dark grey/ brown	Moderate	MUD, silty/sandy, gravel	42	3	15	7	78
-0.65	-0.90	Dark grey/ brown	Moderate	MUD, silty/sandy, gravel	44	6	12	4	84
-0.90	-1.15	Dark grey/ brown	Moderate	MUD, silty/sandy, gravel	46	7	11	8	81
-1.15	-1.40	Dark grey/ brown	Moderate	MUD, silty/sandy	39	6	0	2	98
-1.40	-1.65	Dark grey/ brown	Moderate	MUD, silty/sandy	38	6	0	5	95
-1.65	-1.90	Dark grey/ brown	Very	MUD, silty/sandy, gravel	32	5	15	5	80
-1.90	-2.15	Dark grey/ brown	Very	MUD, silty/sandy	35	4	0	9	91

General Observations:

Location was selected to ensure retrieval of back barrier sediments. Dominant sediment type was mud.
The water content is generally high (~40%), and decreases with depth. Organic content is low.
Maximum depth reached was -2.15mOD. No peat layer exists

Type	Dutch gouge	Location	HU13	Date	10/04/2016

Depth (mOD)		Colour	Stiffness	Description	Water content (%)	Organic content (%)	Gravel (%)	Sand (%)	Mud (%)
From	To								
-1.20	-1.45	Brown	Moderate	MUD, silty/sandy, gravel	16	1	30	66	4
-1.45	-1.70	Grey	Moderate	MUD, silty/sandy, gravel, shell fragments	34	4	5	49	46
-1.70	-1.95	Dark brown	Moderate	MUD, silty/sandy, gravel	26	3	12	58	30
-1.95	-2.20	Dark grey	Moderate	MUD, silty/sandy, gravel	23	2	7	66	27
-2.20	-2.45	Dark grey	Moderate	MUD, silty/sandy, gravel	23	2	16	65	19
-2.45	-2.70	Dark grey	Moderate	MUD, silty/sandy, shell fragments	26	3	2	58	39
-2.70	-2.95	Dark grey	Moderate	MUD, silty/sandy, gravel	28	2	6	37	57
-2.95	-3.20	Dark grey/ brown	Moderate	MUD, silty/sandy, shell fragments	31	4	2	33	64
-3.20	-3.45	Dark grey/ brown	Moderate	MUD, silty/sandy, gravel	29	4	7	54	39
-3.45	-3.70	Dark grey/ brown	Moderate	MUD, silty/sandy, shell fragments	32	4	3	20	77
-3.70	-3.95	Dark grey/ brown	Moderate	MUD, silty/sandy, gravel	29	3	4	49	48
-3.95	-4.20	Dark grey/ brown	Moderate	MUD, silty/sandy, gravel	28	3	11	46	44
-4.20	-4.45	Dark grey/ brown	Moderate	MUD, silty/sandy	27	3	1	55	45

General observations:

Location was selected as difficult to find a location along this stretch that did not have gravel near the surface. Managed to walk out into the channel, and collect a core within the centre of the channel. Dominant sediment type was mostly mud, however proportions of sand increase towards the surface. This may be due to the location beneath the channel.
The water content is generally high (<30%), and decreases with depth. Organic content is low.
Maximum depth reached was -4.45mOD
No peat layer exists

Type	Dutch gouge	Location	HU14	Date	10/04/2016				
Depth (mOD)		Colour	Stiffness	Description	Water content (%)	Organic content (%)	Gravel (%)	Sand (%)	Mud (%)
From	To								
-0.5	-0.8	Dark grey/ brown	Moderate	MUD, silty/sandy, gravel	38	4	10	34	56
-0.8	-1.0	Dark grey/ brown	Moderate	MUD, silty/sandy, gravel	37	2	33	15	52
-1.0	-1.3	Brown	Moderate	MUD, silty/sandy, gravel	31	5	21	27	52

General observations:

Difficult to reach any deeper than -1.3m due to layers of coarse gravel in the substrate. Dominant sediment type was mud, but there were high proportions of sand and gravel also.
Water content is generally high (<40%), with low organic content.
No peat layer exists.

Type	Drill rig	Location	HU14.5	Date	04/05/2016

Depth (mOD) From	Depth (mOD) To	Colour	Stiffness	Description	Water content (%)	Organic content (%)	Gravel (%)	Sand (%)	Mud (%)
+0.40	+0.10	Dark grey	n/a	GRAVEL	5	2	79	18	3
+0.10	-0.20	Dark grey	Moderate	MUD, silty	35	3	0	25	75
-0.20	-0.60	Dark grey	Moderate	MUD, silty	31	2	0	30	70
-0.60	-1.10	Dark grey	Low	MUD, silty/sandy	31	3	0	30	70
-1.10	-1.40	Dark grey	Low	MUD, silty/sandy	16	1	62	2	36
-1.40	-1.60	Dark grey	Low	MUD, silty/sandy	26	2	0	5	95
-1.60	-2.05	Dark grey	Low	MUD, silty/sandy	28	2	20	40	40
-2.05	-2.60	Dark grey	Low	MUD, silty/sandy	18	1	0	83	17
-2.60	-3.60	Dark grey	Low	MUD, silty/sandy	23	1	1	74	25
-3.60	-3.80	Dark grey	Moderate	MUD, PEAT	33	15	0	25	75
-3.80	-3.90	Dark grey	Moderate	MUD, PEAT	40	6	0	13	87
-3.90	-4.00	Dark grey	Moderate	MUD, PEAT	44	7	0	50	50
-4.00	-4.40	Dark grey	Moderate	MUD, PEAT, shell fragments	47	15	0	30	70
-4.40	-4.45	Dark grey	High	MUD, silty/sandy	14	1	1	36	63
-4.45	-4.50	Dark grey	Low	MUD, silty/sandy, gravel	11	1	75	0	25
-4.50	-4.60	Dark grey	n/a	GRAVEL	8	0	52	38	10

General observations:

This location was selected for the drill rig. A maximum depth of -4.6mOD was reached, and penetrated the gravel base layer beneath the mud substrate. The substrate dominant sediment type was mud, with a high water content (<50%). Organic content was low apart from layers containing peat -3.6 to -4.4mOD. High proportions of sand for some layers are noted, especially between -2.05 and -3.6.

Type	Dutch gouge	Location	HU15	Date	07/05/2016

Depth (mOD)		Colour	Stiffness	Description	Water content (%)	Organic content (%)	Gravel (%)	Sand (%)	Mud (%)
From	To								
+0.10	-0.10	Dark grey/ brown	Moderate	MUD, silty/sandy, gravel	43	5	31	7	62
-0.10	-0.40	Dark grey/ brown	Moderate	MUD, silty/sandy	51	5	0	7	93
-0.40	-0.60	Dark grey	Moderate	MUD, silty/sandy	47	5	0	2	98
-0.60	0.90	Dark grey	Moderate	MUD, silty/sandy, shell fragments	46	5	0	3	97
-0.90	-1.40	Dark grey	Moderate	MUD, silty/sandy, gravel	46	6	0	0	100
-1.40	-1.60	Dark grey	Moderate	MUD, silty/sandy	37	4	0	8	92
-1.60	-1.90	Dark grey	Moderate	MUD, silty/sandy	36	4	0	16	84
-1.90	-2.20	Dark grey	Moderate	MUD, silty/sandy	35	3	0	24	76
-2.20	-2.40	Dark grey	Moderate	MUD, silty/sandy	28	3	0	41	59
-2.40	-2.60	Dark grey	Moderate	MUD, silty/sandy	25	2	0	52	48
-2.60	-3.00	Dark grey	Moderate	MUD, silty/sandy, shell fragments	30	2	0	38	62
-3.00	-3.10	Dark grey	Moderate	MUD, silty/sandy, gravel, shell fragments	22	1	7	8	84
-3.10	-3.40	Dark grey	Moderate	MUD, silty/sandy, shell fragments	33	2	0	23	77
-3.40	-3.60	Dark grey	Moderate	MUD, PEAT, silty/sandy	25	5	4	71	25
-3.60	-3.80	Dark grey/ brown	Moderate	MUD, PEAT, silty/sandy	46	8	1	27	73
-3.80	-4.00	Dark grey/ brown	Moderate	MUD, PEAT, silty/sandy	45	12	1	40	58
-4.00	-4.10	Dark grey	High	MUD, PEAT, silty/sandy	34	13	0	69	31

General observations:

This was a good location for use of the Dutch gouge- with -4.10mOD reached. The dominant sediment type is mud, with high water content (<50%). Organic content is low apart from for layers containing peat where it increases. Sand content for some layers is high.
Peat was only located between -3.4 and 4.10mOD. The limit was impenetrable gravel.

Type	Dutch gouge	Location	HU15.5	Date	07/05/2016

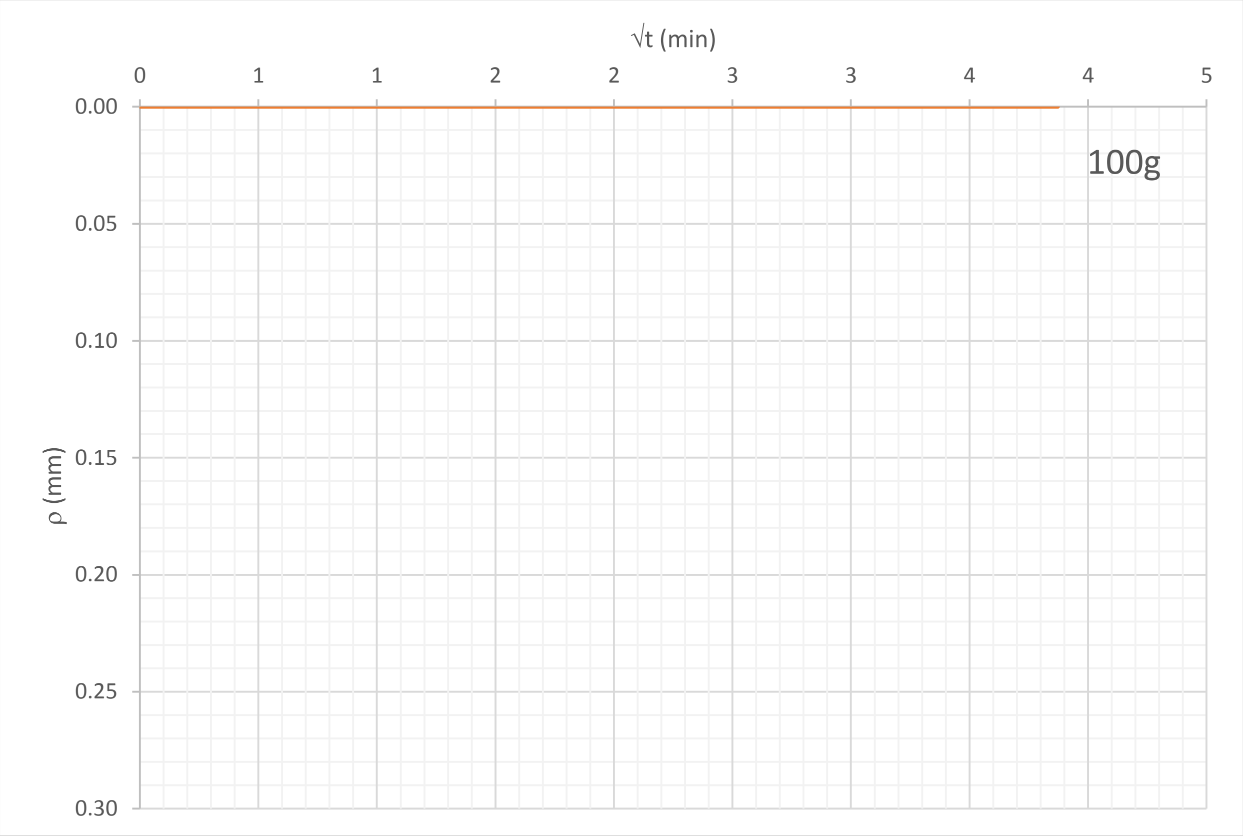
Depth (mOD) From	Depth (mOD) To	Colour	Stiffness	Description	Water content (%)	Organic content (%)	Gravel (%)	Sand (%)	Mud (%)
+0.60	+0.35	Dark grey/ brown	Moderate	MUD, silty/sandy, fibres, shell fragments	52	5	1	10	89
+0.35	+0.10	Dark grey/ brown	Moderate	MUD, silty/sandy, fibres	49	7	0	2	98
+0.10	-0.15	Brown/ orange	Moderate	MUD, silty/sandy, gravel, fibres	49	7	2	2	97
-0.15	-0.40	Brown	Moderate	MUD, silty/sandy, fibres	50	6	0	2	98
-0.40	-0.65	Dark grey/ brown	Moderate	MUD, silty/sandy, fibres	41	4	0	3	97
-0.65	-0.90	Dark grey/ brown	Moderate	MUD, silty/sandy, fibres	38	4	0	3	97
-0.90	-1.15	Dark grey	Moderate	MUD, silty/sandy	34	3	0	26	74
-1.15	-1.40	Dark grey	Moderate	MUD, silty/sandy, shell fragments	34	3	0	26	74
-1.40	-1.65	Dark grey	High	MUD, silty/sandy, shell fragments	26	3	0	58	42
-1.65	-1.90	Dark grey	High	MUD, silty/sandy, fibres, shell fragments	33	4	0	10	90
-1.90	-2.15	Dark grey	Moderate	MUD, silty/sandy, shell fragments	35	5	0	6	94
-2.15	-2.40	Dark grey	Moderate	MUD, silty/sandy	44	5	0	5	95
-2.40	-2.65	Dark grey/ brown	Moderate	MUD, silty/sandy, shell fragments	42	6	0	5	95
-2.65	-2.90	Dark grey/ brown	Low	MUD, PEAT, silty/sandy	64	65	0	0	100

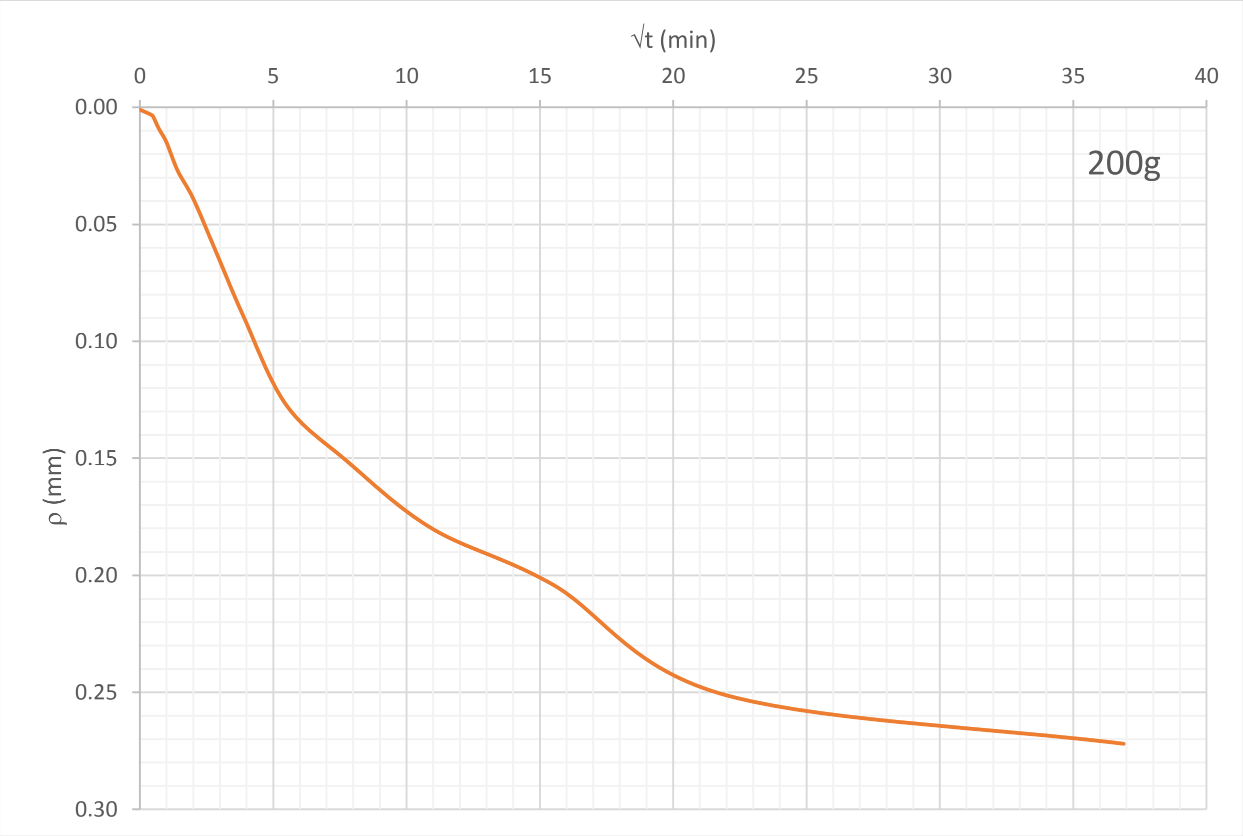
General observations:

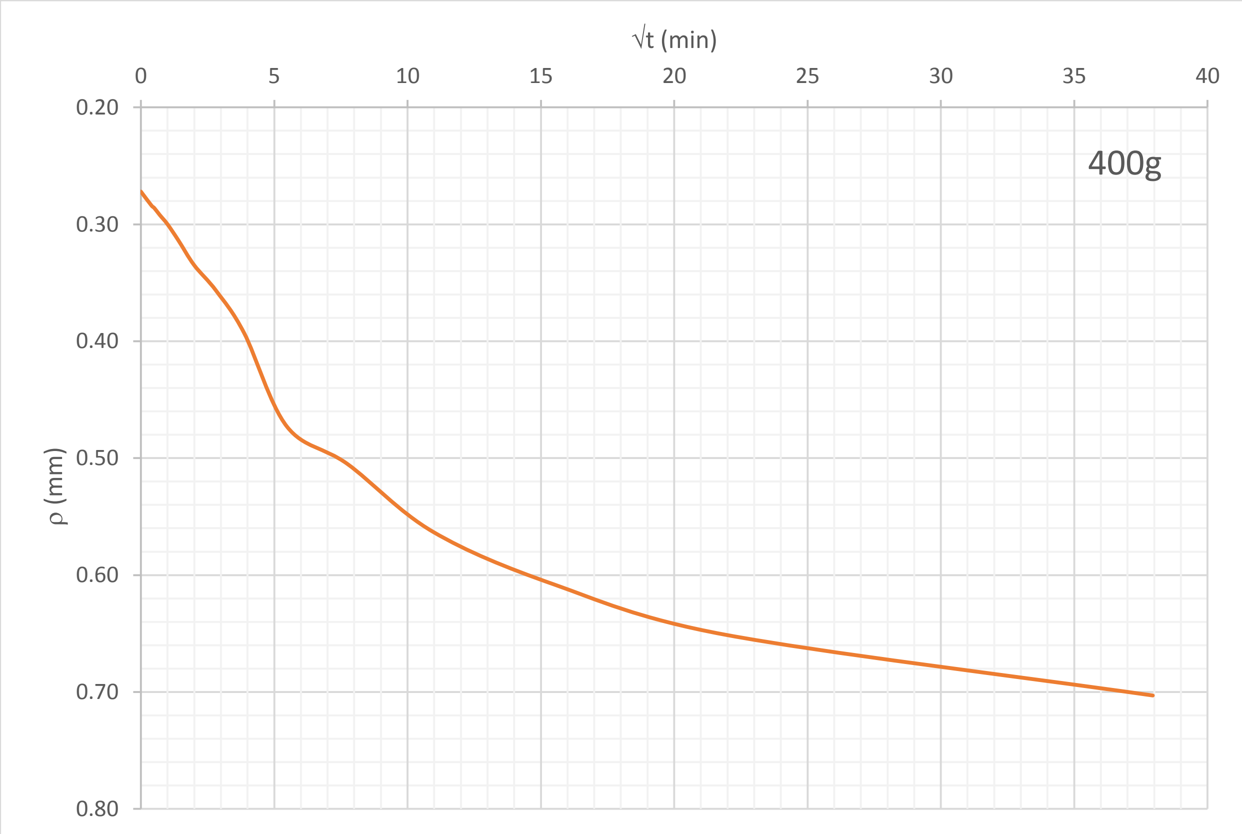
Dominant sediment type is moderately stiff, silty/sandy mud. Water content is generally high (40-65%). Peat only found at base layer with high organic content (65%). Sandy layer found -0.9 to -1.9m. Maximum depth reached was -2.9mOD

Appendix D. Plots of oedometer specimen consolidation over time, for each load step.

HU14.5, 50mm (-2.6m).



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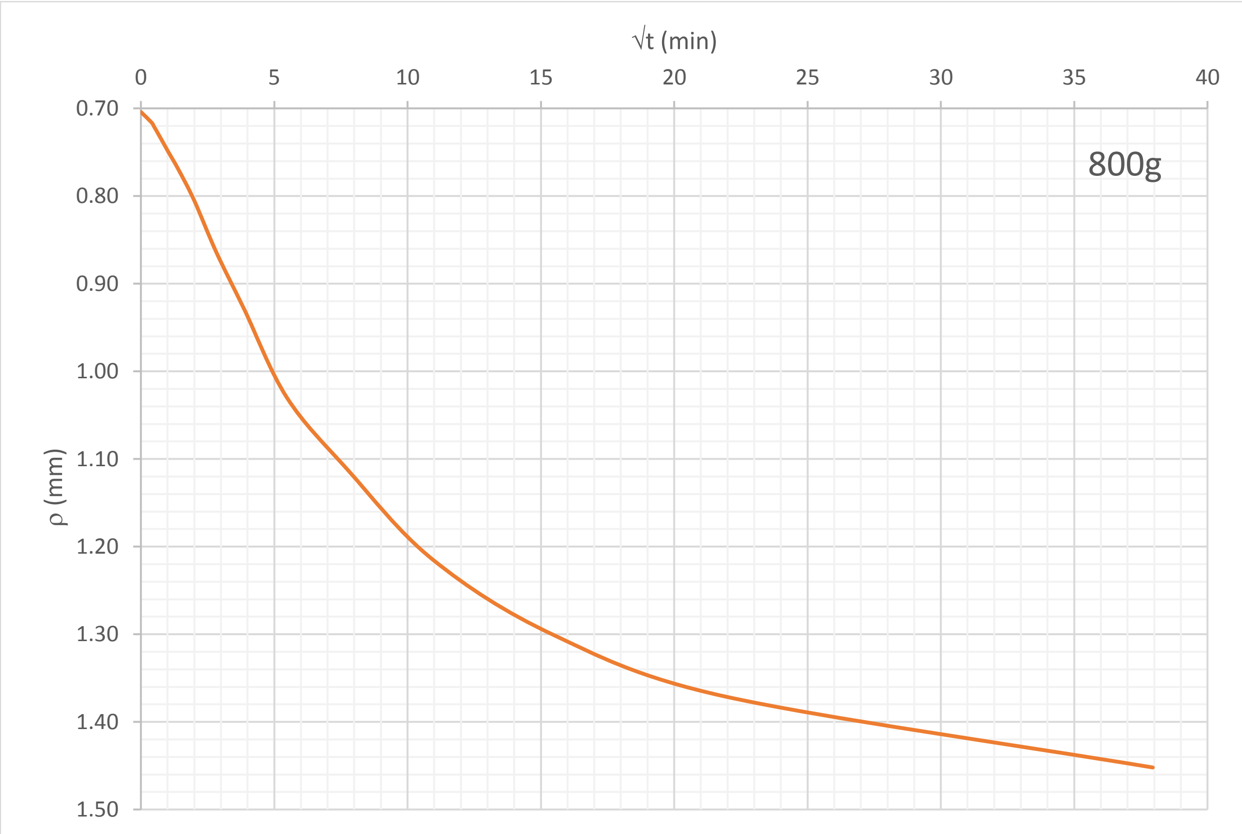
0.70
0.80
ρ (mm)
400g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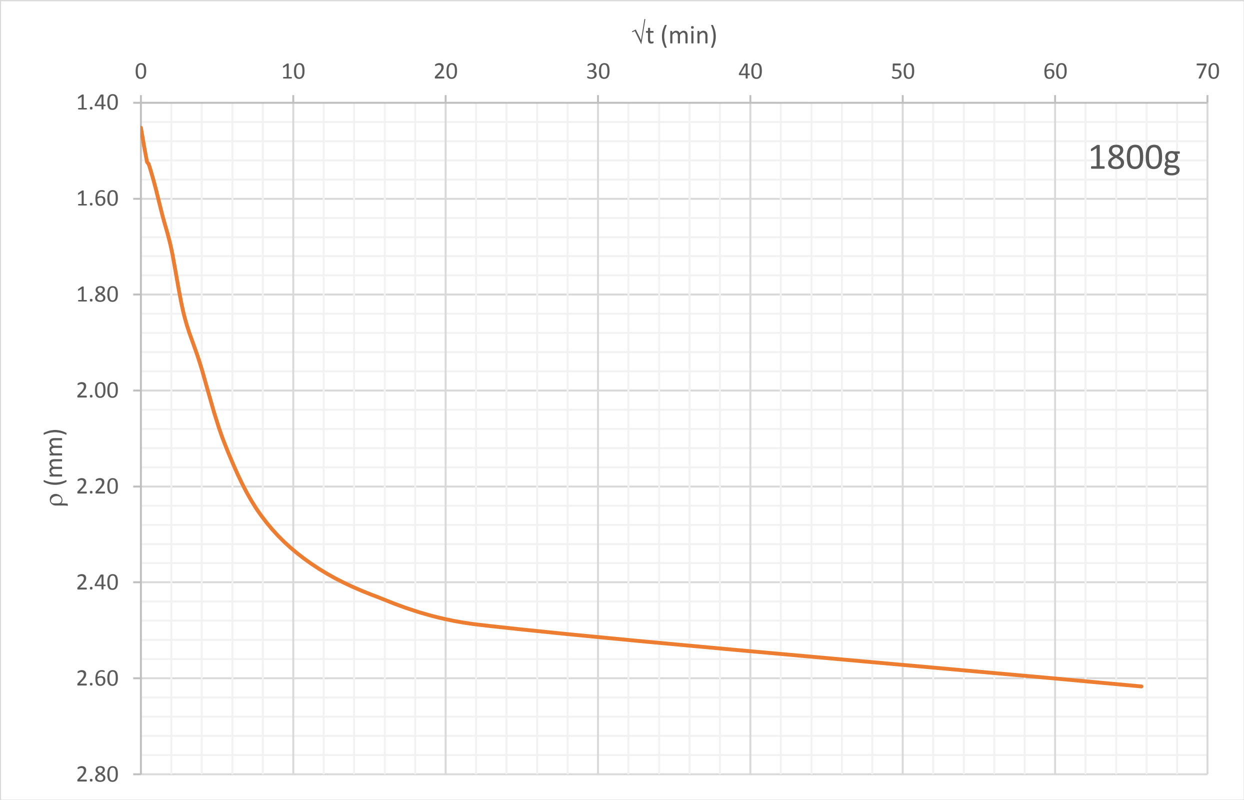
1.50
ρ (mm)
800g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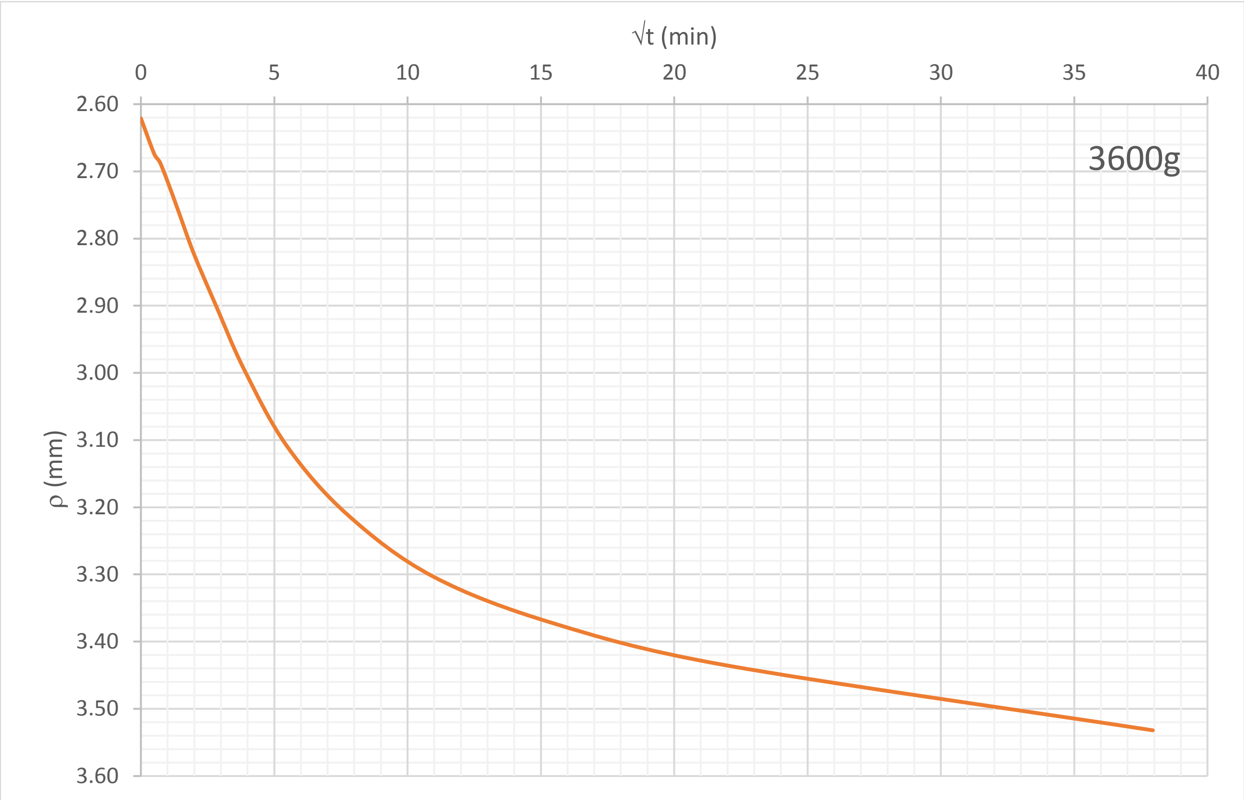
2.80
ρ (mm)
1800g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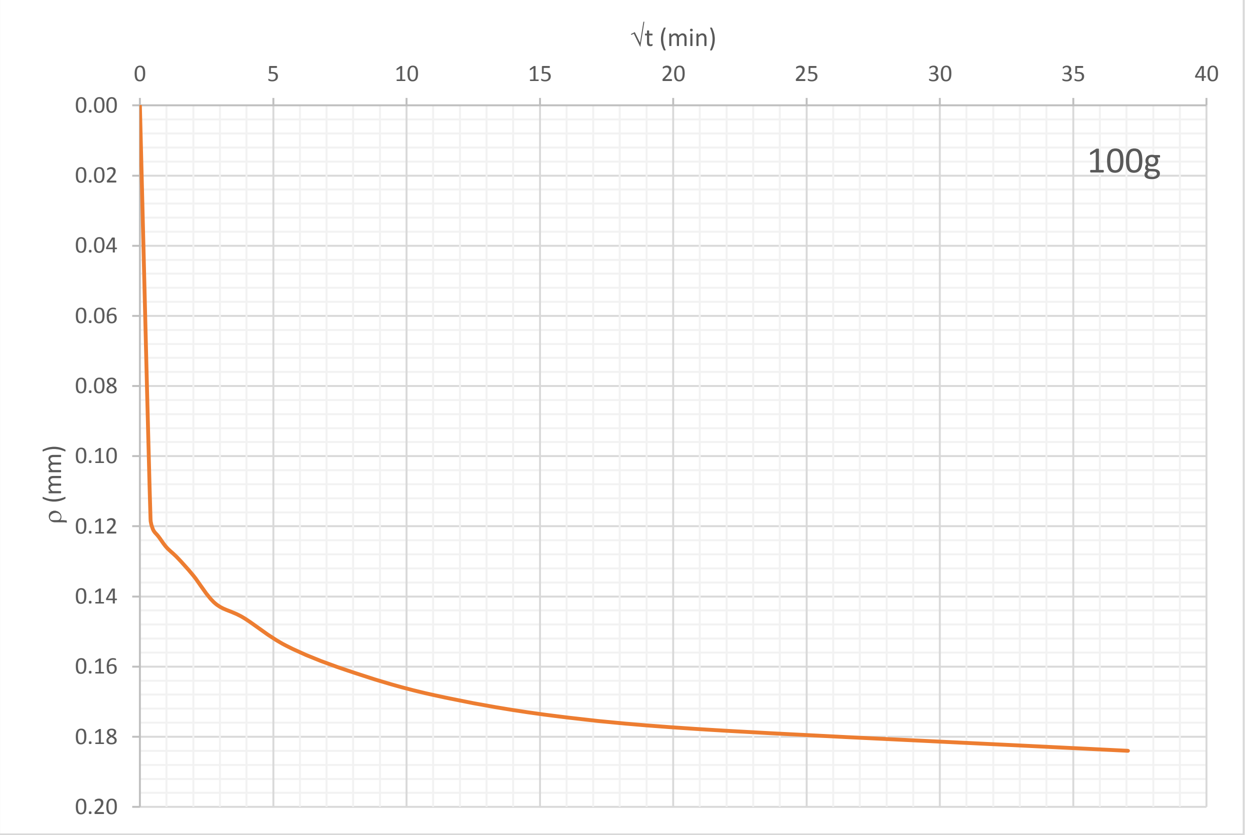
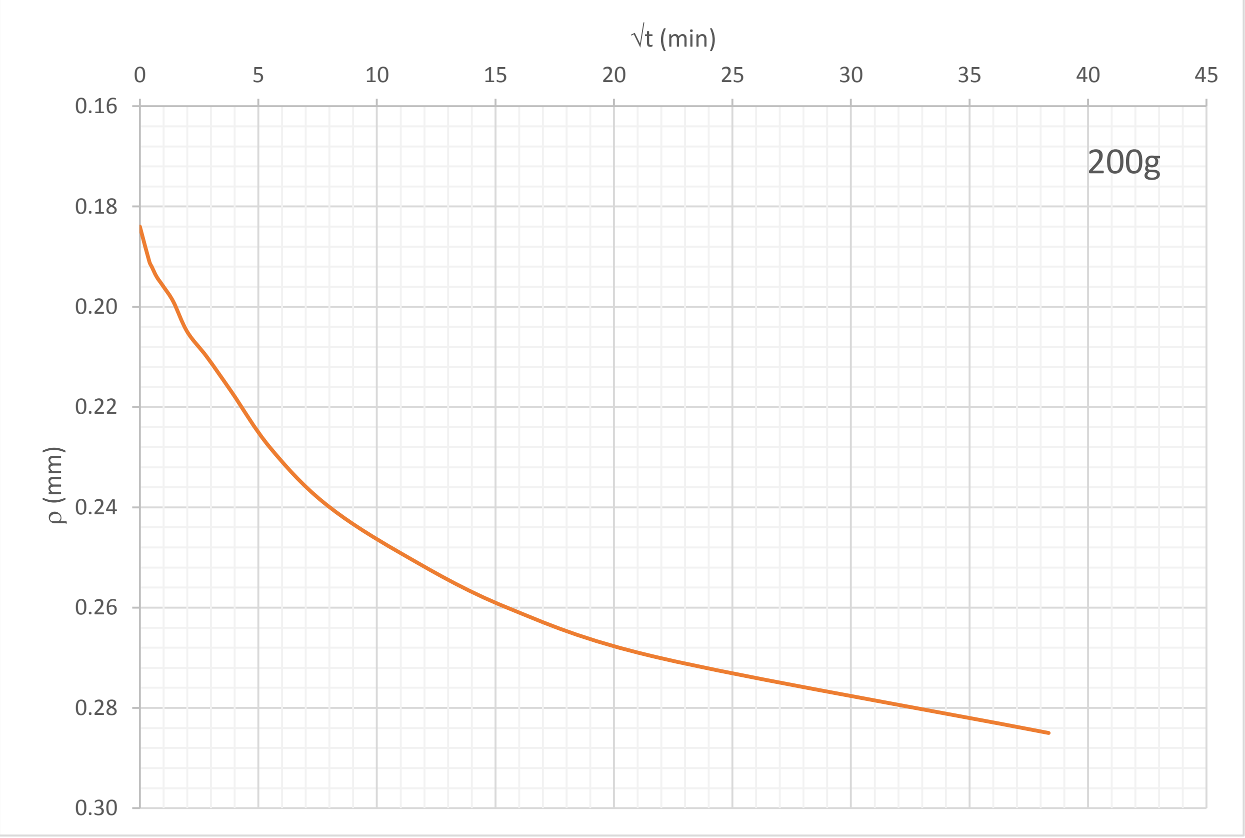
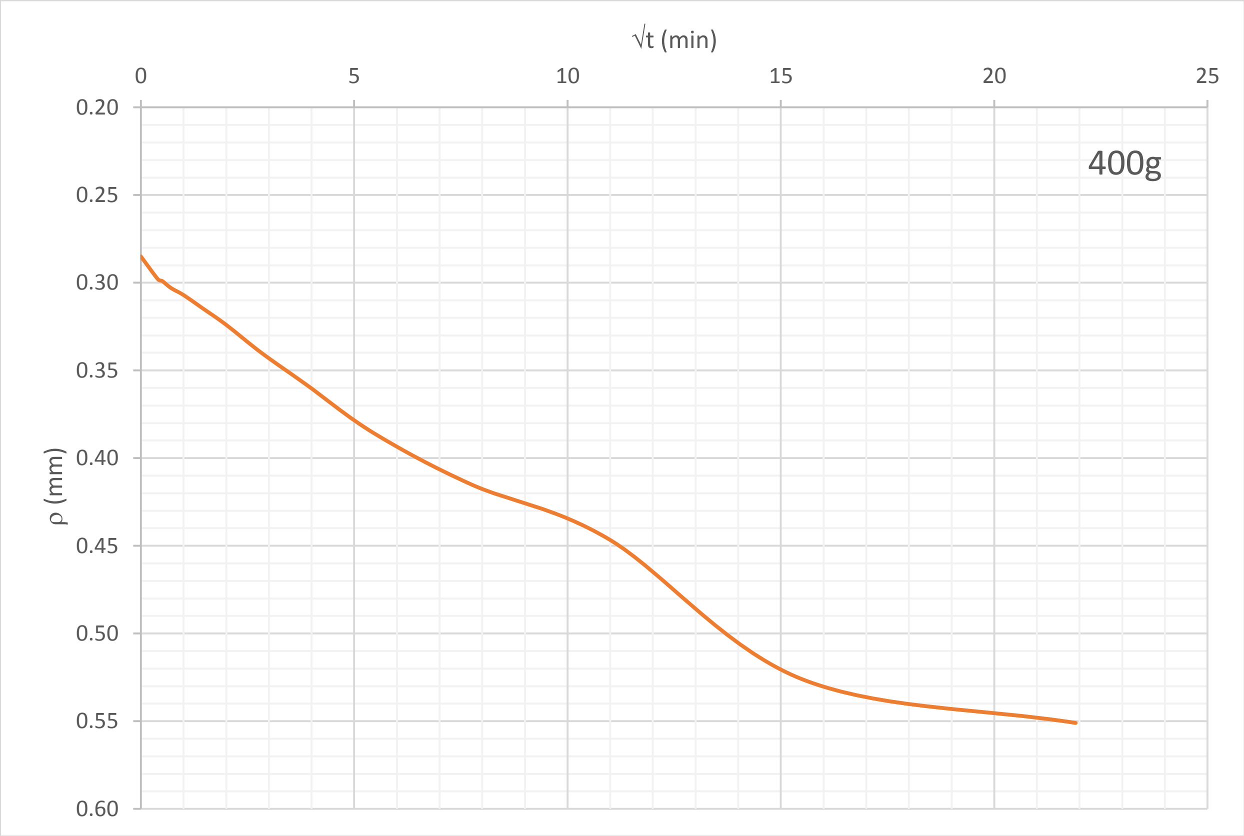
3.50
3.60
ρ (mm)
3600g

HU14.5, 76mm (-1.6m).



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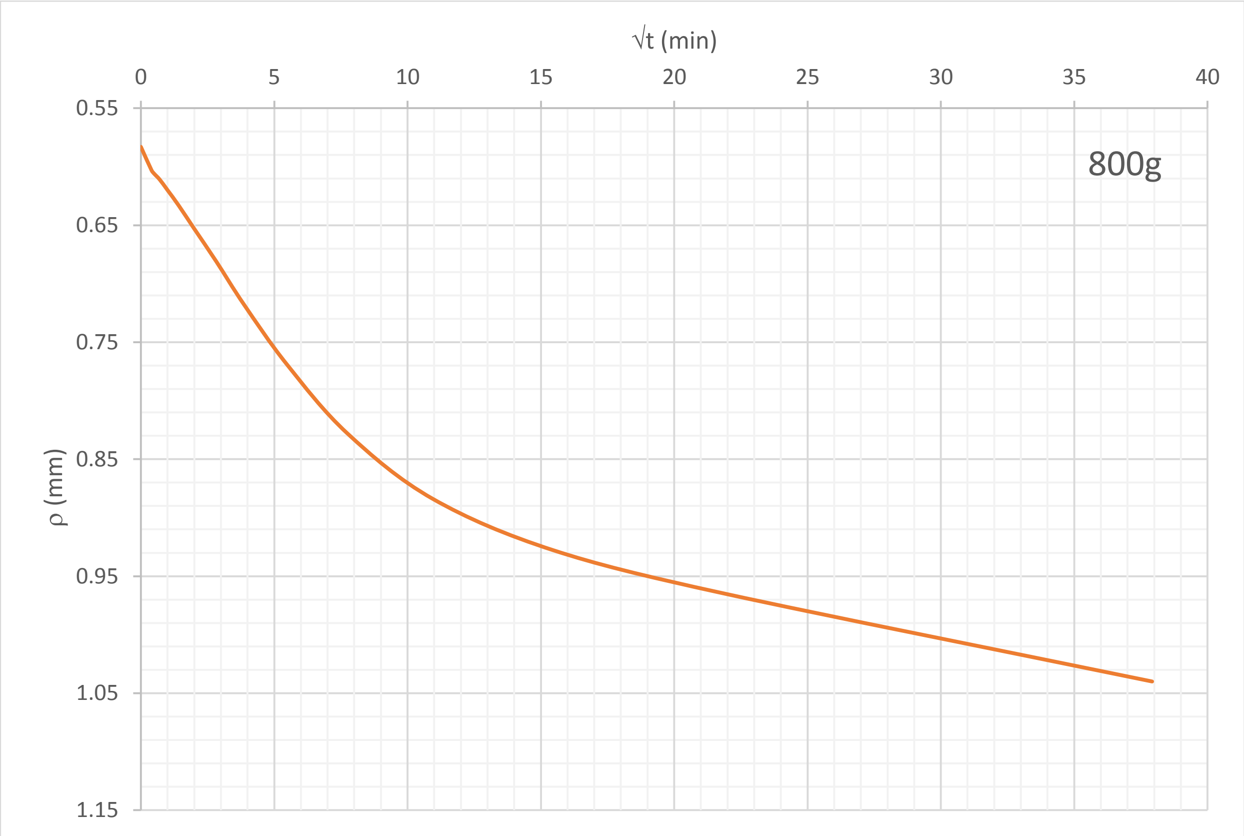
0.60
ρ (mm)
400g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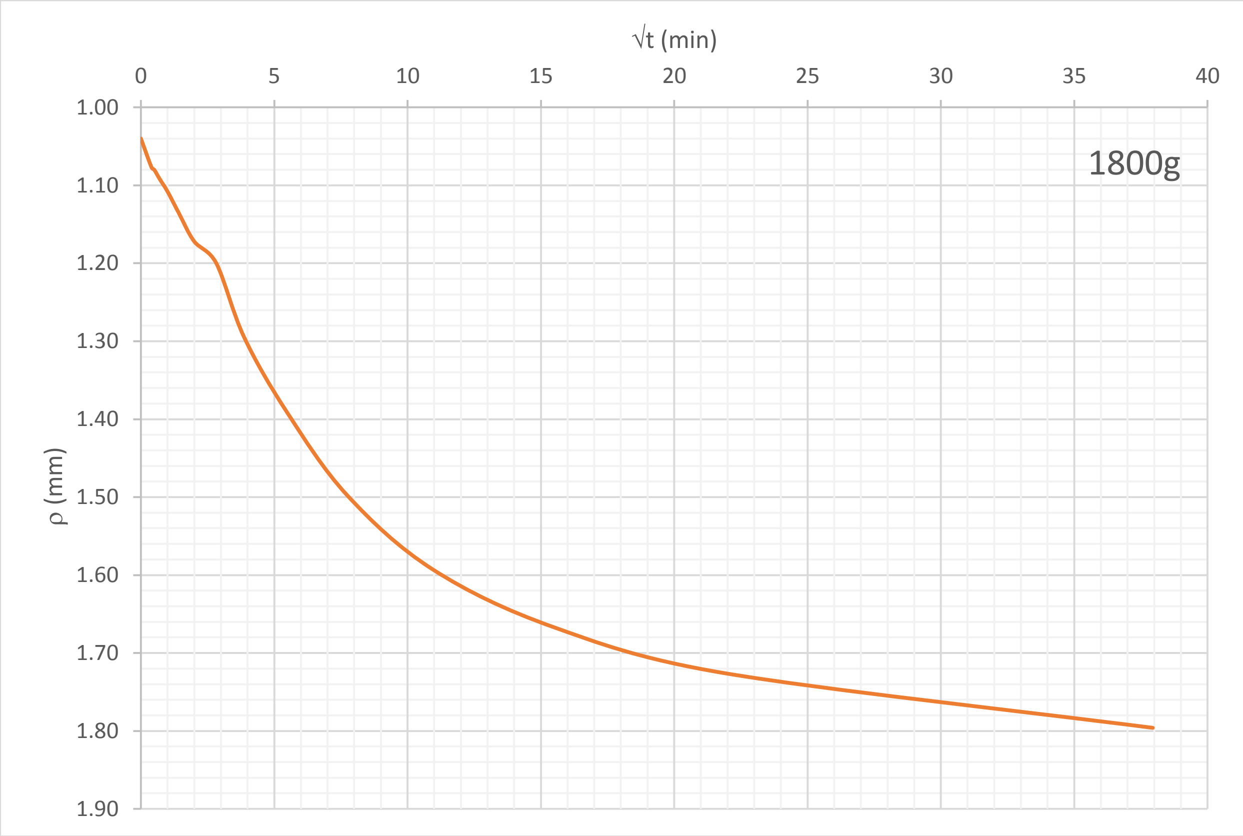
1.15
ρ (mm)
800g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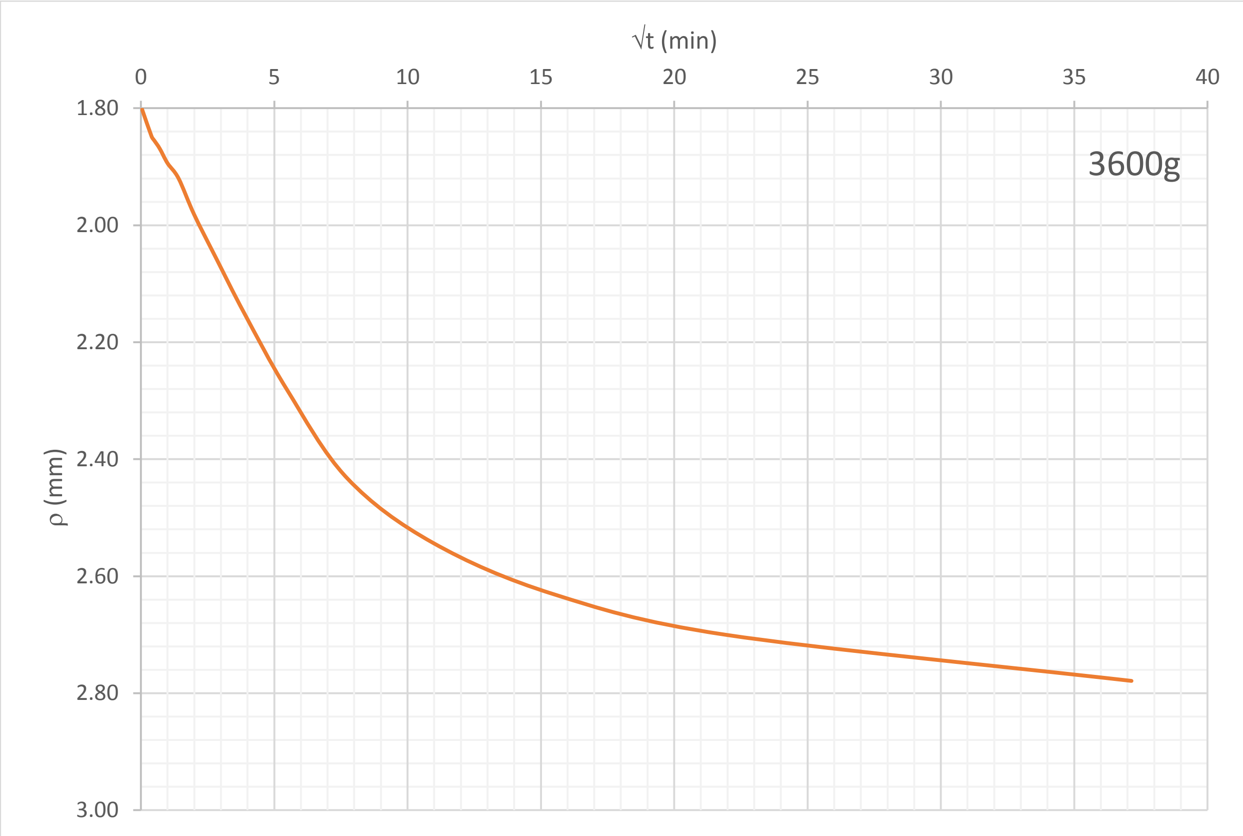
1.90
ρ (mm)
1800g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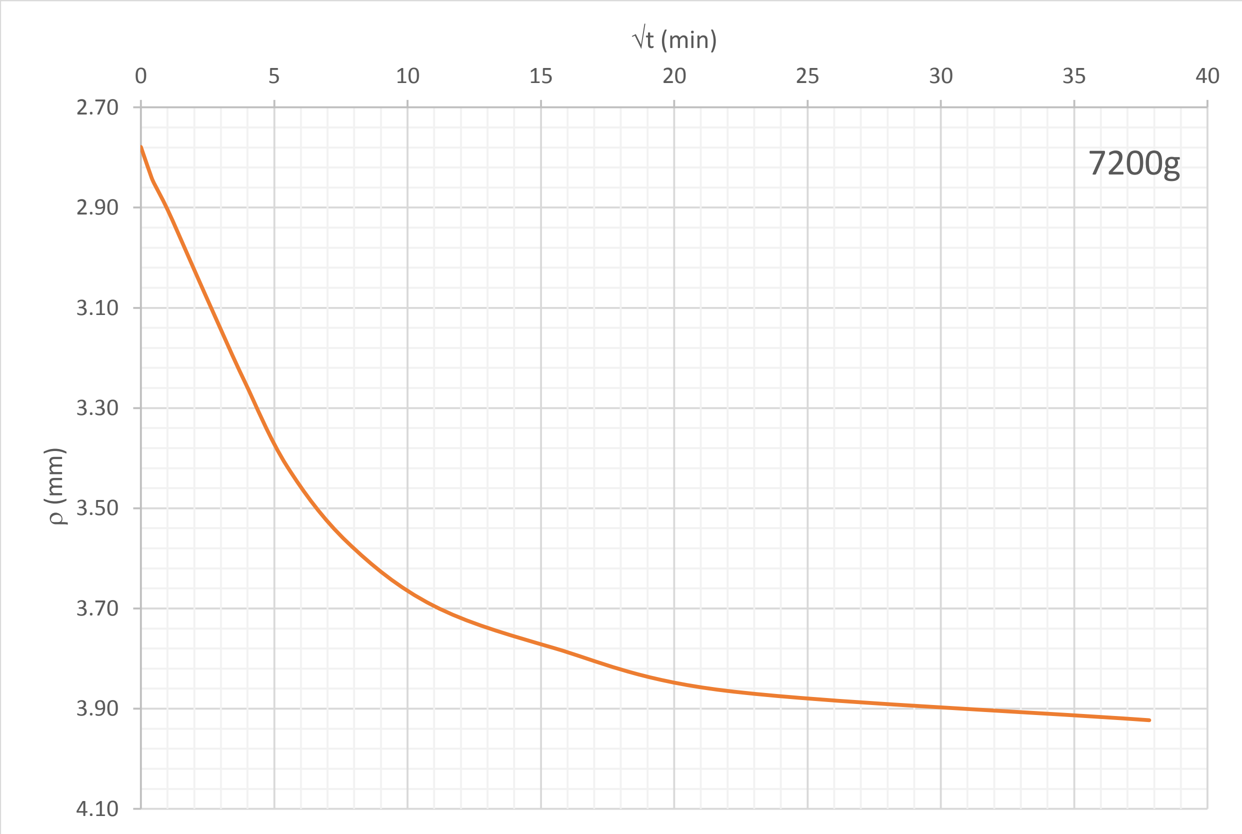
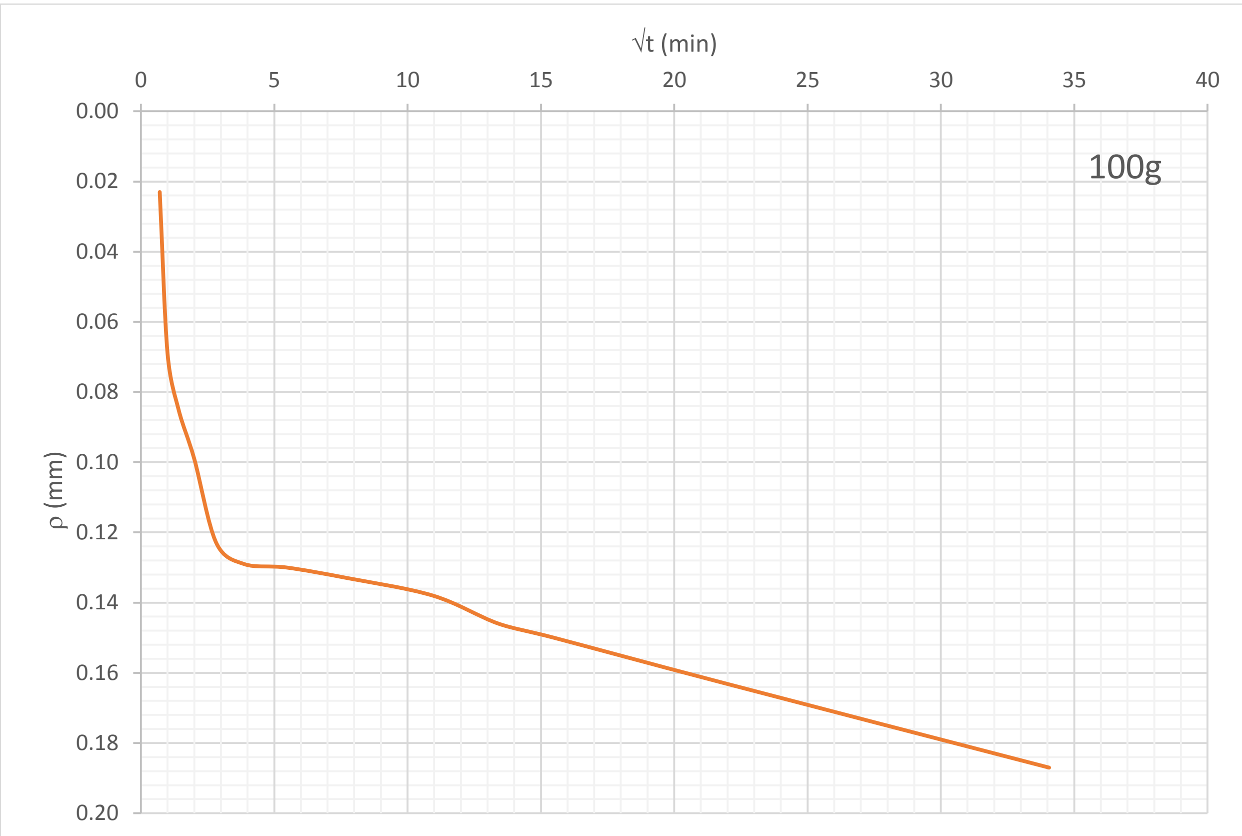
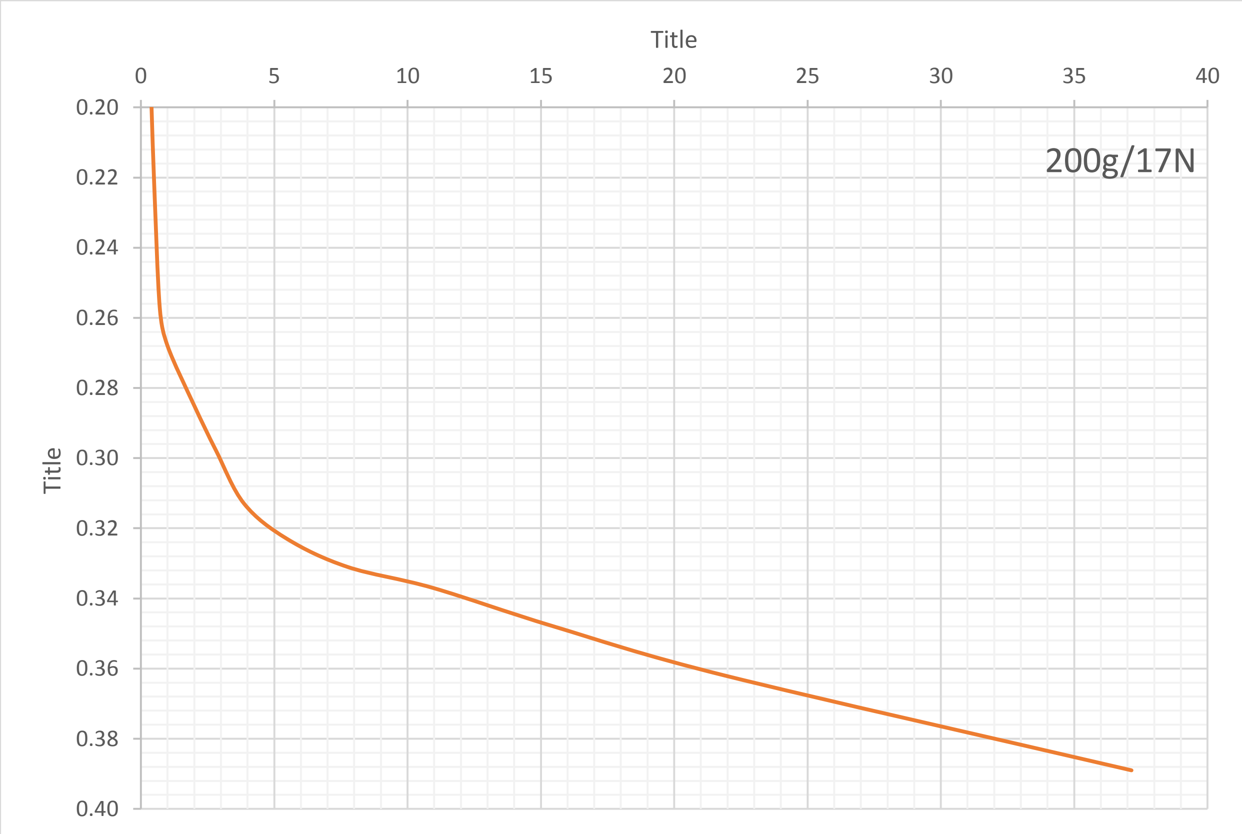
3.00
ρ (mm)
3600g



HU14.5 LHS, 75mm (-0.6m).

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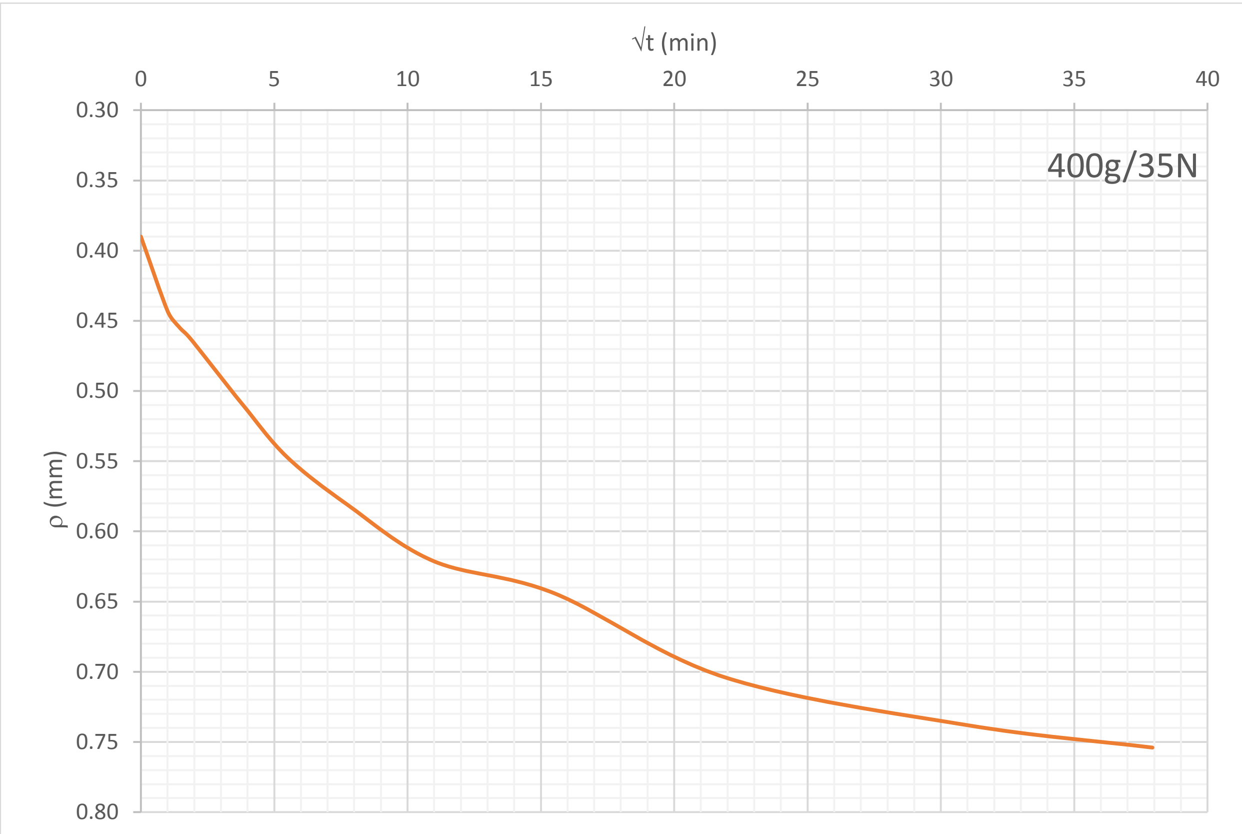
0.40
Title
200g/17N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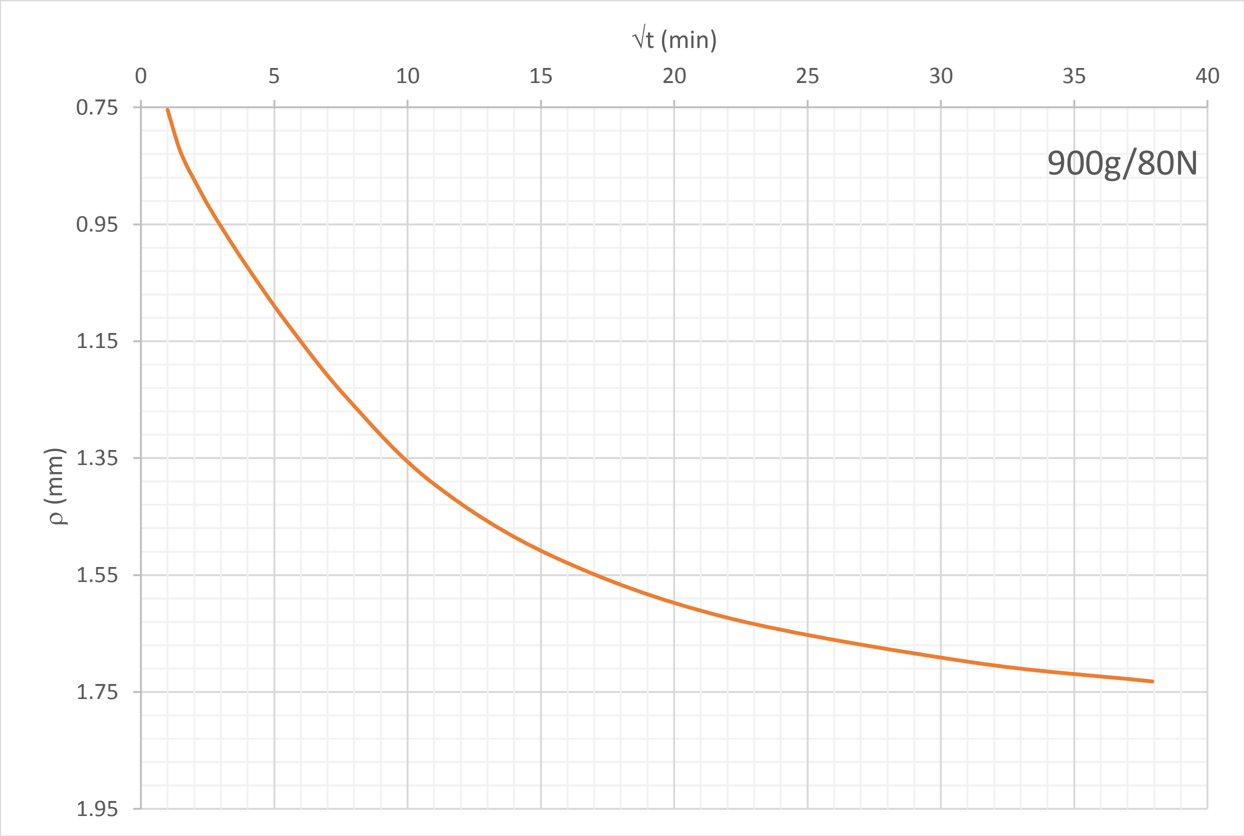
ρ (mm)
400g/35N



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0.95
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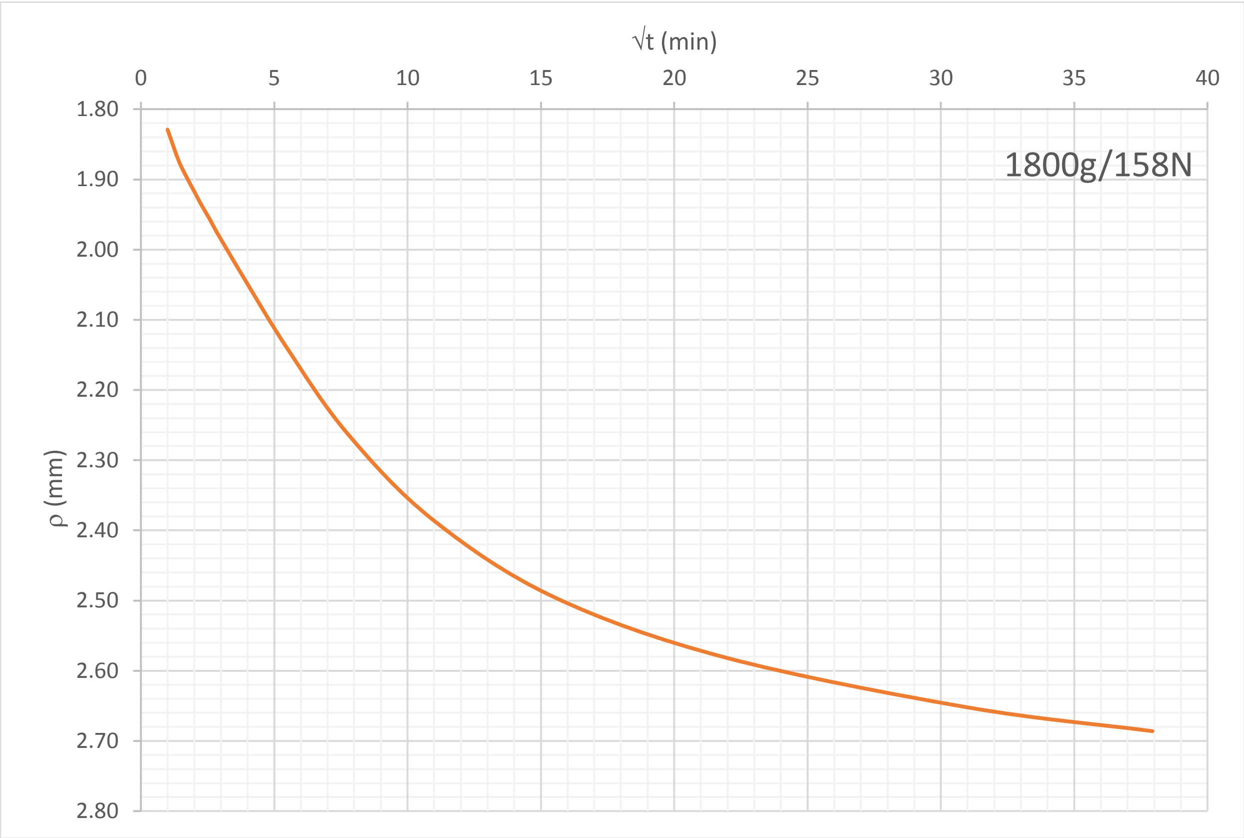
1.35
1.55
1.75
1.95
ρ (mm)
900g/80N



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1.80
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2.30
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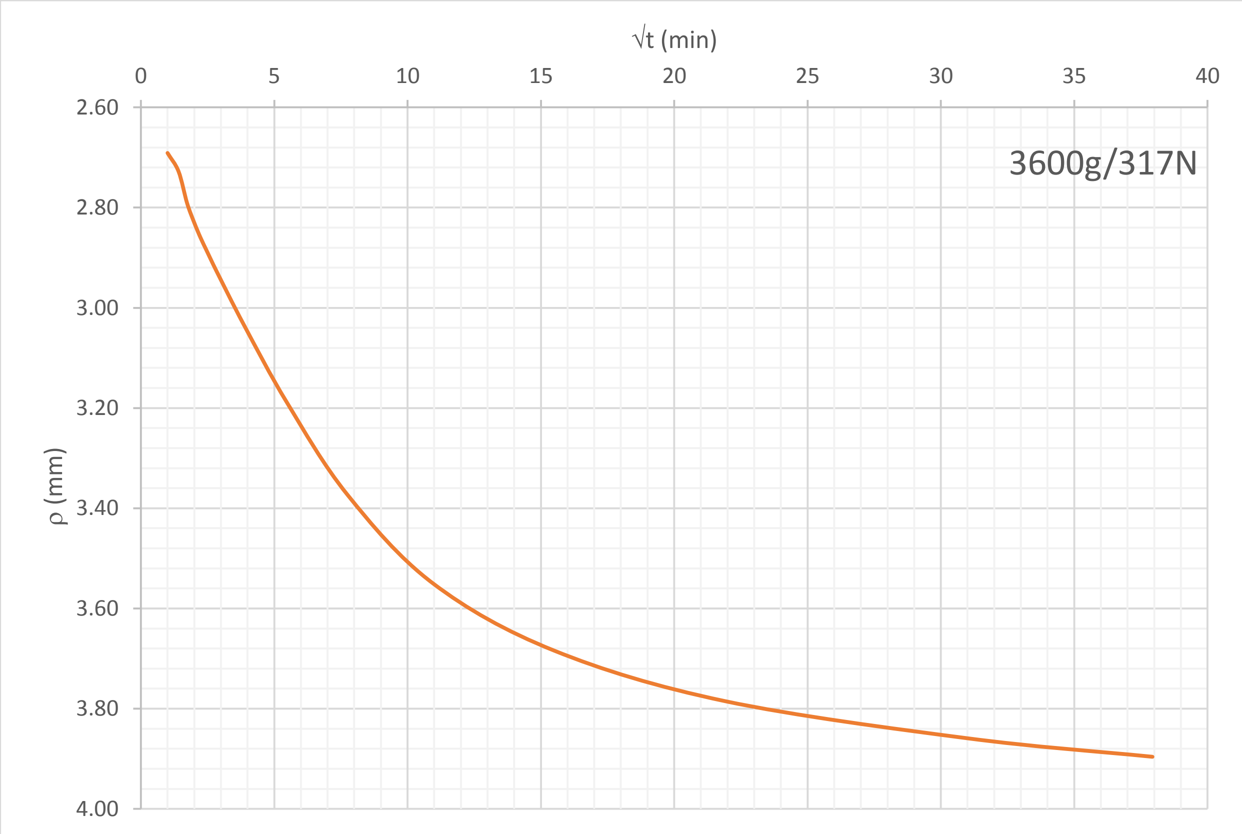
2.50
2.60
2.70
2.80
ρ (mm)
1800g/158N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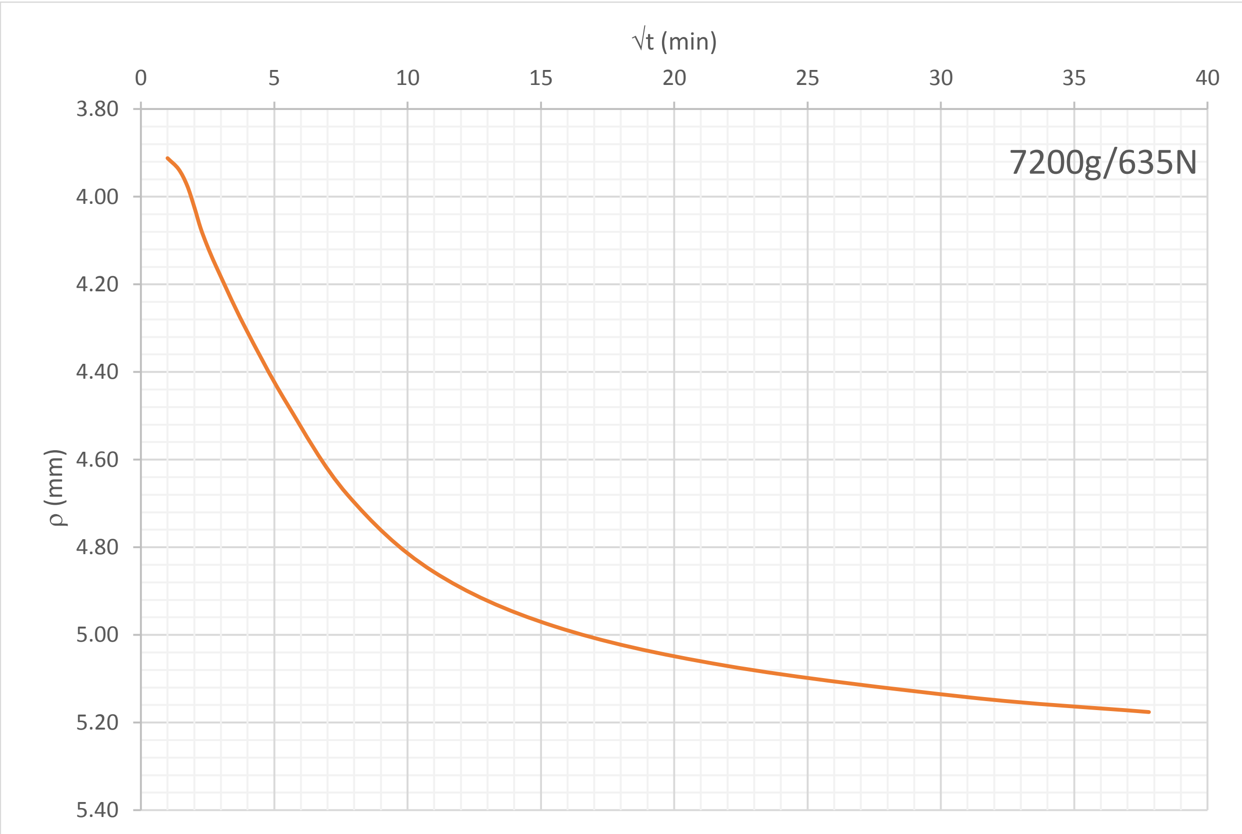
ρ (mm)
3600g/317N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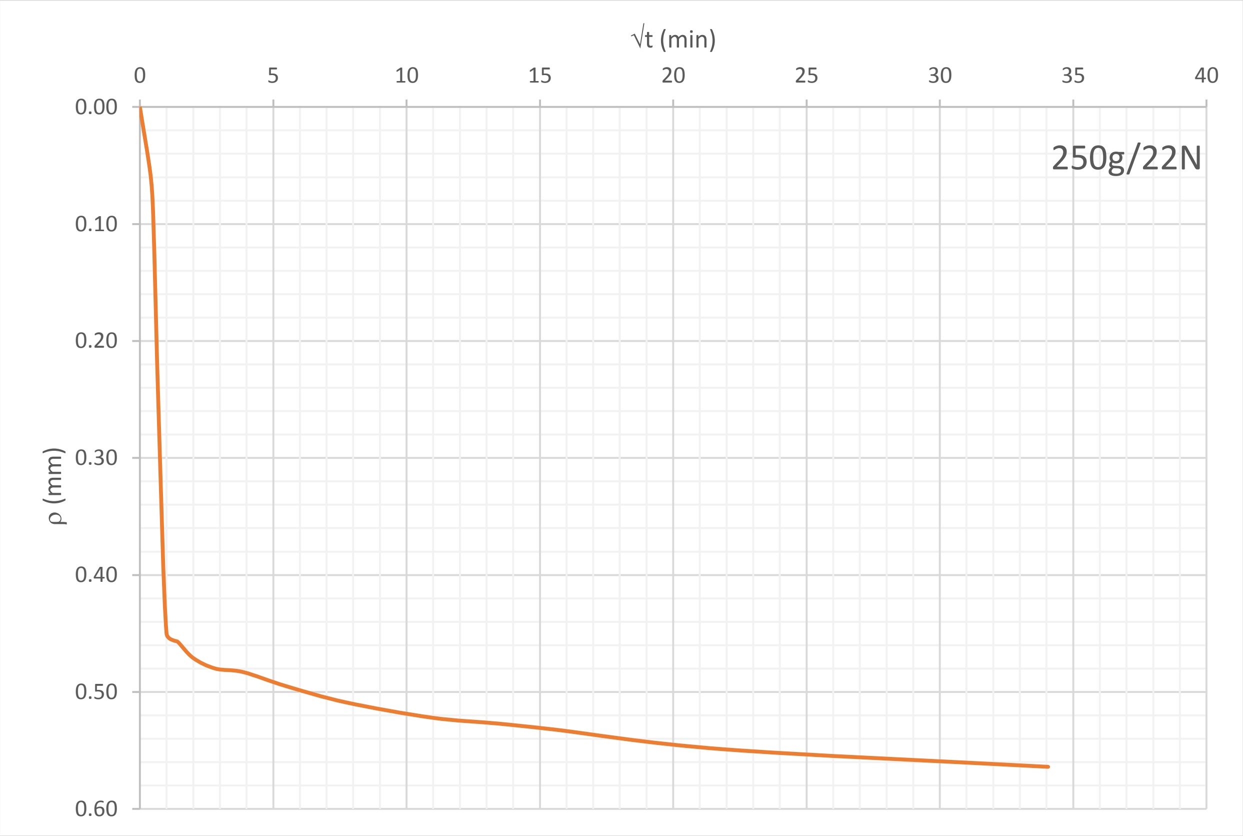
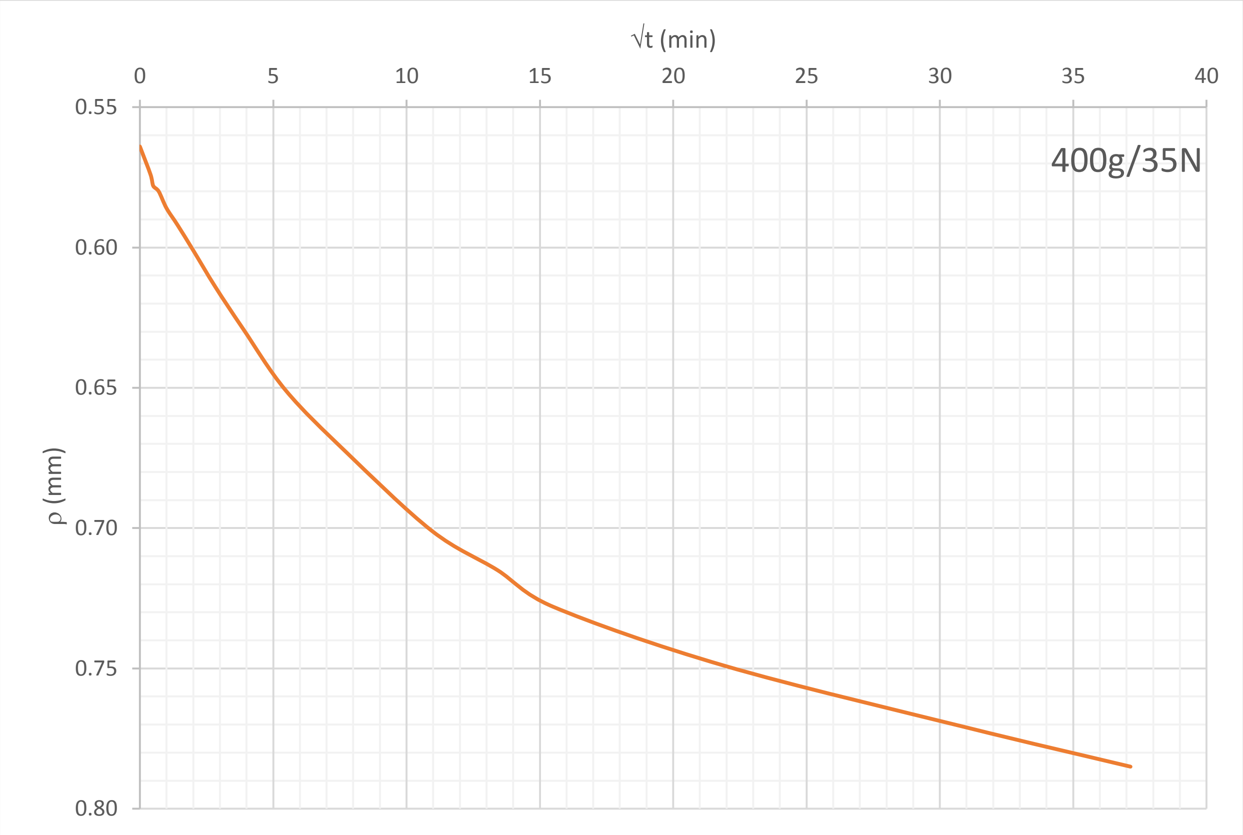
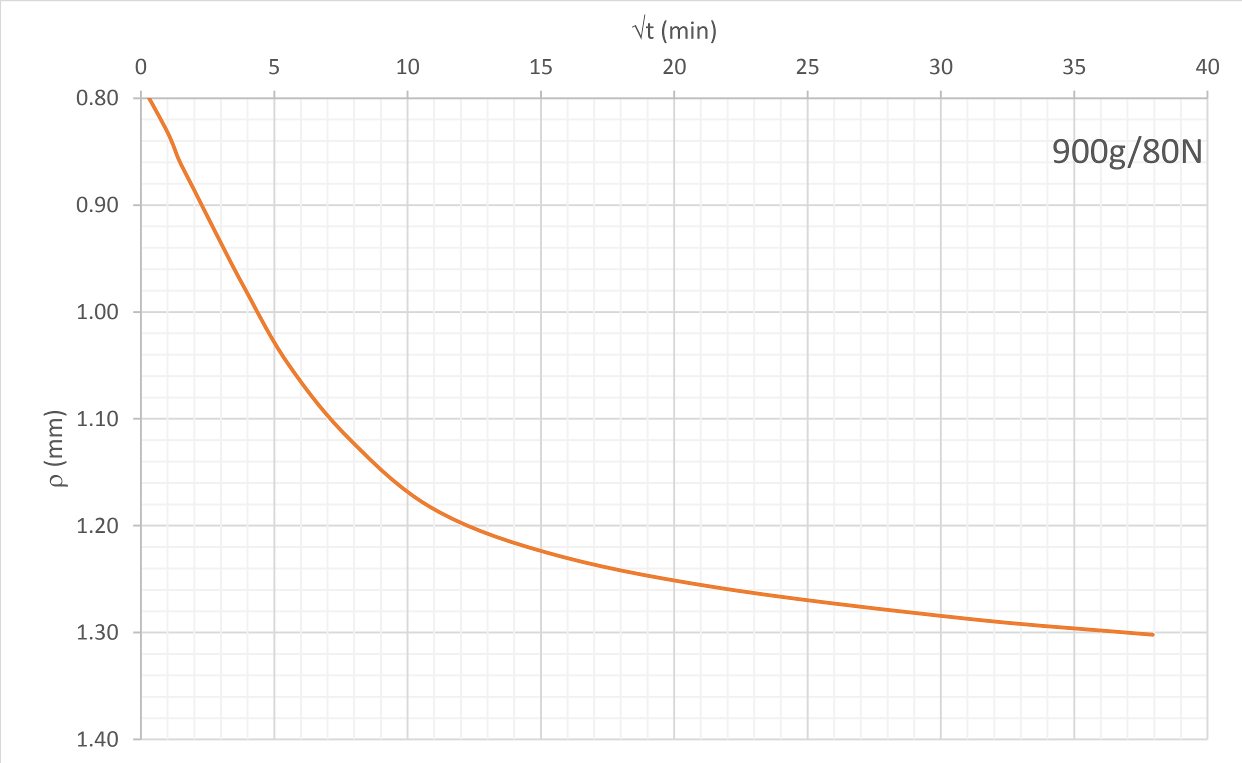
10
15
20
25
30
35
40
3.80
4.00
4.20
4.40
4.60
4.80
5.00
5.20
5.40
ρ (mm)
7200g/635N

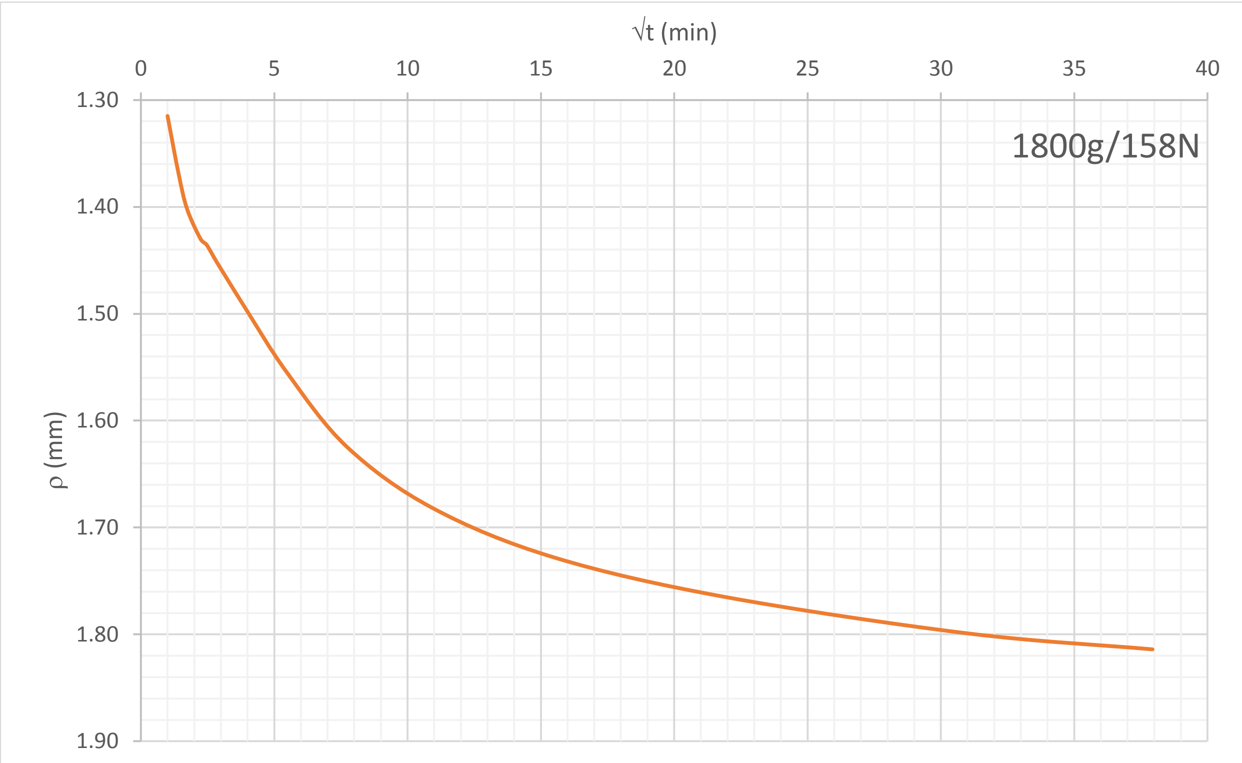
HU14.5 RHS, 75mm (-0.6m).



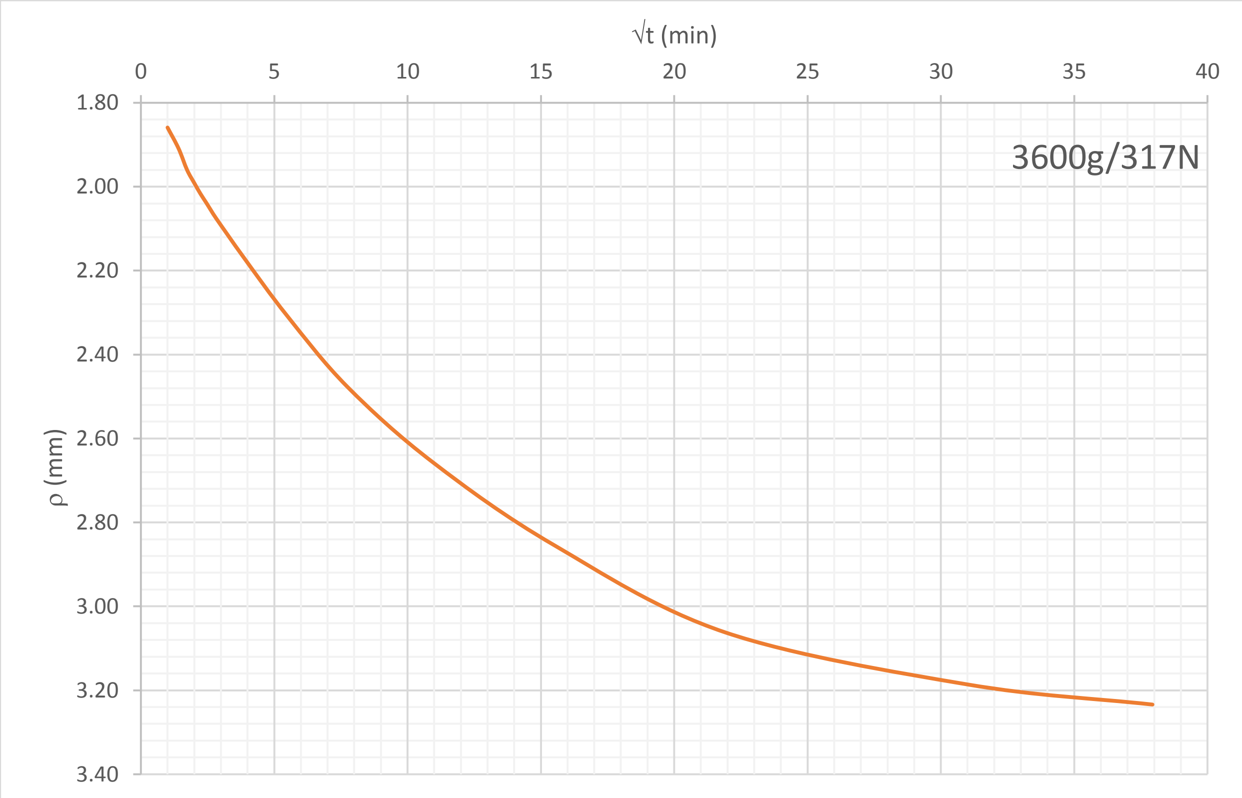




√t (min)
0
5
10
15
20
25
30
35
40
0.80
0.90
1.00
1.10
1.20
1.30
1.40
ρ (mm)
900g/80N



√t (min)
0
5
10
15
20
25
30
35
40
1.30
1.40
1.50
1.60
1.70
1.80
1.90
ρ (mm)
1800g/158N


√t (min)
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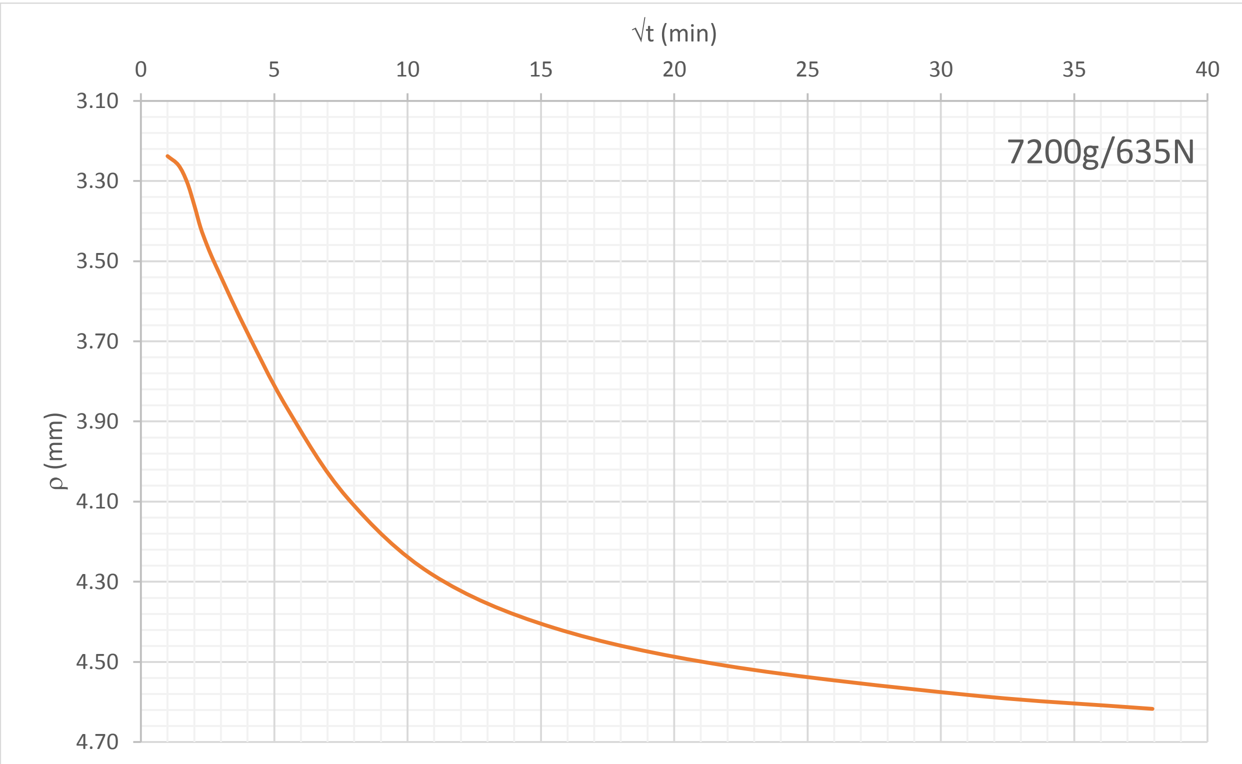
5
10
15
20
25
30
35
40
1.80
2.00
2.20
2.40
2.60
2.80
3.00
3.20
3.40
ρ (mm)
3600g/317N


√t (min)
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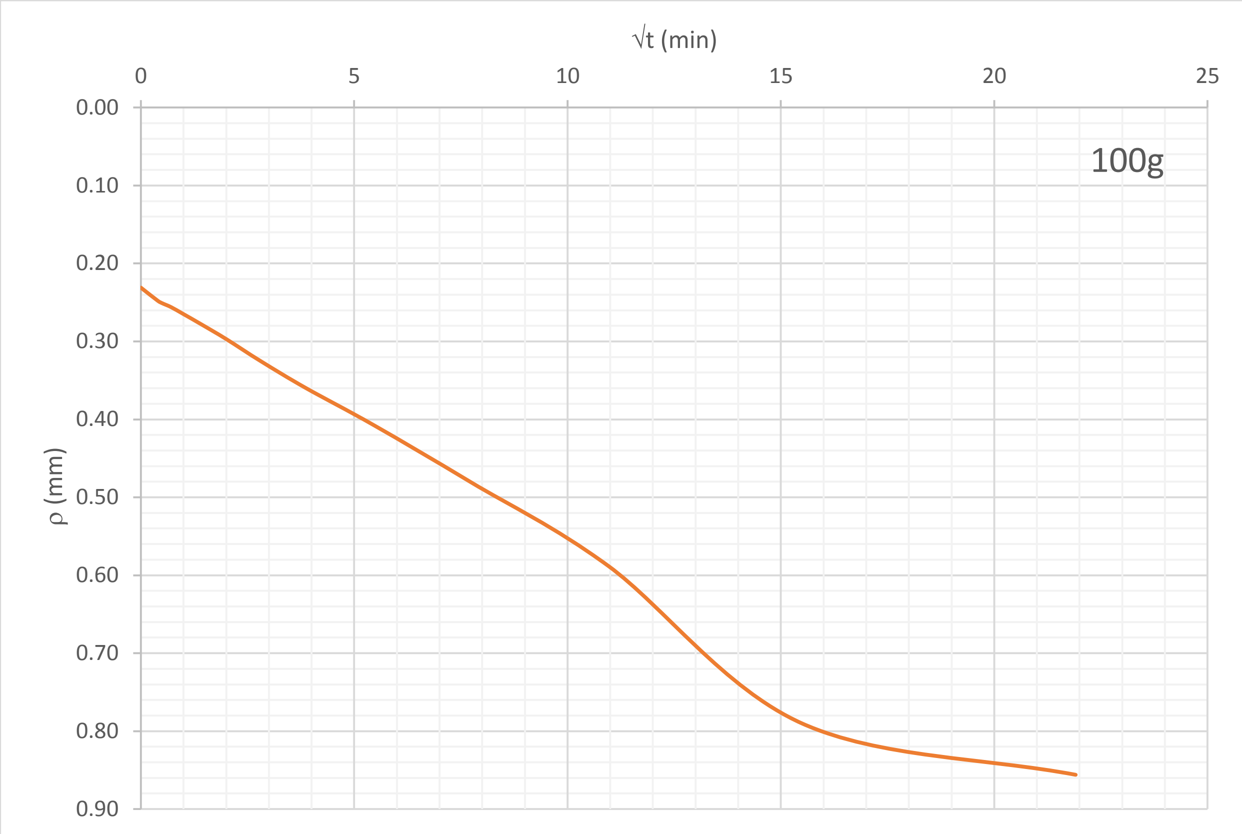
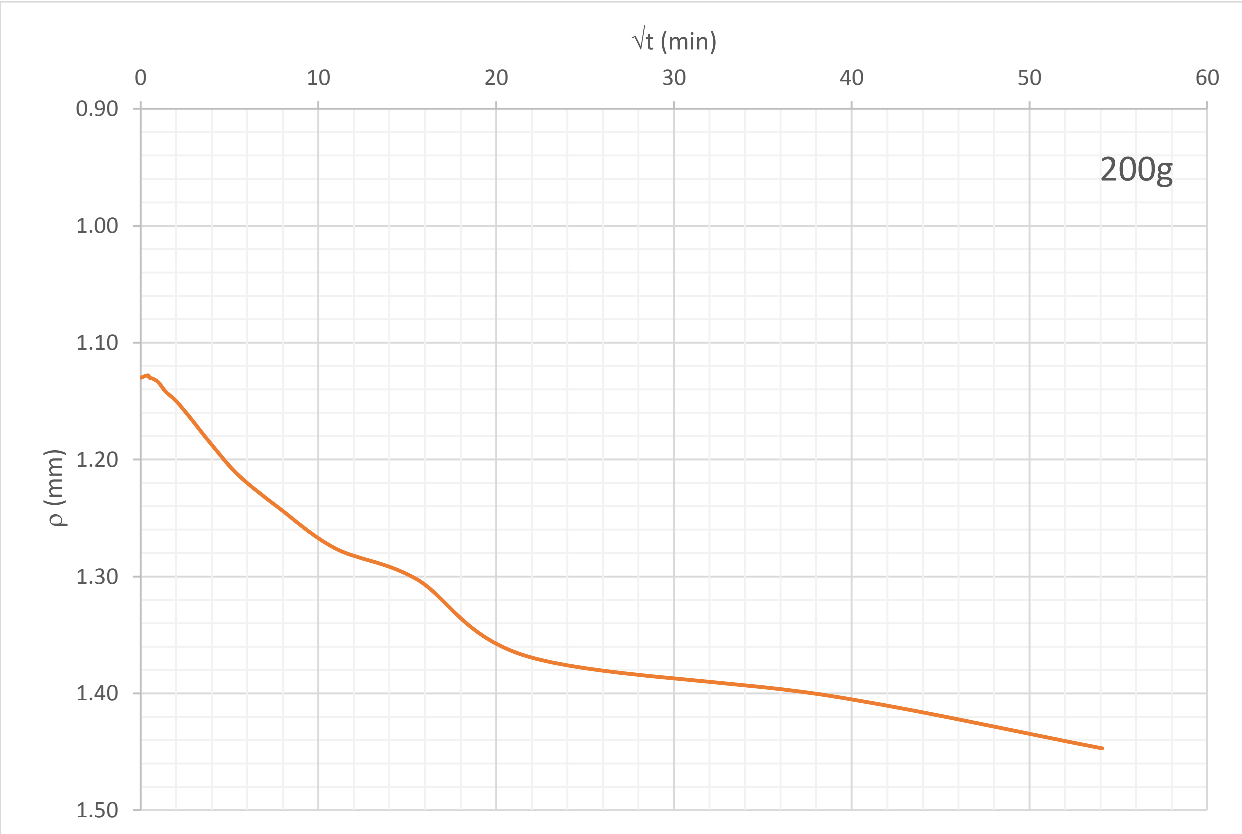
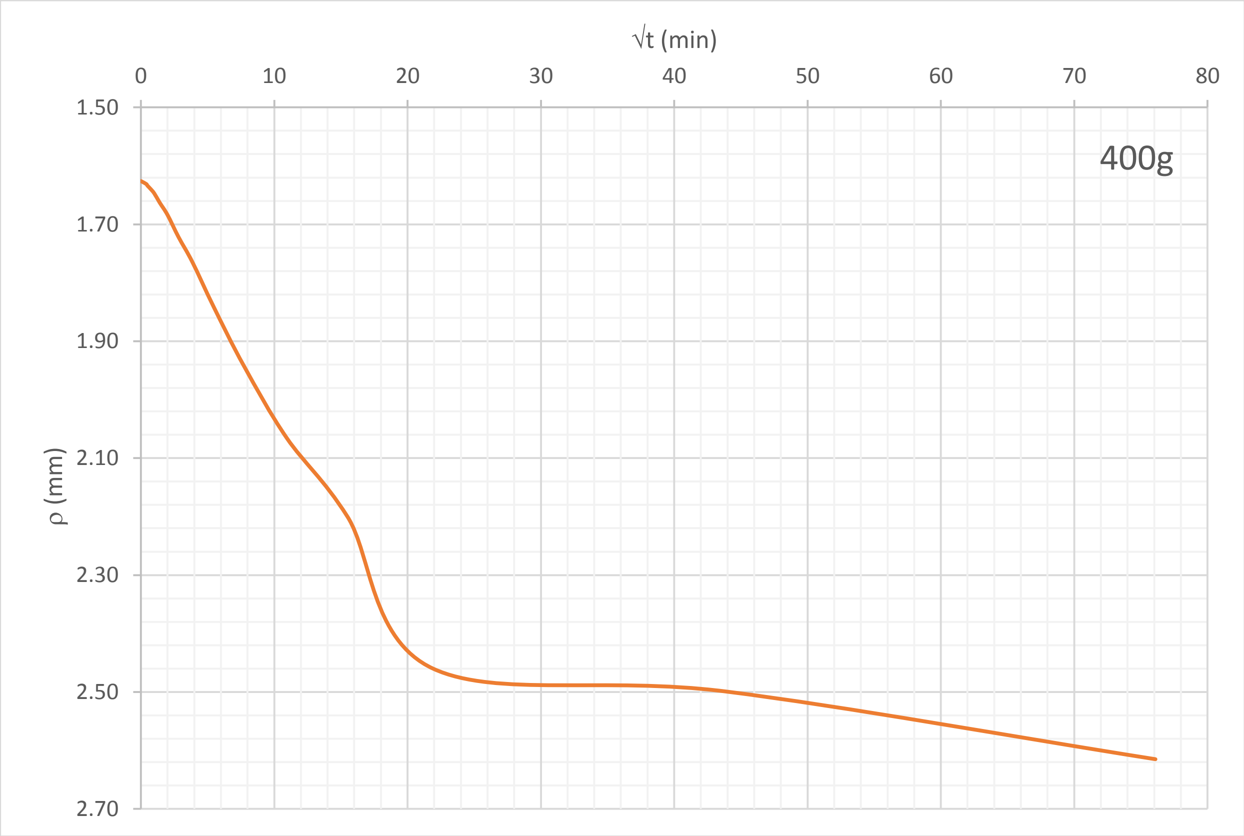
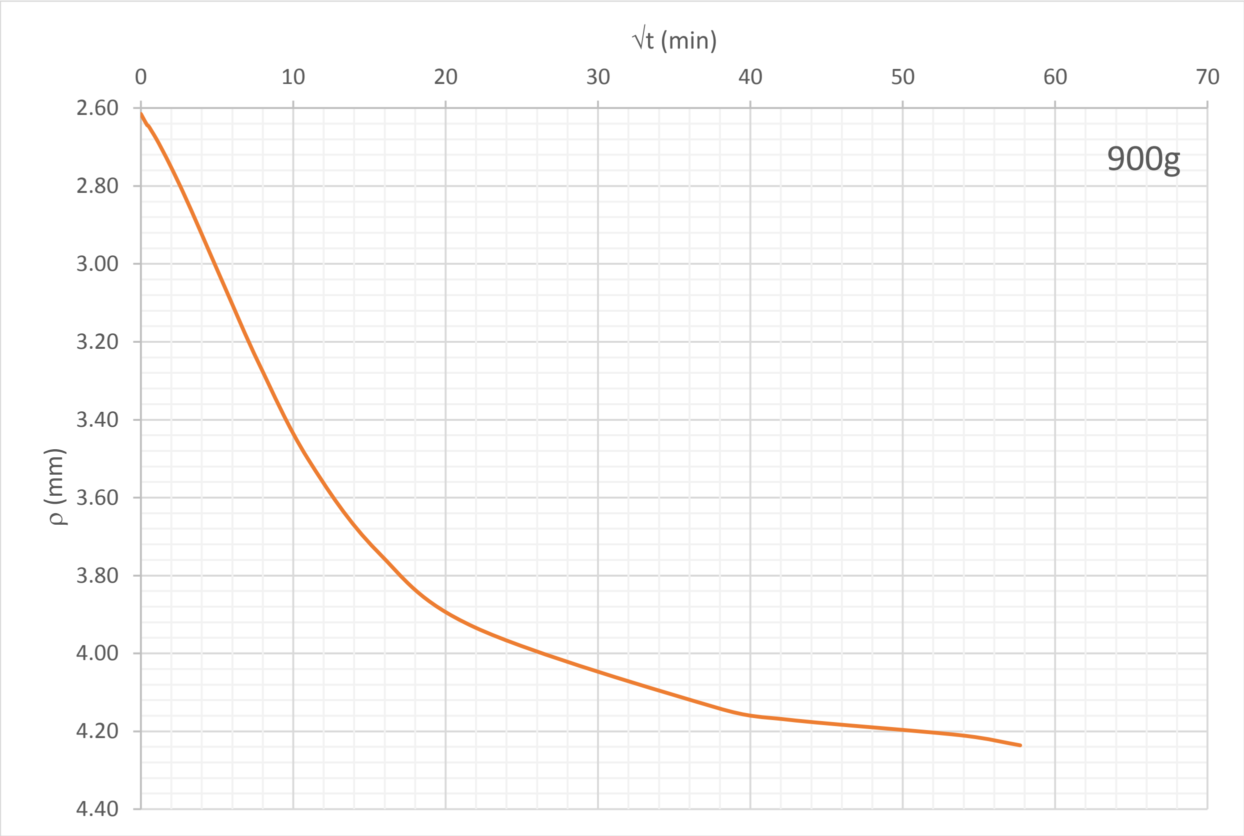
5
10
15
20
25
30
35
40
3.10
3.30
3.50
3.70
3.90
4.10
4.30
4.50
4.70
ρ (mm)
7200g/635N

HU8, 50mm (-0.15m).






√t (min)
0
10
20
30
40
50
60
70
80
1.50
1.70
1.90
2.10
2.30
2.50
2.70
ρ (mm)
400g


√t (min)
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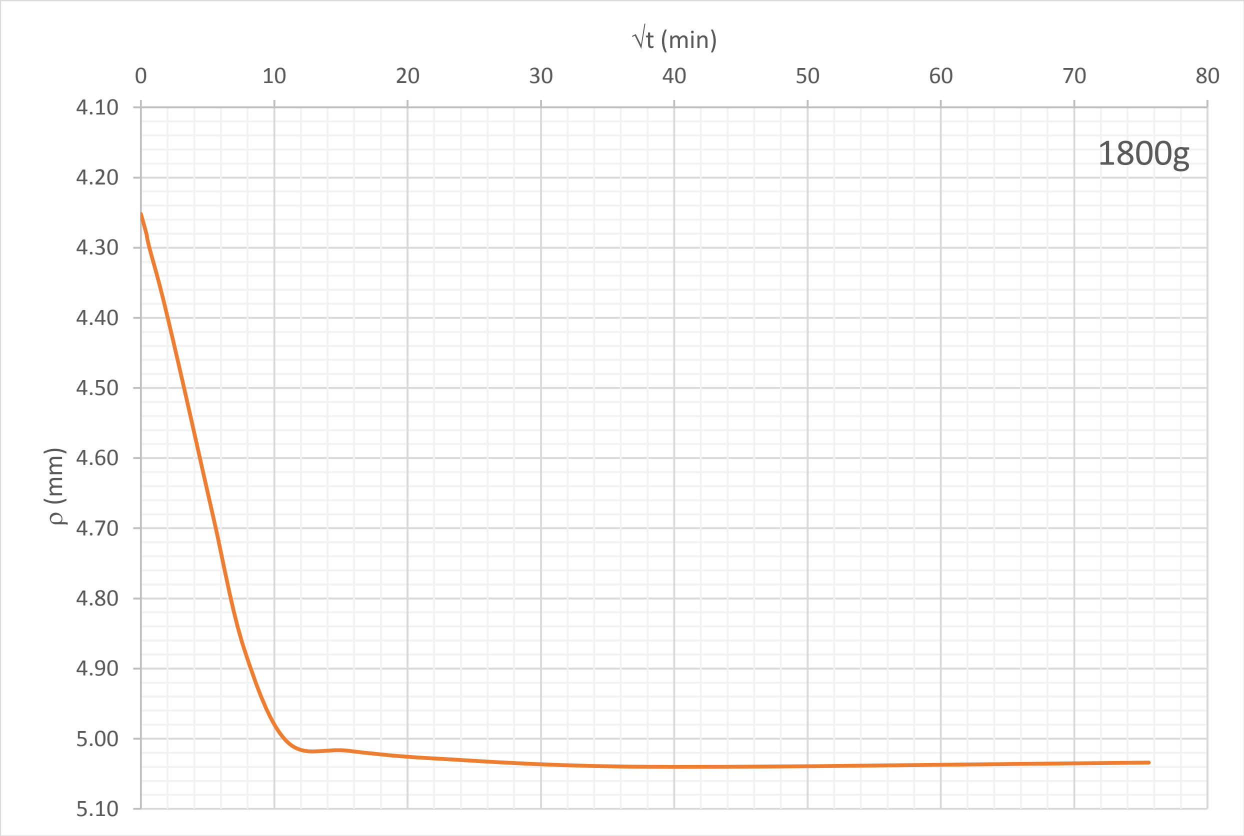
20
30
40
50
60
70
2.60
2.80
3.00
3.20
3.40
3.60
3.80
4.00
4.20
4.40
ρ (mm)
900g


√t (min)
0
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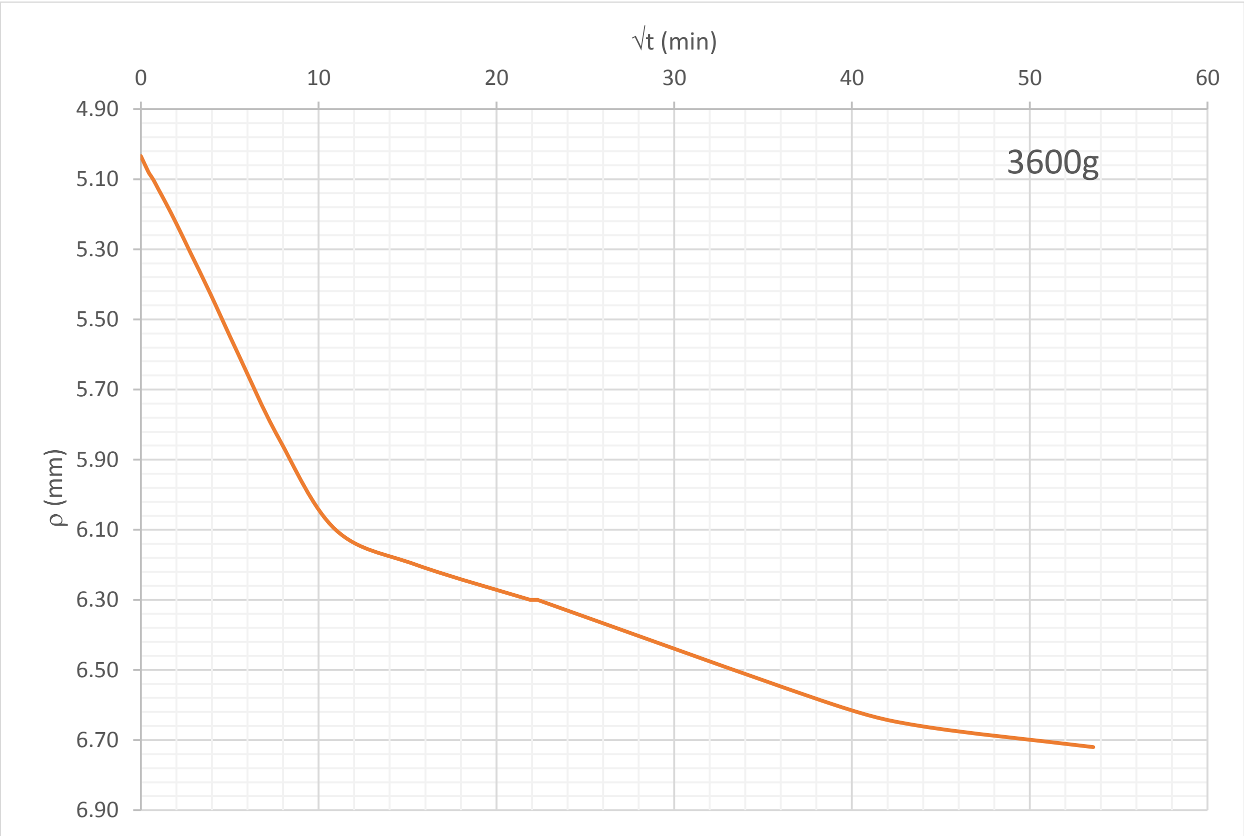
20
30
40
50
60
70
80
4.10
4.20
4.30
4.40
4.50
4.60
4.70
4.80
4.90
5.00
5.10
ρ (mm)
1800g


√t (min)
0
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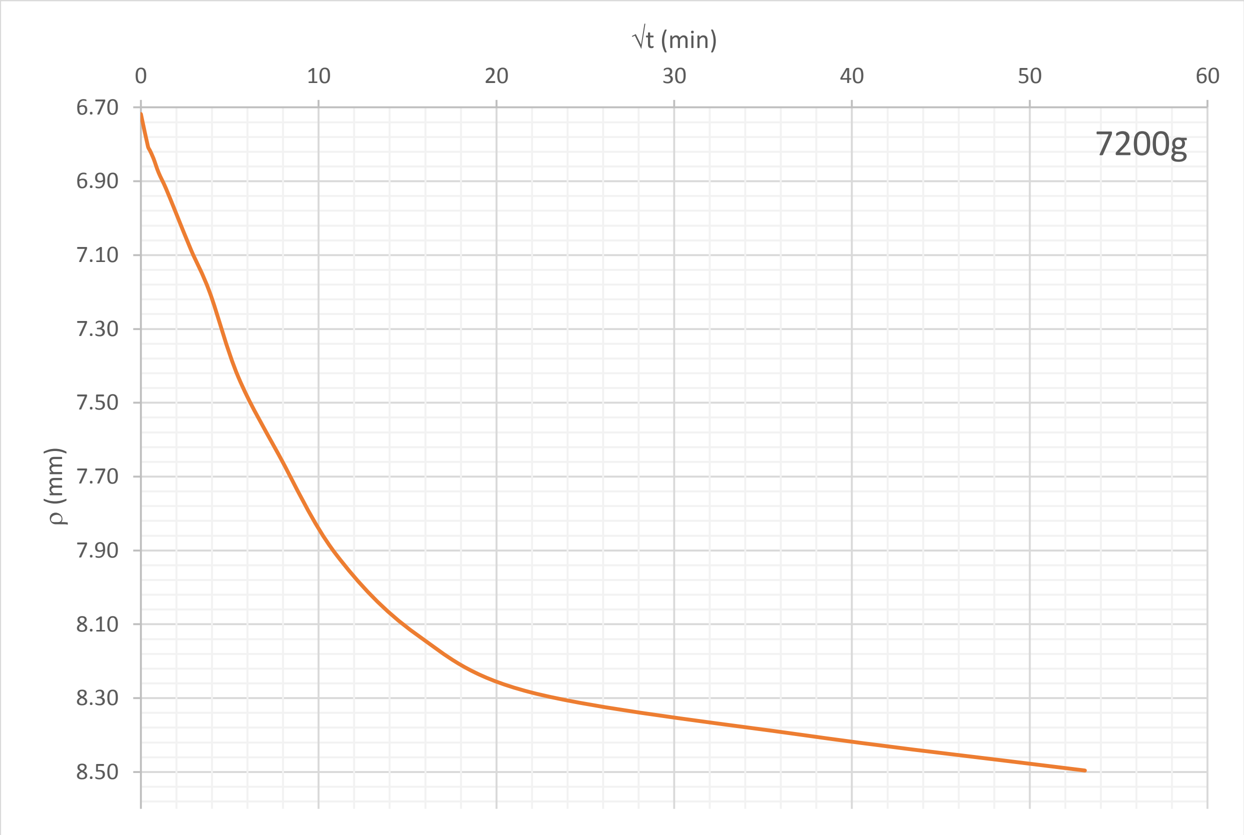
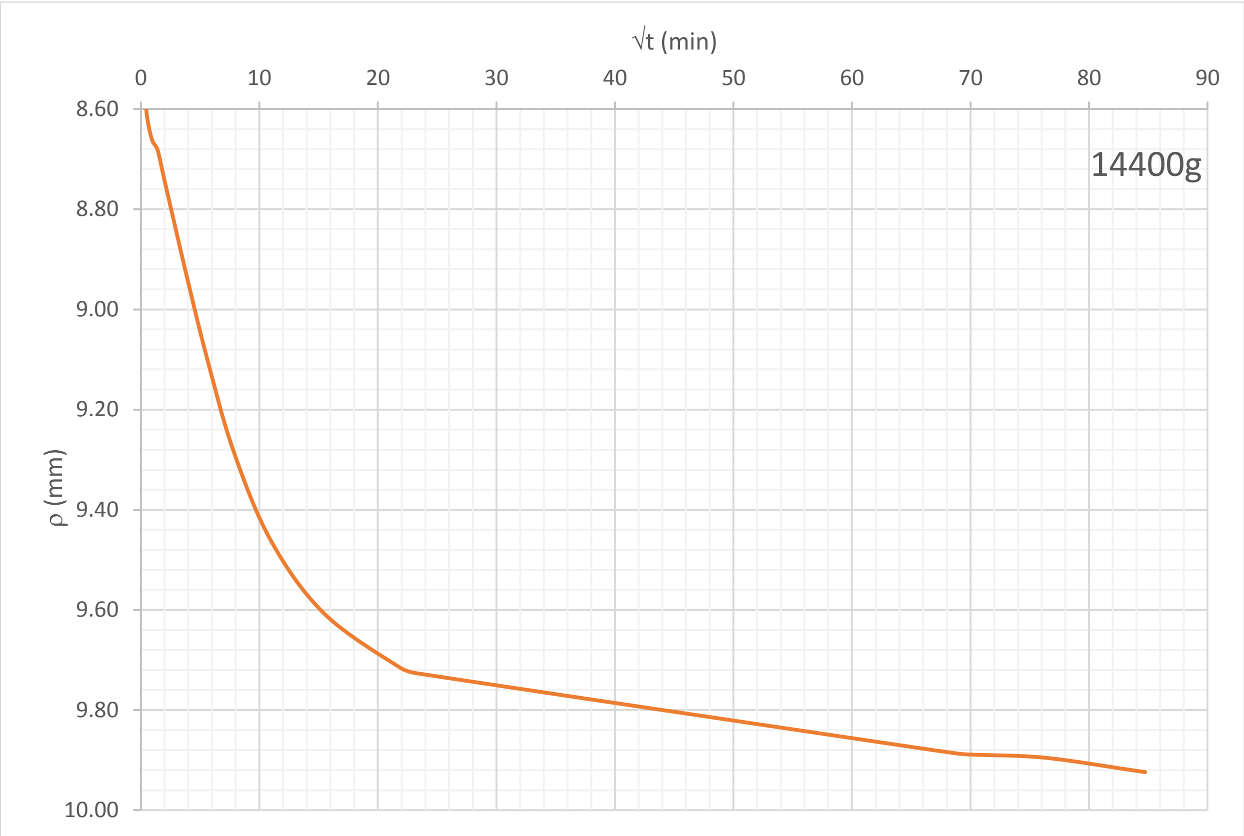
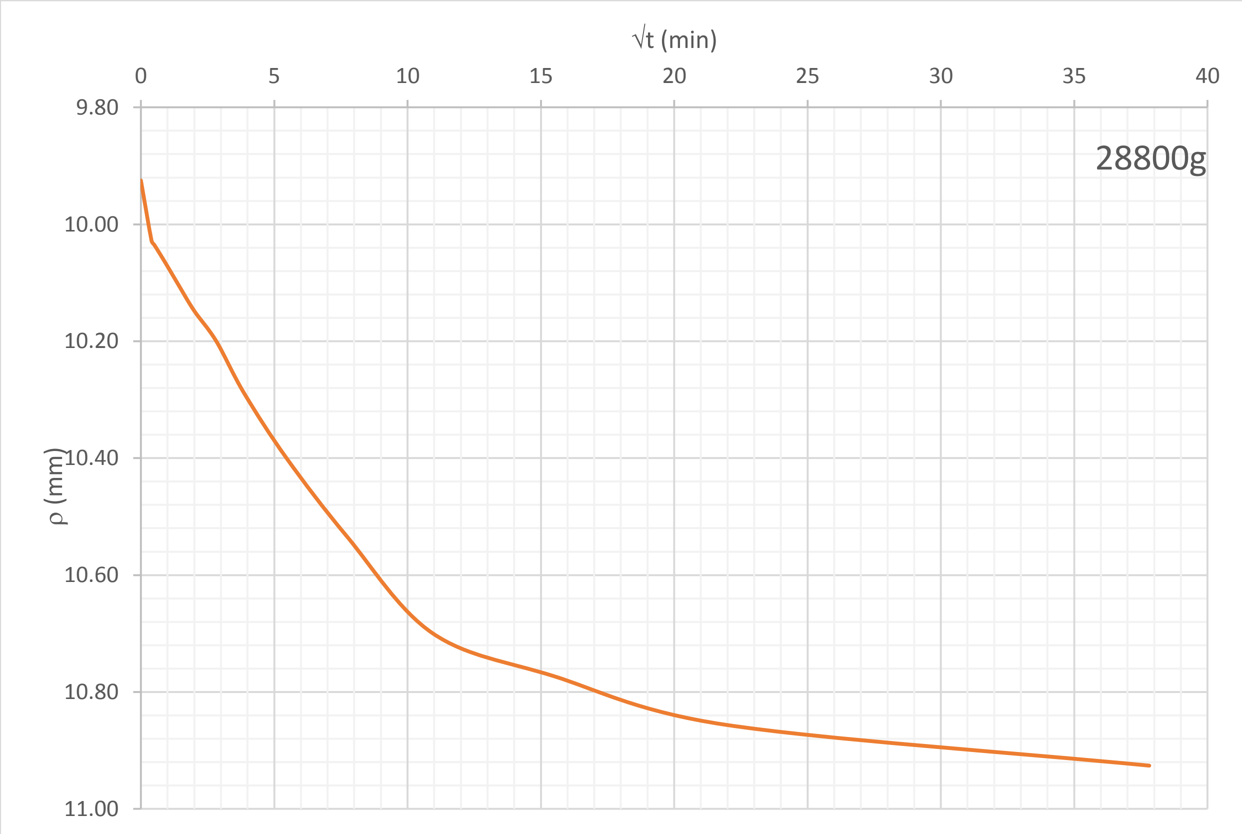
20
30
40
50
60
4.90
5.10
5.30
5.50
5.70
5.90
6.10
6.30
6.50
6.70
6.90
ρ (mm)
3600g


√t (min)
0
10
20
30
40
50
60
6.70
6.90
7.10
7.30
7.50
7.70
7.90
8.10
8.30
8.50
ρ (mm)
7200g


√t (min)
0
10
20
30
40
50
60
70
80
90
8.60
8.80
9.00
9.20
9.40
9.60
9.80
10.00
ρ (mm)
14400g


√t (min)
0
5
10
15
20
25
30
35
40
9.80
10.00
10.20
10.40
10.60
10.80
11.00
ρ (mm)
28800g