

Properties and Applications of Polymer Support Fluids in Geotechnical Engineering



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Abstract

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Synthetic polymer fluids have been used as an alternative to conventional bentonite slurries for the excavation of foundation elements over the last two decades. Thus far their use has excited polarised views in the industry – some construction professionals avoid using them after hearing ‘horror stories’ about past failures whereas others believe that they are the future of support fluids because of the many benefits that they can offer. This situation has been found to be mainly due to a lack of understanding of what polymers really are and their properties. To address this issue, the various polymer products currently available in the market have been categorised based on their intended functions, and the fundamental properties of some of them have been characterised. The research showed that significant difference exists between polymers even of the same chemical family. Based on a series of rheological experiments, it was found that the properties of polymer fluids are highly dependent on the governing shear rate, time, and a range of site dependent factors. The research also showed that the current procedure for the interpretation of viscosity data is incorrect. After an analysis of the properties of currently available polymers, the latter part of the thesis describes a field trial carried out at a site in Stratford, East London. The aims of the trial were twofold: to compare the performance between piles constructed using bentonite and polymer fluids, and to assess the effect of extended pile bore open time for pile bores supported by polymers. From the results, it was found that the polymer piles showed much stiffer load–settlement response than the bentonite pile, and that a pile bore open time of up to 26 h had no adverse effect on the pile performance. The load test results have been carefully back-analysed using a range of methods. It was found that, compared to their bentonite counterparts, polymer fluids can improve the interface shearing resistance with Thanet Sand and the shear modulus of the Lambeth clay.

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Nomenclature

Roman symbols			
$[A^-]$	molar concentration of the ionic form of acid	$[HA]$	molar concentration of acid
a	Mark-Houwink parameter	I_p	plasticity index
a	slope of tangent modulus line	J	compliance
b	y -intercept of tangent modulus line	J_0	instantaneous compliance
c'	effective cohesion	J_e	equilibrium compliance
c_{HCl}	concentration of HCl solution	K	Mark-Houwink parameter
c_p	concentration of polymer solution	K	flow consistency index
c_p^*	critical polymer concentration	K_0	coefficient of earth pressure at-rest
D_b	pile base diameter	K_a	acid dissociation constant
D_s	pile shaft diameter	pK_a	the negative log of acid dissociation constant
E_b	secant deformation modulus for soil beneath pile base at 25% of ultimate stress	K_e	effective column length of shaft transferring shaft resistance
E_c	elastic modulus of concrete	K_s	coefficient of earth pressure for the shaft
E_p	elastic modulus of pile material	k	initial gradient of load transfer curve
E_{pt}	tangent modulus of pile material	L_0	upper length of a pile carrying no load or low loads by shaft resistance
E_u	undrained Young's modulus		
f	frequency	L_f	length of a pile transferring load to the soil by shaft resistance
f_{cube}	unconfined compressive cube strength of concrete	M	molecular weight
f_{cyl}	unconfined compressive cylinder strength of concrete	M_s	flexibility factor representing movement of a pile relative to the soil when transferring load by shaft resistance
f_s	sleeve friction		
G	shear modulus		
G'	storage (elastic) modulus	m_p	mass of polymer specimen
G''	loss (viscous) modulus	N	blowcounts per 300 mm of penetration in a SPT
G^*	complex modulus	N_q^*	alternative bearing capacity factor
G_{oper}	initial operative shear modulus	n	flow behaviour index
G_{vh}	elastic shear modulus (vertical propagation, horizontal polarisation)	P_h	load at pile head
h	thickness of bentonite filter cake	P_{yy}	normal stress along y axis

P_{zz}	normal stress along z axis	ε_{cc}	concrete creep strain
p'	mean effective stress	ϕ'_{cs}	critical state angle of shearing resistance
q_b	base stress	ϕ'_p	peak angle of shearing resistance
q_{bf}	ultimate unit base resistance	γ	shear strain
q_c	cone resistance	γ_0	shear strain at pile wall
r_0	pile radius	$\dot{\gamma}$	shear (strain) rate
r_m	maximum radius of zone of influence	$\dot{\gamma}_{crit}$	critical shear rate
S_u	undrained shear strength	$[\eta]$	intrinsic viscosity
s_p	solid content of polymer	η	strain-softening parameter
t	time	η	viscosity
t_{MF}	Marsh funnel viscosity	η_0	zero-shear viscosity
t_{sol}	efflux time of solvent	η_p	plastic viscosity
U_b	ultimate base resistance	η_{rel}	relative viscosity
U_s	ultimate shaft resistance	$\ln \eta_{rel}/c_p$	inherent viscosity
u	pore water pressure	η_{sol}	viscosity of solvent
V_{HCl}	titration volume of HCl solution	η_{sp}	specific viscosity
w	local pile displacement	η_{sp}/c_p	reduced viscosity
w_b	base displacement	η_{∞}	infinite-shear viscosity
w_{bf}	base displacement at q_{bf}	Λ	retardation time in Kelvin-Voigt element
w_s	deflection of pile shaft	λ	curve-fitting parameter in Carreau model
x	number of acrylamide units in HPAM molecule	$\mu\varepsilon$	microstrain
y	number of acrylate units in HPAM molecule	ν	Poisson's ratio
Z	atomic number	ν	kinematic viscosity
z	depth	ρ	density
Greek symbols		ρ_b	bulk soil density
α	adhesion factor	σ	axial stress in pile
β	$K_s \tan \delta$	σ'_n	normal effective stress
Δ	pile head settlement	σ'_v	vertical effective stress
Δw_{res}	displacement to residual shear stress	τ	shear stress
δ	interface angle between pile and soil	τ_0	shear stress at pile wall
δ	phase angle	τ_0	yield stress in Bingham plastic and Herschel-Bulkley models
ε	axial strain in pile	τ_f	failure shear stress

τ_f	flow stress from amplitude sweep test	HPAM	partially hydrolysed polyacrylamide
τ_p	peak shear stress	LVR	linear viscoelastic region
τ_r	residual shear stress	MBI	methylene blue index
τ_y	yield stress from amplitude sweep test	ML	maintained load
ξ	yield parameter	MSDS	material safety data sheet
ζ	load transfer parameter	NSF	negative skin friction
		OD	Ordnance Datum

Abbreviations

BBGE	Balfour Beatty Ground Engineering	PAC	polyanionic cellulose
b.g.l.	below ground level	PAM	polyacrylamide
BH	exploratory borehole	PHPA	partially hydrolysed polyacrylamide
CEC	cation exchange capacity	PIT	pile integrity test
CFD	computational fluid dynamics	RH	relative humidity
CMC	carboxymethyl cellulose	SEM	scanning electron microscope
CPAM	cationic polyacrylamide	SPERW	Specification for Piling and Embedded Retaining Walls
CPT	cone penetration test	SPT	standard penetration test
CSL	cross-hole sonic logging	DVL	specified working load
CSR	cyclic stress ratio	UU	unconsolidated undrained
CTRL	Channel Tunnel Rail Link	U100	‘undisturbed’ 100-mm diameter soil sample
DEM	discrete element method		
<i>DH</i>	degree of hydrolysis		
<i>DS</i>	degree of substitution		
DVL	design verification load		
EC	electrical conductivity		
EDX	energy-dispersive X-ray		
EOR	enhanced oil recovery		
EPSRC	Engineering and Physical Sciences Research Council		
ESEM	environmental scanning electron microscope		
FHWA	Federal Highway Administration		
GPC	gel permeation chromatography		
HEC	hydroxyethyl cellulose		

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Chapter 1

Introduction

1.1 Overview

1.1.1 General Introduction

Polymer fluids refer to the aqueous solutions of polymeric compounds made up of large molecules with repeating structural units. Polymer fluids are part of our everyday life as the human body actually produces polymer fluids to help us function. For example, saliva is a natural polymer fluid, as it contains carbohydrate molecules known as mucopolysaccharides (Bhat *et al.*, 2010). These macromolecules constantly interact with each other hydrodynamically in solution and together they can give rise to some special fluid behaviour such as the stringiness and the sliminess that we all commonly observe. In the food industry, xanthan gum, which is also a polysaccharide, is commonly used as a thickening agent (food additive code E415) in the production of low-fat mayonnaise and salad dressings. Similarly in civil engineering, polymer fluids have found their ways into many different applications, ranging from the plasticizers used for concrete production to the lubricant used for priming the line of a concrete pump. For the last two decades, polymer fluids have also been used for supporting pile boreholes and diaphragm wall trenches during the excavation phase. Water-based bentonite slurries are traditionally used for this purpose, but they are now becoming less attractive due to the increasingly stringent environmental regulations and the resulting high disposal costs. Foundation engineers are therefore turning to polymer fluids which, if used properly, can offer many environmental and cost benefits. However, because the properties of polymer fluids are significantly different from their bentonite counterparts, even after two decades of application practising engineers are still struggling to master the use of this ‘new’ technique and problems occasionally occur. It is against this background that this thesis is produced. As the title suggests, this thesis is concerned with the properties

and applications of polymer fluids in geotechnical construction. Figure 1.1 illustrates schematically the stabilising action of support fluids in an excavation. Figures 1.2 and 1.3 show the application of polymer fluids on a construction site in the UK.

1.1.2 A Brief History of Polymer Support Fluids

By the mid 1970s, the use of bentonite slurries in civil engineering was established as a mature technology after two and a half decades of application (Hutchinson *et al.*, 1975; Fleming & Sliwinski, 1977). However, due to the depleting reserves of good quality Wyoming bentonite and liquid waste disposal issues, engineers at that time started to look for alternatives. A proprietary polymer product known as ‘Telmarch’ was reported to have been used in Japan for diaphragm walling in the 1970s but interest in this seems to have been short lived (Ikuta, 1974; Xanthakos, 1979). According to S. A. Jefferis (personal communication, March 2010), from that period until the early 1990s natural biodegradable polymers such as carboxymethyl cellulose (CMC), xanthan gum, and guar gum were also tried at intervals, but none of them managed to make any real impact on the industry practice. KB Technologies (2002) explain that these polymers failed because they suffer from problems such as incompatibility with concrete, high fluid loss, and cleaning difficulties. Probably due to their limited success there are only a few published papers that describe this early phase of development (Beresford *et al.*, 1989; Zapico, 1991a,b; Tallard, 1992). Although many of these natural polymers are still available in the market, they are now mainly used for other applications such as water-well drilling (Howsam & Hollamby, 1990), and construction of permeable reactive barriers and deep drainage trenches (Day & Ryan, 1992; Day *et al.*, 1999).

For the construction of structural foundations, the real breakthrough came in the mid to late 1980s, when a specific type of synthetic polymer known as partially hydrolysed polyacrylamide (HPAM or PHPA) was promoted by a small number of drilling fluid suppliers and investigated by researchers in the United States (e.g. Kheng, 1989; O’Neill, 1991). Since then the use of synthetic polymer fluids has increased considerably and the practice has spread to other parts of the world. Synthetic polymer technology has also developed to include a much wider range of chemicals in order to fulfil other functions, such as fluid loss control, accelerated particle sedimentation, and soil encapsulation.

These chemicals are commonly added as additives to the base HPAM polymer on site or pre-blended into a single product by the supplier. As a result, polymers are now often sold as a whole package of materials and described as a ‘polymer system’ since more than one chemical is used. Due to these developments, the former clear distinction between synthetic and natural polymers has become blurred since some polymer systems may include synthetic, natural, and chemically adapted natural products depending on the choice of properties required on site (Goodhue & Holmes, 1995; Goodhue & Wilkerson, 2005). Figure 1.4 summarises the key events in the development of polymer support fluid in the form of a timeline.

To give more information about HPAM, Figure 1.5 shows its chemical and empirical formulae, and Figure 1.6 shows a photograph of some granules. Figure 1.7 shows separate photographs of polymer and bentonite fluids to demonstrate the difference in their appearance.

1.1.3 Required Functions and Properties of Support Fluids

Before going any further into the details, it may be useful at this point to briefly consider the required functions of an ideal polymer support fluid. Some of these functions are listed in Table 1.1. It can be seen that, in addition to its main duty of keeping an excavation open, the ideal fluid is expected to fulfil many other functions, ranging from having a maintainable performance after repeated re-use to giving a clean concrete-soil interface at the completed pile shaft. To fulfil these functions, the polymer is expected to possess certain physical and chemical properties and these are also listed in Table 1.1. For example, as given in the first row of the table, for a fluid to adequately support an excavation, it should have certain rheological properties in order to minimise the amount of fluid loss (seepage) into the ground. The density of the fluid is also a factor because it controls the amount of the stabilising force acting on the excavation sidewall. In addition to fluid properties, there are also many non-fluid related factors that can influence the stability of an excavation, and these range from the conditions of the soil and groundwater to the design of the excavation tool. As shown in this example, the properties of a support fluid can play a key role in the success of the construction and final performance of a foundation element. One of the purposes of Table 1.1 is therefore to link these different

aspects together. The table can also be used to serve as a quick reference to indicate which parameters are useful for the current application. A fuller discussion on the required functions of an polymer support fluid can be found in Jefferis *et al.* (2011).

1.1.4 Terminology

Fluid

Since the first use of polymers in excavation support, many different terms have been used rather loosely to refer to them. The material itself has been described as ‘mud’, ‘slurry’, or ‘fluid’, and descriptive terms such as ‘drilling’, ‘boring’, ‘excavating/excavation’, ‘support’, or a combination of these, such as ‘excavation-support’, have all been used to describe its function. The term ‘mud’ is clearly an influence of the petroleum industry since a drilling-fluid engineer on an oil rig is most often known as the mud engineer (Darley & Gray, 1988). The term ‘slurry’ implies a suspension of solids in a liquid, e.g. cement slurry, and it seems to have been applied to polymer solutions due to their high viscosity. Although both of the terms, mud and slurry, can be used interchangeably to describe bentonite and other clay-based suspensions, it is incorrect to use them to describe polymer solutions since polymer solutions are obviously neither a mud nor a suspension. This leaves us with only the last choice, i.e., ‘fluid’. Regarding the descriptive terms, although most of them can be used to describe the action of digging, only the terms ‘support’ or ‘excavation-support’ can highlight the main function of the fluid, i.e., supporting an excavation. As a result, in this thesis the term ‘polymer support fluid’ (as used in the title of this thesis), ‘polymer fluid’ or simply ‘polymer’ will be used to describe the material. This decision also corresponds with the terminology of the Federation of Piling Specialists (2006) and the Institution of Civil Engineers (2007) which use the term ‘support fluids’.

Polymer

The word ‘polymer’ is used in this thesis to refer to any water-soluble polymer, and specific descriptive terms such as ‘natural’, ‘synthetic’, and ‘HPAM’ will be added as and when necessary to indicate particular polymer properties. The abbreviation HPAM is usually found in the polymer science literature, whereas the term PHPA has historically been used by writers of oil-well drilling and civil engineering articles (with some exceptions

such as Sorbie, 1991; Brown *et al.*, 2010). In this thesis, PAM will be used to abbreviate the non-ionic polyacrylamide and HPAM to represent the anionic partially hydrolysed PAM.

Geotechnical

As some readers of this thesis may not have a geotechnical engineering background, the relevant technical terms are also defined here. A foundation is the lowest and supporting layer of a structure, and a pile is a common type of deep foundation which is used to transmit the load of a structure through the upper weak soil layers to the stronger layers below. Piles may be formed from any suitable material such as concrete, steel, and wood. A bored pile is concrete cast-in-place pile, which is formed by excavating a borehole with support fluid as necessary and then inserting a reinforcing steel cage followed by pouring concrete through a placement tube. In this thesis, a ‘polymer pile’ means a bored pile constructed using polymer support fluids and similarly a ‘bentonite pile’ is one that is constructed using bentonite fluids.

When a pile is subjected to an axial compressive load, soil resistance will develop along the pile shaft and at the pile base. These forces are known as the shaft and end resistances of the pile; the sum of the two will be the total resistance of the pile. For geotechnical design and back-analysis, the term ‘unit shaft resistance’ refers to the maximum (ultimate) shaft resistance per unit contact area at a particular depth, and similarly the term ‘unit base resistance’ refers to the highest resistance per unit area of the pile base. The settlement of a pile means the downward movement of the pile head due to the applied compressive load. The word ‘displacement’ is also used for the local movement of a pile such as those occur at a local instrumentation level or at pile base.

1.2 The Driving Force behind Polymer Support Fluids

1.2.1 General

Since the introduction of polymer fluids into the foundation industry, their environmental and operational benefits have always been the driving force for the adoption of the technology. These benefits are briefly discussed below to highlight the main reasons for the change from bentonite to polymer fluids.

1.2.2 Operational Benefits

Polymer fluids can offer many benefits to the actual operations, and the most notable of them is their lower space requirement in comparison to bentonite slurries (i. e., smaller site footprint). For example, Balfour Beatty Ground Engineering (BBGE, previously known as Stent Foundations) in the UK have been using polymer fluids on many redevelopment projects in central Glasgow, Scotland since 1999. Polymers are chosen as the support fluids for these projects because, unlike bentonite, they do not require bulky and elaborate equipment for storage, mixing and cleaning (Lennon *et al.*, 2006) and the redevelopment sites in central Glasgow, as in other urban sites, are usually very small. Figure 1.8 shows a photograph of such a site in central Glasgow measuring merely 24 m by 40 m and requiring sixty-two 750 mm diameter bored piles to be installed, i. e., approximately one pile every 4 m. The size of the site and the scope of the work mean that polymer fluids were the only feasible option, as the ancillary plant required for bentonite would simply occupy too much working space. Figure 1.9 shows how polymer fluids can be mixed by simply sprinkling the granules into the mix water which is under agitation with compressed air. During the construction of the Channel Tunnel Rail Link (CTRL) East Kent–Ashford to Cheriton section (Contract 440), there were also reports that polymer fluids were chosen because setting up a bentonite plant on some sites would have been almost impossible due to space restrictions (Anonymous, 2001).

Another advantage of using polymers is the saving in time. Polymer was chosen for the CTRL project because the ancillary plant was compact and so moving it from site to site was relatively easy, whereas mobilising a full bentonite set-up for each location would have taken much valuable programme time (Anonymous, 2001). Because polymers do not require such a long hydration time as bentonite (typically 1 h for polymers versus 24 h for bentonite), further time saving can also be achieved during the preparation stage. In fact, since emulsion polymers can produce reasonable viscosity immediately after mixing with water, it is even possible to add them directly into an excavation and mix them with a piling auger. This method of mixing is therefore commonly advertised by some suppliers as another advantage over bentonite. Figure 1.10 is a photograph showing the addition of an emulsion polymer directly into a pile bore. It is interesting to note that when the

‘short piling’ scandal broke in Hong Kong in 2000, a guilty contractor was found to have exploited this exact advantage by using support fluids prepared by emulsion polymers to stabilise pile bores which should have been supported by full-length casings (Independent Commission Against Corruption, 2011). That was the very first time polymer fluids were used in Hong Kong. The first permitted use was in 2001 for the construction of the West Rail Tuen Mun Station in which the author was involved.

1.2.3 Environmental Benefits

Polymer fluids can also offer environmental benefits when compared to their clay-based counterparts. Although bentonite is classified as a non-hazardous waste under current UK legislation, it can still be highly polluting to water if it is released into the environment (NetRegs, 2011). For projects near a watercourse, polymer fluids are therefore the ideal replacement because they will not pose a danger to fish by building up on their gills and causing suffocation (Schünmann, 2004). Thasnanipan *et al.* (2003) also explain that in Bangkok the primary reason for switching from bentonites to polymers in most cases is to minimise the problems associated with the environmental issues caused by bentonite slurries.

Another environmental benefit that polymers can offer is waste reduction. Used polymer fluids can be treated on site by adding bleach (sodium hypochlorite solution) and acid to bring the viscosity and pH down to acceptable levels. With the water undertaker’s approval, the treated fluids can then be discharged directly into the sewerage system. On the other hand, used bentonite fluids have to be collected by a licensed carrier and sent to a regulated facility to be treated so that they can be disposed as solid wastes, as the European Landfill Directive banned all liquid wastes from disposal to landfill from October 2007. Finally, because polymer fluids also have the ability to ‘encapsulate’ excavated materials and retain their natural moisture contents, they need not be disposed of as wet spoil and can be re-used on site (Jefferis *et al.*, 2011), thus allowing further reduction in construction waste.

1.3 The Need for More Research

Despite the many environmental and operational benefits that polymer fluids can offer, this technology has yet to receive the attention that it deserves both from academia and industry. That is not to say there has been a total lack of research, but rather that the work that has been carried out has failed to bring their acceptance to the next level. For example, in the UK, construction of diaphragm walls is still almost exclusively carried out using bentonite, and it is not uncommon to hear from piling contractors that some clients object to the use of polymer fluids after hearing rumours about some past and often less successful polymer projects. In terms of industry guidance, the latest edition of the ICE Specification for Piling and Embedded Retaining Walls (SPERW) is totally silent on the required properties of polymer fluids, although it specifies bentonite fluids in detail (Institution of Civil Engineers, 2007). These are only a few examples of the current status of a technology which has shown a potential to completely replace bentonite fluids in the future and yet is still being rejected by some practitioners. In the author's opinion, the reason why polymer fluids have excited such polarised views is most likely due to a lack of understanding of the material properties and their effects on foundation behaviour. To illustrate this point further, two specific problems are discussed below.

‘Polymer is polymer’ – a lack of understanding of the basic properties

Unlike bentonite, which is basically a sodium or sodium-activated calcium montmorillonite clay, polymers are much more complex both in their chemical composition and in their variety (Section 1.1.2). Because chemistry is not a core subject in many civil engineering curricula and practising engineers are all busy professionals, very few engineers in the foundation industry currently understand the difference between various polymer fluids/systems and their relative strengths and weaknesses. As a result, most engineers treat all polymers alike, i. e., ‘polymer is polymer’ and just not bentonite. This lack of awareness or ignorance is undoubtedly the greatest obstacle to the further advancement of the technology, since without an understanding of what polymers are and how they behave, it is impossible to learn from our past experience and move forward. The common use of proprietary names by the polymer suppliers has certainly done nothing to help this

situation.

Case histories and research studies not related to polymer properties

Although many case histories and laboratory studies have been carried out using polymer fluids, only a handful of them pay due attention to the material properties and their effect on the results. Very often, the support fluid is only described as a ‘polymer’ without any further details. In some better cases, the name of the product, the dosage, and the fluid test results are described but only a few of them actually attempt to relate the test results to fluid properties. This practice is analogous to describing a piece of geotechnical research without considering the role of soil properties. This happens because of the first problem, i.e., most engineers think ‘polymer is polymer’. The unfortunate result is that useful experience gained from successful polymer projects often cannot be learned from, but failures due to misuse of the materials are almost always perceived as the failing of the whole technology.

To conclude this section and to emphasise again the need for more research, it is perhaps best to quote the inventor of the Marsh funnel, Hallan N. Marsh, who famously said the following in the opening statement of his paper in 1931 (Marsh, 1931):

The subject of mud sounds so simple, uninteresting and unimportant that it has failed to receive the attention that it deserves, at least as applied to the drilling of oil wells. As a matter of fact, it is one of the most complicated, technical, important and interesting subjects in connection with rotary drilling.

If we replace the words ‘mud’ with ‘polymer fluids’ and ‘oil wells’ with ‘foundations’, then what was said by Marsh eighty years ago remains very true today.

1.4 Aims of Present Research and Structure of Thesis

In response to the two key issues raised in the previous section, the aims of the present research are to:

- Characterise different types of polymer materials available on the market today;
- Investigate the effects of support fluid types and properties on the behaviour of soils and foundations.

To achieve the first research aim, Chapter 2 presents the results of some physicochemical tests which reveal the fundamental properties of the polymer materials currently available in the market. Chapter 3 then presents the results of some further tests to show the rheological properties of selected polymers. These results lay down the foundation for the interpretation of other work and are essential to the understanding of other chapters. To achieve the second aim, Chapter 4 presents the results of some full-scale pile load tests carried out at a site in Stratford, East London. These chapters are followed by some concluding remarks given in Chapter 5.

Due to the nature of this study, the materials presented in this thesis actually span several subject areas, ranging from the characterisation of polymer materials at nanoscale (polymer science) to their non-Newtonian solution properties (chemical engineering) and their effects on the behaviour of deep foundations (geotechnical engineering). To allow better presentation of the findings, a review of the relevant literature will therefore be presented in each individual chapter.

Chapter 2

Fundamental Polymer Properties

2.1 Introduction

As discussed in Section 1.1.2, the types of polymers available for foundation construction have expanded from the early days of a few biodegradable and HPAM polymers to a much wider range of chemicals, and yet many engineers still see polymers as a single product. This situation is compounded by the common use of trade marks by the polymer suppliers which give no indication of the polymer type and inconsistent advice given by some of them. To change this situation and to develop an understanding of what these chemicals really are, the available polymer technologies in the petroleum and construction industries are first reviewed in Sections 2.2.1 and 2.2.2. As some readers of this thesis may not have a strong background in chemistry, a brief description of the dissociation behaviour of polymers will also be given to aid the understanding of other parts of the thesis.

In response to the first research aim given in Section 1.4, following the review sections the similarities and differences between various polymer products will be examined. The polymers are compared in terms of their fundamental properties, including: (*a*) microscopic appearance; (*b*) chemical composition; (*c*) solid content; (*d*) solubility; (*e*) degree of hydrolysis; (*f*) molecular weight; (*g*) microstructure in solution. Note that although at first sight these properties may seem irrelevant to real practice, they are actually very important as they will allow us, for the first time, to assess and compare different polymer products in a scientific way. An understanding of these properties will also allow us to explain the behaviour of a polymer product which would otherwise not be possible.

It should be noted that, in the proposed analyses, only selected polymers from the major suppliers are tested, as it is not practicable to include all available products in a study of this kind. Additional polymers such as the natural carboxymethyl cellulose (CMC), xanthan gum, and guar gum as well as standards of synthetic (H)PAM are included in some of the analyses for comparison or method verification.

2.2 Literature Review

2.2.1 Polymer Products for Hydrocarbon Exploration

In terms of support fluid technology, civil engineers have always looked for ideas from our counterparts in the petroleum industry. According to Darley & Gray (1988), bentonite slurries were used for the first time in oil well drilling in 1928. However, it was not until 1945 that bentonite fluids were used for the construction of low-permeability cut-off walls (Kramer, 1946) and until the early 1950s for structural diaphragm walls (Veder, 1953). Similarly, polymer fluids, both natural and synthetic, have been used for oil well drilling and reservoir flooding since the early 1950s but were only introduced into construction much later (see Section 1.1.2). Therefore it is felt that it is appropriate to start this literature review first by looking at the available products in the oil industry. However, as much information can already be found in the literature (e. g. Chatterji & Borchardt, 1981; Darley & Gray, 1988; Littmann, 1988; Caenn *et al.*, 1989; Sorbie, 1991), only a brief review of the different polymer types and their uses is given.

Cellulosics

Cellulose is an insoluble organic compound which exists naturally in the primary cell wall of plants. It has the chemical formula $(C_6H_{10}O_5)_n$ and a molecular structure shown in Figure 2.1. It can be seen from the figure that each anhydroglucose unit in a cellulose molecule has three hydroxyl groups. By substituting carboxymethyl groups ($CH_2COO^-Na^+$) for some of the hydrogens of these hydroxyls, carboxymethyl cellulose (CMC) is formed. The average number of hydroxyl groups subject to substitution per anhydroglucose unit is known as the degree of substitution (DS). Although in theory all three hydrogens can be replaced, for commercial use DS typically ranges from 0.7 to 1.0 as the most desirable physical properties are usually obtained at this range (Hercules Incorporated, 1999). Figure 2.2 shows the structure of a CMC with a DS of 1.0.

Because of the presence of the carboxyl groups ($-COO^-$), CMC is soluble in water and has an anionic character. The electrostatic repulsion between the negative sites causes the randomly coiled chains to stretch linearly, and this in turn causes an increase in the solution viscosity. According to Hercules Incorporated (1999), CMC products with a DS

of 1.2 do not have viscosities as high as products of DS of 0.7 prepared in the same way, because the solubility of the latter is lower and therefore the molecules are highly swollen but not completely dispersed. The viscosity of a CMC fluid, of course, also depends on the chain length or molecular weight of the molecule. A high-viscosity CMC typically has a molecular weight of around 700,000 g/mol and a DS of between 0.7 and 0.8. For oilfield applications, CMC is commonly used as a drilling fluid additive for fluid loss or filtration control. In drilling fluid standards such as British Standards Institution (2009a) and American Petroleum Institute (2010), high purity CMC is also referred to as polyanionic cellulose (PAC).

Another commonly used cellulosic polymer is hydroxyethyl cellulose (HEC), which is formed by substituting hydroxyethyl groups ($\text{CH}_2\text{CH}_2\text{OH}$) for some of the hydrogens in the hydroxyl groups as shown in Figure 2.3. Unlike CMC, HEC is non-ionic and is therefore stable in polyvalent brines. Because of this property, HEC is often used to increase the viscosity of solids-free brines which are commonly used as completion and workover fluids¹.

Xanthan gum

Xanthan gum is a polysaccharide produced by the micro-organism *Xanthomonas campestris* in a fermentation process. Figure 2.4 shows the structure of a xanthan molecule consisting of a cellulose-like backbone and pendant side chains which are attached to every second glucose unit in the backbone. The side chains shield the backbone against enzymatic attack and give the molecule a rigid rod-like structure. This molecular structure gives rise to the many unusual properties found in xanthan solutions but not those in other flexible polymers. These include relative insensitivity to temperature, pH, mechanical shear, and ionic strength of solution (hardness, salinity). The solution of a xanthan gum also has good suspension properties and is highly shear thinning.

Xanthan gum has found many applications in oil and gas explorations. For drilling, it can be used in a potassium ‘mud’ system to prevent the swelling of shale and in other water-based fluid systems to increase the solid-carrying capacity. Because of its rheological

¹A completion fluid is a solids-free fluid placed in an oil or gas well to facilitate final operations prior to initiation of production. A workover fluid is a well-control fluid, typically a brine, that is used during workover operations such as the repair or stimulation of an existing production well (Schlumberger, 2011).

properties and resistance to high temperature (up to 135°C), the solution of xanthan gum is also widely used for flooding oil reservoirs in enhanced oil recovery (EOR) operations. Details of these uses are covered in the textbooks mentioned above in detail and are not repeated here.

Guar gum

Guar gum is a hydrophilic polysaccharide produced from the beans of guar plants. The seeds are dehusked, milled, and screened to obtain the guar gum. Chemically, it is a galactomannan with a galactose on every other mannose unit as shown in Figure 2.5. Guar gum is completely non-ionic and therefore can viscosify a wide variety of brines – a particularly useful feature when there is an absence of fresh water supply. The aqueous solution of guar gum is non-Newtonian and can be cross-linked by reagents such as sodium borate (borax) to give a high gel strength. In the oilfields, guar gum is mainly used to prepare viscous solutions (known as ‘slugging fluids’) for flushing boreholes.

Polyacrylamide

Polyacrylamide is a synthetic polymer formed from repeating acrylamide monomer units. For industrial applications, it is often used in its partially hydrolysed form with some of the acrylamide units substituted by acrylates as shown in Figure 1.5. In the literature, partially-hydrolysed polyacrylamide is commonly abbreviated to HPAM or PHPA, the latter being more frequently used in the oil industry. Because of the presence of the acrylate units, when dissolved in water HPAM has an anionic character and this can significantly increase the viscosity of the solution over a range of suitable alkaline pHs. According to Caenn *et al.* (1989), the degree of hydrolysis that gives the maximum viscosity is approximately 35%, although Rabiee *et al.* (2005) show that the peak can also occur at around 20%. Because HPAM is anionic, it is sensitive to the ionic strength of the solvent and hence water quality is important. Unlike xanthan gum, HPAM has a flexible molecular structure which can lead to a random coil conformation.

HPAM has proved to be a useful chemical for several applications in the oilfields. For example, 30-40% hydrolysed HPAM is often used in a potassium ‘mud’ system to drill in water-sensitive shale because it can help protect the walls of the hole from disintegration. This protective mechanism is thought to be due to the anionic molecules attaching

themselves to the positive sites on the surface of the shale, thus inhibiting its swelling and disintegration. Like xanthan gum, HPAM is also commonly used for EOR because of its rheological properties.

2.2.2 Polymer Products for Foundation Construction

A survey of the existing literature has been carried out to get an overview of the polymer products currently available for foundation construction. Based on what has been found, it is felt that the current situation is rather worrying for three main reasons. Firstly, it is found that the vast majority of information is supplied by the polymer suppliers and very little comes from the research community. This means that researchers have contributed very little to the development of the technology itself, and that they have only been using what they are given. This is perhaps not surprising since very few geotechnical researchers would have the interest and chemical knowledge necessary for the development of polymer products. Secondly, unlike the bentonite suppliers who commonly provide data on the basic properties of their products on their technical data sheets (e. g. water content, particle size), none of the polymer suppliers included in this survey do so. Thirdly, it is very common for the suppliers to use trade names instead of generic terms to describe their products. Although this is perfectly normal business behaviour to establish a competitive advantage, it does create a huge obstacle for those engineers who wish to understand the properties of polymer materials. Therefore, in the author's opinion, the suppliers are also partly responsible for the limited acceptance of polymer fluids despite many years of promotion, as for the vast majority of foundation engineers this technology is still no different from a 'black box'.

As a first step towards better understanding of the different polymer products, Table 2.1 summarises the products currently available in the market according to their intended functions. This table should provide a starting point for anyone who wish to understand the currently available polymer systems. It can be seen from the table that apart from a few blended products, most suppliers currently promote a base polymer plus additives system. The base polymer could be supplied as dry powder or in liquid emulsion and is typically based on the chemistry of HPAM. This is probably because HPAM is a good viscosifying agent and the solution viscosity, when unwanted at the end of a project, can

easily be removed by adding an oxidising agent (e.g. bleach). It also can be seen that one supplier still proposes the natural CMC (carboxymethyl cellulose) as the base polymer.

A wide range of additives has also been made available. Their functions include site water treatment, fluid loss control, increased excavation stability, and many more. It is interesting to see that the number of additives sold by each supplier varies considerably, from almost none to a whole range of products designed for every possible site scenario. This happens because although additives are often necessary for optimised fluid performance, they also can add to the contractors' confusion over the correct application procedure for polymer fluids. In fact, a sceptical site operative from a foundation contractor told the author that he believed that polymer suppliers encourage the use of additives only to make more profits, and that he was therefore reluctant to use them unless absolutely necessary. The author also came across the other extreme, where the site operative applied the additives liberally thinking that they would not do any harm, but in fact, as shown by Jefferis & Mavroulidou (2003), overdosing with certain additives is likely to reduce the rebar-concrete bond and can destroy the base polymer. Both ways of thinking are likely to stem from a lack of understanding of what the additives really are and their intended functions. It should be noted that not all additives shown in the Table 2.1 are polymers; some of them are simple chemicals (e.g. sodium carbonate, also known as soda ash) or clay minerals (e.g. bentonite). To provide further information about these products, Appendix A provides a list of them together with chemical information on them as found in the suppliers' literature and on the material safety data sheets (MSDS). This list should help anybody interested in this technology to gain a better understanding of the different products.

Blended polymers, i.e., products containing both base polymers and additives in a single package, have also been made available by a supplier over the last several years. It is claimed that these products eliminate the need for using different additives and pH adjustment in most cases, thus simplifying the process for site personnel. Another advancement is that one of these products, known as HydroCUT, is specifically designed for the construction of diaphragm wall using hydromills. The supplier claims that this product can keep the cutting face of the milling equipment clean by effectively encapsulating

the clay materials and preventing them from breaking down. The traditional HPAM is unlikely to be able to achieve this since the vast amount of small cuttings produced by the milling process easily could exhaust the polymer fluids.

In terms of the physical form of the products, it can be seen from the table that all suppliers sell their base HPAM polymers in dry granular form and that some suppliers also sell liquid emulsion products. The latter merit more discussion since their composition is more complex. Although many day-to-day emulsion products such as milk and vinaigrette are the oil-in-water type, emulsion polymers are the opposite since the polymer molecules are contained in numerous aqueous solution droplets dispersed in a continuous phase of oil. Therefore this type of emulsion is often called the water-in-oil, inverse, or invert emulsion. When mixed with excess water, these polymers ‘invert’ and uncoil with the help of the surfactants originally surrounding the solution droplets. Water then becomes the continuous phase to yield a dilute but viscous polymer solution. As far as the additives are concerned, they may be supplied as powder, granules, or liquid form due to their highly varied chemical composition. Those supplied in liquid form are mentioned in Appendix A but no further discussions of these products are given since they are relatively less important than the base polymers.

2.2.3 Dissociation of Polymers in Solution

As some readers of this thesis may not have a strong background in chemistry, the dissociation (ionisation) of polymers in solution is described here to aid the understanding of the following sections. To understand this topic, it is best to start the discussion with simpler non-polymeric chemicals. For example, in dilute aqueous solutions sulfuric acid (H_2SO_4) dissociates completely into positive (H^+) and negative (SO_4^-) ions because it is a strong electrolyte (acid) with a $\text{p}K_a$ of -3 , where $\text{p}K_a = -\log K_a$ and K_a is the dissociation constant of the acid. Generally speaking, an acid having a negative $\text{p}K_a$ value is regarded as strong, and one with a positive value as weak. Acetic acid (CH_3COOH) is an example of a weak electrolyte (acid) due to its relatively high $\text{p}K_a$ value of 4.76. In water, acetic acid dissociates only partially with the vast majority of the molecules actually existing in unionised form.

Polymers containing charged functional electrolyte groups are known as polyelectrolytes.

Similar to their non-polymerised counterparts, they can also be divided into ‘strong’ or ‘weak’ depending on the dissociation properties of the electrolyte groups. In the case of an HPAM, because it contains an acrylate unit (Figure 1.5) and the pK_a value of acrylic acid is 4.25, it is a weak polyelectrolyte. Therefore, when dissolved in water a significant proportion of the molecules may remain unionised. The exact degree of dissociation is controlled by the pH of the solution according to the Henderson-Hasselbalch equation:

$$pH = pK_a + \log \frac{[A^-]}{[HA]} \quad (2.1)$$

where $[A^-]$ and $[HA]$ denote the concentrations of the ionic and non-ionic forms of the acid respectively. It can be seen that this equation can be used to estimate the amount of dissociation if the pK_a of the acid and the pH of the final solution are known. It should be noted that in an unbuffered system, the final pH of the solution will depend on the pH of the solvent and the concentration of the acid added. For instance, a 1 M acetic acid solution will give a final pH of 2.4 and hence the degree of dissociation will only be 0.4%². In the case of an HPAM, because the acrylate unit is typically supplied as a sodium salt ($CH_2CHCOONa$), when dissolved in water the dissociated carboxylate anion ($-COO^-$) will cause a slight increase in the pH as the demand for protons (H^+) becomes higher. In a good solvent without the presence of other cations (e.g. due to salinity or hardness), the charges of these anions are unshielded and the polymer units and the chains will repel one another. As a result, the polymer chains become fully stretched out, causing an increase in the solution viscosity and many other interesting non-Newtonian properties – a detailed discussion on this topic will be given in Chapter 3.

With the above discussion in mind, to maximise the viscosifying power of an HPAM at a given concentration, one would need to raise the pH of the solvent to increase the degree of dissociation (Equation 2.1). This is why polymer suppliers always recommend foundation contractors to raise the pH of mix water from the natural value of around 7 to 10-11 by using either Na_2CO_3 (soda ash) or KOH (potassium hydroxide), as the degree of dissociation will in theory be increased by 1000 to 10,000 times according to the equation.

² $pK_a = 4.76$ and $pK_a = -\log(K_a) \therefore K_a = 1.7378 \times 10^{-5}$
 $[H^+] = \sqrt{K_a[HA]} = \sqrt{1.7378 \times 10^{-5} \times 1} = 4.1687 \times 10^{-3}$
 $pH = -\log[H^+] = 2.38$
 $pH = pK_a + \log([A^-]/[HA])$ (Equation 2.1) $\therefore [A^-]/[HA] = 10^{(2.38-4.76)} = 0.4\%$

However, if the result is examined more closely then it can be seen that at pH 7 the ratio of ionised to non-ionised forms of an HPAM is already in excess of 250 if calculated using a pK_a value of 4.6 (e.g. Sun, 2011 report a back-calculated pK_a range of between 4.56 and 4.65). When expressed in percent, this means that the polymer is 99.6% ionised at this pH. It can therefore be said that a high pH is not in itself a prerequisite for achieving high degree of dissociation and hence solution viscosity, but that it only serves to keep the pH in a ‘healthy’ alkaline level in case of contamination and/or dilution by acidic ground water. Another main reason for the addition of Na_2CO_3 or KOH is to remove the hardness in the water by reducing the amount of free calcium ions, which if left in the water can significantly reduce the viscosity by bridging the anionic polymers and reducing their effective volume.

Finally, it is also worth noting that because water is a polar solvent, the dissociated $-COO^-$ groups, which are also highly polar, will attract a cloud of water molecules due to hydrogen bonding and ion-dipole attractions as depicted in Figure 2.6. Similarly, the hydrogen atoms in the amide groups ($-CONH_2$) are also sufficiently positive to form hydrogen bonding with like molecules and with water as shown in Figure 2.7. These intermolecular forces, among other physicochemical effects, are thought to play a role in the reduction of swelling of water-sensitive shales as will be shown by the case histories given in Chapter 4.2.

2.3 Microscopic Appearance

2.3.1 Background

Since all dry polymers are supplied as fine granules or powder, it is not possible to identify them by naked eye, although PAM-based polymers commonly have an appearance slightly resembling that of granulated white sugar (Figure 1.6) and natural polymers are usually finer. Similarly, liquid emulsion polymers typically have a rich milky colour and are therefore not distinguishable from one another just by their appearance. An appreciation of their microscopic appearance is therefore the first step towards a better understanding of their basic material properties, and this is the aim of this section.

2.3.2 Test Method

A scanning electron microscope (SEM) was used to characterize the shape, size and surface texture of the dry polymers. The advantages of a SEM over a light microscope are that SEM micrographs have a greater depth of field and the resolution is better – the former was especially important for this work since the granules could be quite big and even the more advanced digital light microscopes can have difficulty producing a sharp image over such a large depth. To prepare the specimens for the SEM, the polymer granules/powder were first secured on a sample holder using a piece of adhesive carbon tape. Since polymer granules are nonconductive, they had to be coated with a very thin layer of gold to make their surface electrically conductive. For most samples, 5 nm of coating deposited in an ion sputterer (Hitachi E1010, at a current of 15 mA for about 60 s) was usually sufficient, but for some up to 8 nm were necessary to prevent charging of the sample, i. e., build up of excess electrons in the sample. If charging occurred, the quality of the image would be significantly reduced.

The SEM used was a Hitachi S-3400N and the images were captured using a secondary electron detector under high vacuum. A photograph of the SEM is shown at Figure 2.8. A combination of an acceleration voltage of 20 kV and a working distance (i. e., the distance between the specimen and the final condenser lens) of around 11 mm were found to produce satisfactory images for a magnification of 40 times. This magnification was used for all the samples to assist comparison between different samples. The exact values of the different imaging parameters can be found at the bottom of each micrograph. Assistance with the actual operation of the SEM was provided by a trained material scientist as specialist skills and experience are required to operate the machine.

An environmental SEM (ESEM) was also used to provide microscopic images of the emulsion polymers. The ESEM used was a Carl Zeiss Evo LS15 and the images were captured using a variable pressure secondary electron detector. A photograph of the ESEM is shown at Figure 2.9. An acceleration voltage of 10 kV and a working distance of 8 mm were used for the work. Since the emulsion samples were found to remain in a liquid state even at low vacuum (50 kPa absolute), no cooling was necessary and the samples were observed at room temperature.

2.3.3 Results and Discussion

Figures 2.10 to 2.13 show micrographs of four commercial dry granular HPAMs, namely SlurryPro CDP³, Shore Pac, POLY-BORE, and EZ MUD DP. For comparison, Figures 2.14 and 2.15 also show micrographs of two (H)PAM standards obtained from Polysciences Inc. As can be seen, these samples have similar particle sizes (0.4 to 1.2 mm) and shape. Many of them also have glassy surfaces and sharp edges which are likely to be the result of cutting during the manufacturing process. As Landolt & Allen (1979) described in their US patent, after polymerization of the monomers in water, the polymers come out of the reactor as a continuous slab of gel and then go through several stages of drying and cutting before finally appearing as fine granules. It is interesting to see that the samples of the EZ MUD DP and HPAM standard no. 1 (Figure 2.13 and 2.14) have rougher surfaces than the others. This is possibly due to slightly different manufacturing conditions or cutting methods.

For comparison, Figures 2.16 to 2.18 also show the micrographs of three natural polymers: guar gum (SC GUM), a xanthan gum (DUO-VIS PLUS NS), and a CMC (Aqualon CMC 7H3SF). It can be seen that these polymers look very different from the HPAMs. This confirms the use of microscopy, as the first means, to identify different types of polymers. Figure 2.19 also shows the micrograph of a blended polymer SlurrySMART. In this micrograph, at least five different types of particles can be found, namely, small bead, large bead, worm-like particle, flaky granule, and HPAM-like granule. This confirms that this polymer product is indeed what it says it is, that is, a blended polymer. It should be noted that the number of chemicals in the blend may be more than five, as polymers manufactured in the same way are likely to look the same.

Figures 2.20 and 2.21 show the two additives (EnhancIT 100 and EnhancIT 200) used for the Enhanced SlurryPro CDP system. This polymer system is sold by the same supplier as the blended polymer SlurrySMART. From the micrographs, it can be seen that these two additives share many of the characteristics with the blended polymer,

³Although SlurryPro CDP is marketed as a ‘Vinyl’ polymer by its supplier, it is classified as a HPAM in this study based on the description of Goodhue & Holmes (1995) and O’Neill & Reese (1999). However, it is also technically correct (but less specific) to use the term ‘vinyl’ to describe a HPAM since it is comprised of the vinyl group ($-\text{CH}=\text{CH}_2$) but with the alkene group ($\text{C}=\text{C}$) polymerized into an extended alkane chain ($\dots\text{C}-\text{C}-\text{C}-\text{C}\dots$). Several other examples of vinyl monomers or polymers can be found in Sorbie (1991).

suggesting the use of similar (types of) chemicals in these two systems. Only the additives of the Enhanced SlurryPro CDP system were tested because this polymer system is one of the most commonly used in the industry.

The imaging work on the liquid emulsion polymers was less successful. Clear presence of the water-based polymer droplets in a continuous oil phase was not seen even at very high magnifications. However, an interesting surface feature was commonly observed and it is shown in Figure 2.22. Although it is not known why individual droplets were not seen, it was possibly due to the electron beam causing damage to the samples. To illustrate this point, Figure 2.23 shows the ‘burn marks’ on the sample surface after several attempts to take images at high magnifications.

2.4 Compositional Analysis

2.4.1 Background

Although some of the dry polymers look very different under the electron microscope, HPAMs polymers are still not easily distinguishable from one another due to the similar manufacturing process. Other techniques are therefore required to show their difference and compositional (elemental) analysis is one of them. Since the SEM used was equipped with an energy-dispersive X-ray (EDX) spectrometer (Horiba EX-250), this offered an opportunity to carry out compositional analyses of the polymers as part of the SEM work.

2.4.2 Test Method

The EDX technique is based on the fact that when a high-energy electron beam hits a specimen, X-rays with energy characteristic of the chemical elements are generated within the region illuminated by the electron beam (Figure 2.24). These characteristic X-rays, together with some continuum X-rays, are then collected by a Si(Li) detector and analyzed by a computer to produce an X-ray spectrum. Although an EDX spectrometer, in theory, can detect elements ranging from boron (atomic number $Z = 5$) to uranium ($Z = 92$), it is commonly known in the field of material science that accurate analyses of light elements, such as carbon ($Z = 6$) and nitrogen ($Z = 7$), are more difficult due to low yield, absorption of low-energy X-rays, etc. Thus, due to this limitation and several

other reasons EDX results are usually regarded as qualitative or semi-quantitative.

2.4.3 Results and Discussion

Figure 2.25 shows the X-ray spectrum of one of the HPAMs. It can be seen that the X-ray peaks characteristic of carbon, oxygen, and sodium were detected and automatically labelled on the plot. Those peaks that were not identified were due to the gold coating. If this result is compared with the chemical structure of HPAMs as shown in Figure 1.5, it can be seen that both hydrogen and nitrogen are missing from the results. This is because hydrogen is not detectable by EDX and nitrogen has a relatively small weight proportion in HPAMs as well as a low Z value. In some of the analyses, nitrogen was actually detected but since the amount was less than three times the standard errors it had to be discounted according to the recommendations of the equipment manufacturer. Dragnevski *et al.* (2009) also noted the absence of a nitrogen peak in their EDX results of nitrogen-containing amides.

Table 2.2 summarises the calculated percentages by weight for all the polymers tested. For the HPAMs, it can be seen that the elemental composition are actually quite similar to each other, suggesting that they are indeed similar chemicals. Although these results are only supposed to be qualitative due to the limitations of the technique, it is useful to gauge their accuracy by comparing them with some theoretical values. If one calculates the elemental content of an HPAM with a degree of hydrolysis of, say, 40%, then one will get a weight percentage of 46% for the carbon, 10% for the sodium, and 27% for the oxygen⁴. The values for the carbon and the sodium are actually quite similar to those given in the table, but the oxygen content differs markedly from the measured values (27% versus 40-45%). This is because the above calculations do not consider the adsorbed water (H₂O) which could be as much as 15% as will be shown later in the Section 2.5. Please note that in the above discussion the degree of hydrolysis of an HPAM refers to the amount (ratio) of acrylamide units which have been hydrolysed to acrylates. Further discussions on this topic will be given in Section 2.7.

Another interesting point to note from the results is the implication of the sodium

⁴As shown in Figure 1.5, the empirical formulae for an acrylamide and a sodium acrylate are C₃H₅NO ($M = 71$ g/mol) and C₃H₃O₂Na ($M = 94$ g/mol) respectively. Therefore, the weight percentage of, say, carbon ($M = 12$ g/mol) in a 40% HPAM equals $(12 \times 3)/94 \times 0.4 + (12 \times 3)/71 \times 0.6 = 46\%$

content of the HPAMs. As the anionic charge of HPAMs is typically balanced by sodium ions (see Figure 1.6), the sodium content of an HPAM can be taken as an indication of the degree of hydrolysis of the polymer. One advantage of the EDX technique is that because it is carried out using dry samples, the sodium content measured should equal the total value and be independent of the dissociation (ionisation) behaviour when the polymer is dissolved in water (see Section 2.2.3). It can be seen from the table that if the dry HPAMs are ranked by their sodium contents, the order starting from the highest will be: Shore Pac, SlurryPro CDP, POLY-BORE, and EZ MUD DP. As will be shown later in Section 2.7, this is indeed the correct order so confirming the usefulness of this kind of quick qualitative analysis carried out as part of the SEM work.

With regard to the natural polymers, it can be seen that their composition is different from that of the HPAMs, although the sodium content of the CMC is rather similar but that is because CMCs, just like HPAMs, are typically supplied as sodium salts.

Lastly, with regard to the blended polymer ‘SlurrySMART’ and the two blended additives ‘EnhancIT 100’ and ‘EnhancIT 200’, as expected their composition is much more complicated than those of the plain polymers. Figures 2.26 to 2.28 present the corresponding SEM micrographs showing the actual particles selected for the analyses. Note that some of the particles in Figure 2.26 were ‘charged-up’ due to the longer exposure period required for the analyses and hence the lower visual quality. Increasing the coating thickness could have solved the problem but it was not done since the micrographs were only taken for supplementary information. It can be seen from the table that the composition of these polymers differs significantly from both the HPAMs and the natural polymers even though some of the particles look rather similar. For example, the particle selected in Figure 2.28c has a very different chemical composition from the HPAMs although it looks like one. This highlights the importance of the elemental analysis since without it this particle would have been mistaken as a HPAM granule when in fact it is not. Based on its appearance and the high nitrogen and chlorine contents, this particle is suspected to have been a cationic polyacrylamide (CPAM), since chlorine is usually the negative ion used to balance the positive (cationic) charge in a CPAM⁵, and nitrogen is widely present in the structure of a CPAM (British Standards Institution, 2008).

⁵Sodium chloride (NaCl) is a simple example of chlorine being the negative ion in an ionic compound.

2.5 Solid Content

2.5.1 Background

For bentonite products, it is very common to see the water content listed as one of the basic parameters on technical data sheets. However, as mentioned previously in Section 2.2.2 this is not the case for polymer products. A polymer product with too high a water content is unlikely to represent good value for money to contractors, but if the water content is too low it may take considerably longer for the granules to hydrate when mixed with water. Therefore, an understanding of the water content will help contractors compare the different products and to ensure consistent quality of the supplied materials. A knowledge of this property is also a prerequisite for the determination of the degree of hydrolysis (Section 2.7). As polymers supplied in both dry granular and liquid emulsion forms are to be analysed, the results are expressed as the solid content (s_p).

2.5.2 Test Method

To obtain the s_p of dry granular polymers, three samples of each product were oven dried for 24 h at 100°C. The s_p was calculated as the ratio of the mass of dry solids to the original (moist) mass of the sample as received. The reported s_p value is taken as the average of the three results. This test procedure is considered to exceed the requirements of the Chinese standard GB 12005.2–89 which specifies 5 h of drying at 105°C (China State Bureau of Technical Supervision, 1989b). The Chinese method is the only standard method that the author can find for the determination of s_p of HPAMs. It should be noted that the amount of water removed by this method could include both the water absorbed by the polymers and, less likely, water molecules removed from within the polymers, as it is known that acid anhydrides could be formed by dehydrating two carboxylic acid groups ($-\text{COOH}$) which are components of the HPAM structure when water is present.

A different test procedure was followed to obtain the s_p of the liquid emulsion polymers. As discussed in Section 2.2.2, polymer molecules are contained in the emulsion as numerous solution droplets dispersed in the continuous oil phase. To assess the actual amount of polymeric materials present, the solids in a known mass of emulsion are precipitated in a

beaker by addition of an organic solvent such as isopropyl alcohol or acetone (Vanderhoff & Wiley, 1966; Lin, 2004). Acetone was used by the author as it was available in the laboratories. After precipitation the solids were oven dried at about 40°C for 4 h to remove the residual acetone and then weighed. The solid content, which is also known as the active content for emulsion polymers, was determined as the ratio of the mass of the precipitates to the original mass of the emulsion. The test was repeated three times for each product and the reported value was taken as the average of the results.

2.5.3 Results and Discussion

Table 2.3 summarises the solid contents of the various polymer products. In addition to those tested in the previous section, three dry HPAM standards and two dry PAM standards were added to the list as the solid contents of these standards were required for other analyses. In the table, the test results of several other properties are also given but only the solid contents are discussed at the moment. It can be seen that the solid contents of the commercial dry HPAMs range from 86% to 93%, and those of the dry (H)PAM standards range from 85% to 95%. Although the solid contents vary, they all meet the minimum requirement of 85% as specified by Landolt & Allen (1979) and Kudomi & Sato (1984)⁶. It is interesting to note that EZ MUD DP, which has a low solid content at 86%, also has a porous structure with flaky surfaces on the granules (Figures 2.13).

The three commercial emulsion HPAMs, namely Shore Pac XX-Poly, Super Mud, and FloSet MUD VL2, were found to have a solid content ranging between 33% and 53%. These values are much lower than those of the dry polymers but are considered normal given the continuous phase of the emulsion is actually oil. In fact, this range is almost exactly the same as the typical 30% to 50% range used for enhanced oil recovery polymers as given in Caenn *et al.* (1989). It is worth mentioning that the supplier of Shore Pac XX-Poly advertises this product as having an active (solid) content of 50% claiming that this is twice that of other inverse emulsion types (CETCO Drilling Products, 2011). Although the test results has proven that the active (solid) content is indeed around 50%, it is not twice that of the other products.

⁶These authors proposed a maximum water content of 15% which is equivalent to a minimum solid content of 85%.

Regarding the solid contents of the three biodegradable polymers and of the blended polymer SlurrySMART, it can be seen in the table that they all have a moderately high solid content of between 90% and 93%.

2.6 Solubility

2.6.1 Background

High-molecular-weight polymers supplied in dry granular form are often difficult to fully dissolve in water. This is because a small degree of cross-linking between the polymer chains is usually unavoidable and the cross-links can prevent the polymer chains from separating from one another in water. This results in the formation of some partially hydrated granules which are only wetted on the outside (e.g. Sanford *et al.*, 1981; ASME Shale Shaker Committee, 2005). These undissolved materials, commonly known as ‘fish eyes’ due to their appearance, contribute little to the viscosity of the solution. Therefore they are often seen as wasted material and are undesirable, despite the fact that they may contribute to the reduction of fluid loss by physically plugging the pores on the excavation sidewalls. However, fluid loss control should not be a main goal of viscosifying polymers as there are special additives which are designed for this purpose (Table 2.1). The solubility or dispersibility of a polymer is also likely to be affected by its granule size, since the finer the granule the easier for them to break up and dissolve in the water.

2.6.2 Test Method

To test the solubility of the different polymer products, a simple test method was devised by the author. The test involved first making a solution of the polymer sample at a concentration of 1 g/L (1 kg/m³). All the polymer fluids were prepared in exactly the same way, and the mixing method will be described in detail later in Section 3.4.3. After the mixing and a rest period of about 24 h, the solution was then poured over a 1.6-mm laboratory sieve. Any ‘fish eyes’ remained on the screen were washed with a gentle stream of water and then weighed. For each polymer the test was repeated at least three times and the result was reported as the mass of ‘fish eyes’ per mass of dry material used (g/g). Beresford *et al.* (1989) used a similar method for their solubility tests but their sieve

opening was 860 μm . A 1.6-mm sieve was preferred as it is the standard screen size of the Marsh funnel (British Standards Institution, 2001; American Petroleum Institute, 2003) thus allowing the test to be carried out on site if desired. A description of the Marsh funnel will be given later in Chapter 3.3.1.

2.6.3 Results and Discussion

Table 2.3 summarises the solubility test results. It can be seen that the amount of ‘fish eyes’ varied considerably between the tested HPAMs but appeared to increase with their molecular weight. For example, the two standards which have low molecular weights, that is, HPAM standards nos. 2 and 3, did not produce any ‘fish eyes’, whereas the medium-molecular-weight HPAM standard no. 1 produced 1.1 g/g and the high-molecular-weight PAM standard no. 1 produced 4.3 g/g. These results showed that whether the polyacrylamide was hydrolysed or not did not seem to affect its solubility. EZ MUD DP, which has a published molecular weight of 10×10^6 g/mol (Liao, 1995), did not produce any ‘fish eyes’ probably due to its medium molecular weight, low solid content, and the porous surface structure of its granules. On the other hand, POLY-BORE produced 17.2 g/g probably due to its high molecular weight and a rather high solid content. Figure 2.29 shows two photographs of the ‘fish eyes’ produced by POLY-BORE.

Regarding the three natural polymers, it is interesting to see that they all dissolved completely up to the detection limit of 1.6 mm. This full dissolution is likely to be due to their different chemical structures (e.g. hydrogen bonding) and manufacturing processes. Despite their high solubility, experience has shown that natural polymers have a higher tendency to clump together during mixing. Therefore, these polymers had to be added more slowly into the water. Finally, the polymer blend SlurrySMART, which is probably a mix of both synthetic and natural polymers, only produced a small amount of ‘fish eyes’. As expected, the three emulsion HPAMs did not form any ‘fish eyes’ since the polymer materials are already wet in their original form.

2.7 Degree of Hydrolysis

2.7.1 Background

For HPAMs, the degree of hydrolysis and the molecular weight are undoubtedly the two most important parameters, because together they dictate the physicochemical properties of the polymer solution. This section is concerned the determination of the former. By definition, the degree of hydrolysis (DH) gives the percentage of acrylamide units in a polyacrylamide chain that are transformed into acrylic acid (or acrylate) units (e.g. St. Kenyeres & Ursu, 1980). If an acrylamide unit is denoted as x and an acrylate unit as y (Figure 1.5), then DH can be expressed as $y/(x + y) \times 100\%$ (Arinaitwe & Pawlik, 2009) and it will range from 0 for polyacrylamides to 100% for polyacrylic acids. Details of the chemical reactions involved in the hydrolysis of (acryl)amides can be found in many chemistry textbooks, and hence are not repeated here. However, it is worth noting that in commercial practice HPAMs are often manufactured by copolymerising chosen proportions of acrylamide and acrylate directly. In spite of this, the term ‘degree of hydrolysis’ is still used to refer to the amount of acrylate units present in the materials.

2.7.2 Test Method

According to Taylor & Nasr-El-Din (1994), the degree of hydrolysis of an HPAM can be determined by at least eight different methods, ranging from the classical acid–base titration to the more sophisticated methods such as carbon-13 nuclear magnetic resonance. After a detailed consideration, the titration method as specified by the Chinese standard GB 12005.6–89 (China State Bureau of Technical Supervision, 1989a) was used for this study. This method is chosen because it does not require any specialist instruments, is easy to carry out, and is the only method that has been incorporated into a standard so the steps are clearly listed out. Essentially, this method specifies the titration of a 100mL HPAM solution of a given concentration with a standard 0.1 M HCl solution. However, to improve the accuracy of this method, it was decided to increase the sample volume from 100 to 200 mL. For emulsion polymers, it is their solid precipitates, not the original liquid emulsion, that are used to prepare the polymer solution. This removes the possible

influence of the mineral oil on the analysis.

As discussed in Section 2.2.3, when an HPAM is dissolved in deionised water the pH is greater than 7 and the degree of dissociation is close to 100%. This justifies the assumption of complete dissociation at the start of the titration, which is necessary for the intended analyses. During the titration, the number of $-\text{COO}^-$ groups will decrease and $-\text{COOH}$ and NaCl will be formed as shown in Figure 2.30. The equivalence point of the reaction is reached when all $-\text{COO}^-$ groups become $-\text{COOH}$. With the titration volume at the equivalence point known, the degree of hydrolysis of the polymer can be calculated using the following equation given by Chen (1985):

$$DH = \frac{c_{\text{HCl}} \cdot V_{\text{HCl}} \cdot [94DH + 71(1 - DH)]}{W} \times 100\% \quad (2.2)$$

where DH is the degree of hydrolysis in percent, c_{HCl} is the concentration of the HCl solution in mol/L, V_{HCl} is the titration volume in mL at the titration end point, and W is the specimen weight in milligrams. The number 94 is the molecular mass of a sodium acrylate unit in g/mol, and the number 71 is the molecular mass of an acrylamide unit also in g/mol. The square bracket term in the equation takes into account the effect of DH on the molecular weight of the reactive monomer units. For the ease of calculation, Equation 2.2 is often rearranged to express DH as follows:

$$DH = \frac{71c_{\text{HCl}} \cdot V_{\text{HCl}}}{1000m_p \cdot s_p - 23c_{\text{HCl}} \cdot V_{\text{HCl}}} \times 100\% \quad (2.3)$$

where the parameter W in Equation 2.2 is now replaced by $1000m_p s_p$; m_p is the mass of the polymer as supplied in grams, and s_p is the solid content determined earlier in Section 2.5.

To determine the titration end point, the Chinese standard specifies the use of a mixed indicator consisting of one drop of 0.1% methyl orange solution and one drop of 0.25% indigo carmine solution. This indicator changes its colour sharply from green to grey at a pH of about 3.6-3.7 and to violet if the pH continues to decrease (Figure 2.31). The end point of the titration is determined when the colour of the indicator becomes grey. However, as it is known that the end point may not always coincide with the true equivalence point for the two reacting solutions and the result is also operator dependent

(everybody sees colours differently), it was decided to also consider the use of electrical conductivity (EC) and pH measurements as possible ways to improve the accuracy of the intended analyses.

To compare the three different methods (indicator, pH, and EC), Figure 2.32 shows the titration curves of an HPAM standard with a known DH of 70%. As can be seen in subfigure (a), the EC curve is bi-linear and the equivalence point can be taken as the point at which the two extrapolated lines intersect. Zhao (2006) also obtains a similar EC curve in his HPAM titration experiment and attributes the bi-linear shape to the following: before the equivalence point is reached the amount of free Cl^- ions in the solution would increase linearly with the amount of HCl added, but after the equivalence point there are no more $-\text{COO}^-$ groups freely available and hence the number of both H^+ and Cl^- ions would increase. Because EC is a measure of free ions in the solution, it has a bi-linear profile and the turning point is the equivalence point. Zhao (2006) suggests the use of the EC curve in replacement of the indicator method because of its higher repeatability and accuracy. In this trial experiment, it can be seen from the EC curve that the equivalence point occurs at a titration volume of 7 mL. Substituting this into Equation 2.3 would give a DH of 70%⁷, which is exactly the same as the value given by the supplier.

The second way of determining the equivalence point is by examining the pH curve, which is plotted as the solid line in Figure 2.32b. As would be expected in a titration experiment involving a strong acid and a weak base, the pH falls rather rapidly at the start but the rate slows down subsequently (see e.g. Clark, 2011). However, as the curve does not clearly show the characteristic sigmoid shape with a sudden drop in the pH near the equivalence point, a direct determination of the point location is not possible. As a result, the first derivative, $\Delta\text{pH}/\Delta V_{\text{HCl}}$, is computed for each step and plotted against the average of the two volumes, $V_{\text{HCl},1}$ and $V_{\text{HCl},2}$. This second curve is shown in Figure 2.32b, and it can be seen that the equivalence point occurs at a titration volume of ≈ 6 mL where $\Delta\text{pH}/\Delta V_{\text{HCl}}$ is the maximum. However, if this value is substituted into Equation 2.3, the calculated DH is only 58%.

Finally, as it is known that the indicator changes its colour sharply at a pH of around 3.65, the titration volume determined by the original indicator method is also shown in

⁷In this example, $c_{\text{HCl}} = 0.1 \text{ mol/L}$, $V_{\text{HCl}} = 7 \text{ mL}$, $m_p = 0.1 \text{ g}$, $s_p = 0.87$. $\therefore DH = 70.1\%$

Figure 2.32*b*. It can be seen that the V_{HCl} obtained by this method is about 6.7 mL, which lies in between the two other values determined above.

Based on the result of the above assessment, the EC method appears to be the most accurate among the three. It is also very straightforward as it only involves simple back-extrapolation of two straight lines with no differentiation required. These advantages are also noted in Zhao (2006). For these reasons, the EC measurement method is chosen for the determination of the equivalence point in other tests.

2.7.3 Results and Discussion

Table 2.3 shows the DH of the HPAM polymers. It can be seen that the measured and theoretical values of the three HPAM standards are quite similar, thus confirming the validity of the method. It can also be seen that the DH of the commercial dry HPAMs range from 26% to 39%, which lie within with the expected range of 30% to 45% given in O'Neill & Reese (1999). The measured DH values of the two products, SlurryPro CDP and EZ MUD DP, also correspond reasonably well with their theoretical values given in Goodhue & Holmes (1995) and Liao (1995), respectively.

As noted previously in Section 2.4, if the four commercial HPAMs are ranked in terms of their sodium content, the order starting from the highest will be: Shore Pac, SlurryPro CDP, POLY-BORE, and EZ MUD DP. This same ranking can again be seen in terms of their DH results. This is actually expected because, as discussed before, the degree of hydrolysis of an HPAM is a measure of the amount of sodium acrylates present in the material.

Finally, compared to the dry HPAMs, the DH of the emulsion HPAMs are generally much lower and in the range from 13% to 23%. The reason for this is not known but could be due to the fundamental difference in their manufacturing method. Since DH is a material property relevant only to HPAMs, no values are given in the table for the natural and blended polymers.

2.8 Molecular Weight

2.8.1 Background

Of all the fundamental properties of polymers, the molecular weight is undoubtedly the most important. This is because the viscosifying power of a polymer is due to the complex interaction between the polymer chains, and only chains that are long enough can produce a strong viscosifying effect. The sheer size of the chain of a typical HPAM polymer used in construction can perhaps be better appreciated by comparing its weight with those of other simple chemicals. According to O'Neill & Reese (1999), HPAM polymers used for construction typically have a molecular-weight range of between 14×10^6 and 17×10^6 g/mol. This means that these polymers on average 86,000 times heavier than glucose ($C_6H_{12}O_6$), which is a simple sugar molecule and has a molar mass of 180 g/mol. The aim of this section is to reveal the actual molecular weights of the commercial HPAMs and to test the validity of the typical weight range given by O'Neill & Reese (1999).

2.8.2 Test Method

Many methods have been developed by the polymer scientists to determine the molecular weights of polymers, including (H)PAMs. These methods include gel permeation chromatography (also known as size exclusion chromatography), ultracentrifugation, light scattering, intrinsic viscosity, and hydrodynamic chromatography (Langhorst *et al.*, 1986). Gel permeation chromatography (GPC) was originally the preferred test method because it is capable of producing the molecular-weight-distribution curve⁸ and the execution of this test was, until very recently, eligible for financial support from the EPSRC (Engineering and Physical Sciences Research Council, 2008)⁹. However, after running this test on a few initial samples, it became apparent that the method was not suitable for high-molecular-weight HPAMs due to the presence of 'fish eyes' or microgels in the test solutions and also the lack of calibrants available for molecular weights higher than 10×10^6 g/mol. As a result, after careful consideration of the remaining options the method of intrinsic viscosity

⁸The molecular weight of a polymer is not a single value but usually covers a wide range (Sperling, 2006).

⁹GPC services were provided by Smithers Rapra in Shrewsbury and were funded by the EPSRC. This scheme has now been discontinued.

was chosen due to its simplicity and long history of use in the field of polymer science. In essence, this method involves measuring the efflux times of the solvent (1.0 M NaCl water solution in this case) and the polymer solution at different concentrations (c_p) using the Ubbelohde viscometer (Figure 2.33), which is named after the German chemist Leo Ubbelohde. The efflux times are used to calculate the relative viscosity (η_{rel}) and the specific viscosity (η_{sp}), expressed as follows:

$$\eta_{\text{rel}} = \frac{\eta}{\eta_{\text{sol}}} \approx \frac{t}{t_{\text{sol}}} \quad (2.4)$$

$$\eta_{\text{sp}} = \eta_{\text{rel}} - 1 \quad (2.5)$$

where η and t are the viscosity and the efflux time of the polymer solution, and η_{sol} and t_{sol} are the viscosity and the efflux time of the solvent. The parameters η_{rel} and η_{sp} are then used to calculate two other viscosity numbers known as the reduced viscosity (η_{sp}/c_p) and the inherent viscosity ($\ln \eta_{\text{rel}}/c_p$). By plotting them against c_p , the intrinsic viscosity, $[\eta]$, is obtained from the y -intercept of the two curves which, in theory, should have the same value. Figure 2.34 illustrates this by showing a schematic plot of η_{sp}/c_p and $\ln \eta_{\text{rel}}/c_p$ versus c_p . As the principles and procedures of this test can be found in standard polymer science textbooks such as Sperling (2006) and in standards such as British Standards Institution (2009b), they are not repeated here. Once $[\eta]$ has been obtained, the molecular weight of the polymer (M) can be determined using the Mark-Houwink equation:

$$[\eta] = KM^a \quad (2.6)$$

where the parameters K and a depend on the particular polymer-solvent system and the solution temperature. For (H)PAMs, many studies have been carried out to establish their values and a summary of them can be found in Munk *et al.* (1980) and Chen *et al.* (2000). To select the most appropriate values of K and a for this study, some of them were compared with the test results of two (H)PAM standards with known molecular weights. Figure 2.36 shows the plot of the inherent and reduced viscosities ($\ln \eta_{\text{rel}}/c_p$ and η_{sp}/c_p) for one of the standards. Figure 2.37 shows the intrinsic viscosities ($[\eta]$) of the two standards and several existing Mark-Houwink correlations. It can be seen that the data points plot reasonably close to the correlation given by Ye *et al.* (1996). Therefore

it was decided to use this equation to estimate the molecular weights of the commercial HPAMs. In this equation, the values of K and a are 2.51×10^{-2} and 0.6785 respectively. It is interesting to see that the a value lies well within the typical range of 0.5 to 0.8 for polymers behaving as random coils (Sperling, 2006). For polymer chains behaving like stiff rods, the value of a could be as high as 2.

Lastly, it is also worth explaining why 1 M NaCl solutions were chosen as the solvents of the polymers. The viscosity of a polymer solution, among other factors, is affected by the molecular weight and charges of the polymer molecules. To obtain accurate molecular weight results from the intrinsic viscosity measurements, the charge effect (also known as electroviscous effect) has to be removed and this can be done by increasing the ionic strength of the solvent. Preliminary experiments have showed that the use of 1.0 M NaCl solutions can guarantee full suppression of the charge effect of various HPAMs as indicated by the reduction in their solution viscosity. Figure 2.35*a* plots the normalised viscosity values of several HPAMs tested under different NaCl concentration levels. It can be seen that overall the viscosities of the various polymer solutions drop by about 60% at high salt concentrations. This is because high sodium ion concentration leads to a shielding of the anionic charges of the HPAM molecules and hence eliminates their mutual electrostatic repulsion. As a result, the hydrodynamic volumes of the molecules become much smaller and the number of chain entanglements significantly reduces (Figure 2.35*b*). Because of the loss of the internal microstructure in the solution, the viscosity is also reduced. Further discussions about the effect of microstructure on solution viscosity will also be given in the next chapter.

2.8.3 Results and Discussion

The molecular weights of the HPAMs are summarised in Table 2.3. It can be seen that the results of the commercial dry HPAMs range from 12×10^6 to 22×10^6 g/mol, which is wider than the typical range of 14×10^6 to 17×10^6 g/mol given in O'Neill & Reese (1999). For the two commercial products (SlurryPro CDP and EZ MUD DP) which have their molecular weight values published, the experimental results also match reasonably well with the published values. As far as the emulsion HPAMs are concerned, it can be seen that their molecular weights are much lower with a range of between 3×10^6 and

8×10^6 g/mol. This is most likely due to the fact that emulsion polymers are produced by a very different process from that used to produce their granular counterparts. Details about the production technique of HPAMs are outside the scope of this thesis, but they can be found in British Standards Institution (1998) or in patents such as Vanderhoff & Wiley (1966), Landolt & Allen (1979), and Kudomi & Sato (1984).

2.9 Microstructure in solution

2.9.1 Background

The last fundamental property examined is the microstructure of the HPAM in solution, i.e., the configuration and orientation of the polymer chains and any inter-molecular interaction that exist between them. Although the degree of hydrolysis and the molecular weight are now understood, it would still be interesting to directly observe the polymer chains to gain a better understanding of how these basic properties translate into molecular structures. An understanding of the microstructure also would be useful for the interpretation of the flow behaviour of the fluid which will be discussed in the next chapter. Due to the restriction on time, only the commercial HPAM ‘Shore Pac’ was chosen for this part of the work, but the results also should be representative of other HPAM products with similar properties.

2.9.2 Test Method

The ESEM described in Section 2.3 (Carl Zeiss Evo LS15) was used to directly observe the polymer chains. An ESEM has the benefits of not requiring gold coating of the specimen as the conventional SEM does. Therefore, the microstructure of the polymer is more likely to be preserved. The sample preparation method generally followed that of Feng *et al.* (2002), as follows: a small droplet of the polymer solution was freeze-dried on a specimen holder to -15°C using a Peltier cooling system before being moved into the ESEM chamber. To reveal the polymer microstructure, the frozen ice had to be removed and this was achieved by gradually reducing the chamber pressure to about 133 Pa (1 Torr) absolute. This allowed the ice within the sample to sublime without damaging the microstructure of the polymer. The images were then acquired under the

secondary electron mode and processed by a computer.

2.9.3 Results and Discussion

Figure 2.38 shows the ESEM micrographs of the HPAM solution at a concentration of 2 kg/m^3 (0.2%). The key imaging parameters are given at the bottom of the micrographs. It can be seen that three-dimensional network structures spread throughout the specimen, slightly resembling the appearance of a bone tissue. This rigid network structure is, of course, the direct result of the freeze-drying process but what it indicates is that the polymer chains, instead of existing individually within their hydrodynamic volumes, were actually entangled with one another in the solution. This is perhaps not surprising if one considers the length of a polymer chain which is estimated to be $87\text{ }\mu\text{m}$ for this particular polymer¹⁰. If the polymer chains were not frozen, then according to Barnes (2000) the movement of the chains, under the incessant action of Brownian motion, should be like ‘snakes in a snake pit’, continuously sliding over each other, forming and disengaging from individual entanglements as they move. For comparison, Figure 2.39 presents the ESEM micrographs of a different HPAM by Feng *et al.* (2002), again showing a three-dimensional network but at concentrations of 1 and 6 kg/m^3 .

It is also interesting to note that the shape of the polymer chains, although entangled with one another, also appeared to be quite stiff. This was likely to be the result of the electrostatic repulsion between the anionic charged sites distributed along the chains. If the electrostatic charges were shielded by, say, adding salts to the solution, then the chains would be expected to be much more flexible and behave like random coils. This was shown by the low calibrated a value in the Mark-Houwink equation when 1 M NaCl solution was used to suppress the charges for the determination of molecular weight (Section 2.8).

Both the finding of this study and the work of Feng *et al.* (2002) has confirmed that HPAM polymer molecules are intertwined and entangled with each other in solution state. This finding, in fact, is not a new concept and is commonly cited as the source of the high and non-Newtonian viscosity of polymers in works such as Barnes (2000) and

¹⁰As given in Table 2.3, the commercial HPAM ‘Shore Pac’ has a molecular weight of $24 \times 10^6\text{ g/mol}$ and a degree of hydrolysis of 39%. The polymer chain is therefore composed of approximately 282,000 monomer units as the average molecular weight of each unit is about 85 g/mol . If one takes the length of a carbon-carbon single bond as 154 pm and assumes that each monomer unit has the length of two carbon-carbon bonds (Figure 1.5), the average length of a polymer chain is therefore about $87\text{ }\mu\text{m}$.

Sperling (2006). What is new is that it contradicts the results of microscopic observations made by other geotechnical engineers. For example, Figures 2.40 and 2.41 show the SEM micrographs of two different HPAMs given by Ata & O'Neill (2000) and Brown *et al.* (2010) respectively. It can be seen that these micrographs did not show a similar network structure and that the molecules appeared to exist only as individual strands. This difference is likely to be due to the sample preparation method (i.e., air-drying and coating) required for imaging under the conventional SEM. In addition, the chain lengths shown by these micrographs appear to be in the region of between 500 and 1000 μm , which are much longer than would be expected for a high-molecular-weight HPAM polymer. Since no basic information about the polymers used was provided with these micrographs, no further checks could be performed to explain these results. The suspiciously long chains and the lack of microstructure inevitably cast doubts on the representativeness of these micrographs.

2.10 Conclusions

This section has examined the fundamental properties of the polymers used for foundation construction. It begins with a comprehensive literature review of the products available in the market and a classification of the products based on their intended functions. A series of separate yet related experiments was then carried out to investigate their basic properties. These included microscopic appearance, chemical composition, solid content, solubility, degree of hydrolysis, molecular weight, and microstructure in solution. These results, for the first time, have shown that there are significant differences between the polymers even though they are nominally all the same chemical, i.e., HPAM. The results should help geotechnical engineers to appreciate that 'polymer' is not a single product, but rather a term used to represent a group of highly varied polymeric materials. The results from this section will also provide the necessary background information for the interpretation of rheological test results given in the next chapter.

Chapter 3

Rheological Polymer Properties

3.1 Introduction

Rheology is the study of the deformation and flow of materials. Most civil engineers are already familiar with the concrete slump test, in which the amount of slump is seen as an indication of the workability of the concrete. In fact, if this phenomenon is considered in rheological terms, the final amount of slump and the rate of slump loss are actually controlled by the yield stress and the plastic viscosity of the material respectively, both of which are rheological parameters commonly used to model yield-stress fluids (Ferraris & de Larrard, 1998). For grouts, a flow cone can be used for the rapid assessment of fluidity in the field (Jefferis, 2003), or a viscometer can be used to measure viscosity in a laboratory (ASTM, 2008). For polymer support fluids, it is just as, or even more, important to understand the rheological properties because it is the complex flow behaviour of these fluids that allows them to stabilise many sandy formations. For instance, Troughton (1992) noted that the use of water as a support fluid could not stabilise the Thanet Sand at Canary Wharf, but recent experience has shown that polymer fluids, which have the same density as water but different rheological properties, can adequately support this formation. The rheological properties of support fluids can also significantly affect the quality and behaviour of foundation elements. An example is the amount of soft sediment accumulated at the base of an excavation, which is controlled by the particle-holding ability of the support fluid.

In response to the needs for further research identified in Chapter 1 and above, the shear viscosity and viscoelasticity of various support fluids have been studied and the results presented in this chapter. Due to the limitation on time, the extensional viscosity of the fluids was not studied although it is also relevant to fluid flow in porous media. This decision might be justified by the fact that the extensional viscosity effect only is only significant at high flow rates (Chauveteau, 1981), but the shear rates applicable to

geotechnical problems are typically very low.

3.2 Basic Non-Newtonian Flow Behaviour

Before the work of other researchers and the results of the current study can be discussed, it is necessary to introduce a few basic concepts about non-Newtonian flow behaviour, as it is not a topic commonly discussed in the civil engineering literature. By definition, a Newtonian fluid is a fluid whose viscosity is a constant, irrespective of the forces acting on it for a given temperature and pressure. Therefore, the Newton's law for fluids can be written as:

$$\tau = \eta \dot{\gamma} \quad (3.1)$$

where η is the viscosity in $\text{Pa} \cdot \text{s}$, τ is the shear stress in Pa, and $\dot{\gamma}$ is the shear (strain) rate in s^{-1} . Note that in some publications the Greek letter ' μ ' is used instead of ' η ' to denote viscosity, but the latter is preferred in this thesis. Water is a Newtonian fluid and it has a (dynamic) viscosity of 1 cP at 20°C. The unit 'cP' stands for centipoise and is numerically equivalent to ' $\text{mPa} \cdot \text{s}$ '. Fluids that do not show Newtonian behaviour are collectively known as non-Newtonian fluids. Figure 3.1 shows some different types of non-Newtonian behaviour in a schematic $\tau - \dot{\gamma}$ plot. The types of fluid shown and some of their real-life examples are: shear-thickening (starch suspension), shear-thinning (polymer solutions and melts), Bingham plastic (clay slurries, fresh concrete), and yield-pseudoplastic (paint, ketchup). Since the $\tau - \dot{\gamma}$ curves of these fluids are no longer straight lines passing through the origin, their viscosity is sometimes described as 'apparent' or 'effective', but these terms are omitted in this thesis as in many other publications. As it is known that polymer fluids commonly display shear-thinning behaviour, i.e., a reduction in viscosity as the shear rate increases, this particular fluid type is illustrated further in Figure 3.2, which shows the viscosity and shear stress curves (also known as flow curve, consistency curve, or rheogram) over a wide range of shear rates from 10^{-12} to 10^5 s^{-1} on a logarithmic scale. The figure shows that although the shear-thinning behaviour is evident over the middle portion of the curve, at very high and low shear rates the behaviour reverts back towards Newtonian, as signalled by the slope of the curve reducing to zero. The limiting viscosities at very low and high shear rates are known as zero-shear viscosity (η_0) and infinite-shear viscosity (η_∞) respectively. Since the middle part of the curve is a straight

line on a log-log scale, it can be described by the power-law model:

$$\tau = K\dot{\gamma}^n \quad (3.2)$$

where K is the flow consistency index in $\text{Pa} \cdot \text{s}^n$ and n is the flow behaviour index (dimensionless). For shear-thinning fluids, n is less than one. Similarly, for shear-thickening fluids, n is larger than one. If n equals one, then the equation then reverts back to the Newtonian form. To model the complete flow curve of shear-thinning materials, the Cross or Carreau models are commonly used, as they incorporate η_0 and η_∞ in their formulation (e.g. Mezger, 2006). The equation of the Carreau model is:

$$\frac{\eta - \eta_\infty}{\eta_0 - \eta_\infty} = \{1 + (\lambda\dot{\gamma})^2\}^{(n-1)/2} \quad (3.3)$$

where n (< 1) is the flow behaviour index and λ is a time constant in s. λ is called a time constant because its reciprocal value (in s^{-1} , i.e., the unit of $\dot{\gamma}$) represents the transition between the Newtonian plateau at very low shear and the shear-thinning region.

To complete this brief discussion of fluid behaviour, the one-dimensional equations for the Bingham plastic and the yield-pseudoplastic (Herschel-Bulkley) models are given in Equations 3.4 and 3.5 respectively as follows:

$$\tau = \tau_0 + \eta_p \dot{\gamma} \quad \text{for } \tau > \tau_0 \quad (3.4)$$

$$\tau = \tau_0 + K\dot{\gamma}^n \quad \text{for } \tau > \tau_0 \quad (3.5)$$

where τ_0 is the yield stress and η_p is the plastic viscosity, and K and n are again the power-law parameters. Note that in the oil industry and in some of the civil engineering literature (e.g. Ata & O'Neill, 1997a; Brown *et al.*, 2010), the Bingham plastic parameters τ_0 and η_p are commonly denoted as YP and PV respectively. When this naming convention is used, the (apparent) viscosity is also commonly referred to as AV (especially at 600 r/min as measured by the Fann viscometer). This naming practice is not adopted in this thesis as it is inconsistent with the nomenclature of other parameters.

3.3 Test Instruments

3.3.1 Basic Descriptions

To aid the understanding of the following sections, the basic operating principles of the various test instruments used for this research are briefly described here. The instruments include: (a) Marsh funnel; (b) Fann viscometer; (c) Brookfield viscometer; (d) Bohlin Gemini rheometer. In essence, the Marsh funnel is the simplest of all and is used to give a rough indication of the fluid viscosity. The Fann and Brookfield viscometers, on the other hand, are capable of measuring viscosity in the true sense and operate over a range of shear rates. The Bohlin Gemini rheometer is the most sophisticated of these instruments and can measure both viscosity and viscoelasticity. A more detailed description of each of these instruments is given as follows.

Marsh funnel

Among all the test instruments, the Marsh funnel is undoubtedly the most popular tool for measuring the viscosity of support fluids. This is reflected by the fact that support-fluid specifications commonly set out the limiting viscosity values based on the result of this test (Federation of Piling Specialists, 1975, 2006; Institution of Civil Engineers, 2007; Brown *et al.*, 2010). As mentioned in Section 1.3, this instrument was proposed in 1931 by H. N. Marsh for testing clay-based drilling fluids (Marsh, 1931) and was subsequently adopted by the civil engineers for testing support fluids. Figure 3.3 shows the Marsh funnel and the accompanying measuring cup. This test is popular because it is very simple and does not require electricity to operate. To run this test, the operator only needs to fill the funnel with the test fluid to the level of the screen at the top (i.e., 1.5 L) and then measure the efflux time for 946 mL (one US quart) to flow out of the tip. The time in seconds is recorded as the Marsh funnel viscosity (t_{MF}), but strictly speaking it is not a viscosity measurement as $\dot{\gamma}$ is not defined. In the literature, the Marsh funnel viscosity is sometimes abbreviated to MFV, but t_{MF} is the preferred symbol in this thesis as it is a time measurement.

Fann viscometer

Another common instrument is the Fann viscometer, which is named after its manufacturer Fann Instrument Company, USA. In standard textbooks or national specifications, it is commonly referred to as the direct-indicating or concentric-cylinder viscometer (British Standards Institution, 2001; American Petroleum Institute, 2003). Just like the Marsh funnel, this instrument was also ‘borrowed’ from the oil industry and its use has been recommended by some civil engineering practitioners as a more accurate way of fluid testing (e.g. Hutchinson *et al.*, 1975; Hodgson, 1977; Beresford *et al.*, 1989). The Fann viscometer is currently sold in three different models: two-speed, six-speed, and variable-speed. Regardless of the model, they all share the same concentric-cylinder design. Figure 3.4 shows a photograph of the variable-speed model which can operate at any rotational speed between 3 and 600 r/min. With the standard bob and rotor geometry combination (R1-B1) this is equivalent to a shear-rate range of between 5 and 1022 s⁻¹ using a conversion factor of 1.703 (American Petroleum Institute, 2006). Figure 3.5 shows a schematic diagram of its key components.

When the instrument is switched on, the rotor (the outer cylinder) turns at a constant angular speed and the torque experienced by the bob (the inner cylinder) is counter-balanced by and measured against a torsion spring (standard spring constant 386 dyne-cm/degree deflection) mounted on the bob shaft. The shear stress (τ) in lb/100 ft² is read through a magnifying lens by the operator (note: 1 lb/100 ft² equals 0.4788 Pa). Since this instrument was originally developed by Savins & Roper (1954) to measure the viscosity of clay-based drilling fluids for oil well drilling, it allows the two Bingham plastic parameters τ_0 and η_p (Equation 3.4) as well as η at 600 r/min (i.e., the so-called YP, PV, and AV) to be calculated by using only the stress readings at 300 and 600 r/min and some simple approximate equations (Darley & Gray, 1988; British Standards Institution, 2001). Similar ‘quick-and-dirty’ expressions have also been constructed to facilitate the calculations of n and K for power-law fluids using the stress readings at 300 and 600 r/min without going through the curve fitting process (Darley & Gray, 1988; American Petroleum Institute, 2006).

Brookfield viscometer

The Brookfield viscometer, named after its designer Don Brookfield, Sr. (now marketed by Brookfield Engineering Laboratories Inc., USA), is another common instrument for measuring fluid viscosity. It is commonly used as a ‘shop floor’ viscometer to check the quality of industrial fluid products such as lubricants, resins, polymers, and adhesives. However, its use for civil engineering applications has been rather limited and typically it can only be found in the testing of chemical grouts (ASTM, 2008) and clay slurries (Abichou *et al.*, 2004). Figure 3.6 shows a photograph of the Brookfield viscometer (model LVDV-II+, spring torque 0.0673 mN·m) used for this research. Unlike the Fann viscometer, it does not require the test fluid to be confined between two cylinders. Instead, it assumes the spindle (i.e., the rotating cylinder) to be immersed in a sea of liquid at an infinite boundary. In practice, however, a 600-mL low form beaker such as the one shown in the photograph will satisfy this condition. More theoretical background about the assumption of ‘infinite sea’ can be found in rheology textbooks such as Chhabra & Richardson (1999). When equipped with one of the standard spindles, LV1, this particular model can operate over a shear-rate range of between 0.066 and 22 s⁻¹, corresponding to a rotational speed of between 0.3 and 100 r/min. If used with special accessories such as the low-shear UL adaptor (Figure 3.7), this viscometer can measure viscosity as low as that of water, i.e., 1 cP, with only 16 mL of test fluid.

Bohlin Gemini rheometer

The last and most sophisticated test instrument used for this research is the Bohlin Gemini rheometer (Figure 3.8), currently marketed by Malvern Instruments Ltd, UK. In contrast to the Fann and the Brookfield viscometers, which can only measure the shear stress at a given shear rate, this rheometer can independently control both the shear stress and the shear rate. Therefore, it can perform a wide range of rheological experiments such as viscometry (including yield stress and thixotropy), oscillation, creep, and relaxation. The instrument is fitted with a Peltier plate which allows accurate temperature control of the sample between -30°C and 200°C. Because of its wide range of capabilities, it is called a ‘rheometer’ rather than a ‘viscometer’.

For viscometry experiments, the cone-and-plate geometry is commonly used, as the

shear rate is constant over the contact surface. Figure 3.9 shows a photograph of a polymer fluid sample being tested for its viscosity. For oscillatory (dynamic) experiments, the parallel-plate geometry is commonly used because the gap, i.e., the thickness of the sample, can be varied to suit the sample and shear rate needs. These two types of measurement geometries are shown in Figure 3.10. Due to the use of an air bearing as the core of the measurement unit (hidden behind the silver shaft in Figure 3.8), this rheometer has a very wide torque range between $0.05\mu\text{N}\cdot\text{m}$ and $200\text{mN}\cdot\text{m}$. This enables the viscosity of the sample to be determined at very low shear rates when the measured torque is also very low.

3.3.2 Calibrations

The instruments mentioned in the previous section were checked for their accuracy before use. The Marsh funnel only requires minimal checking, as it is known that when water is used t_{MF} should be $26\pm0.5\text{s}$. This was found to be the case with the author's funnel as it was brand new. To check the accuracy of the Fann and the Brookfield viscometers, silicone oils with known kinematic viscosities (ν) of 100 and 1000 cS were obtained from Basildon Chemicals Company Ltd. and used as calibration fluids (note the unit 'cS' stands for 'centistoke' and is a common unit for expressing kinematic viscosity ν ; 1 cS equals $1 \times 10^{-6}\text{m}^2/\text{s}$). Since $\eta = \nu\rho$ and the density (ρ) of the 100-cS and 1000-cS fluids are 0.968 and $0.974\text{g}/\text{cm}^3$ at 25°C respectively, the (dynamic) viscosities of these two fluids are 96.8cP and 976cP . Figures 3.11 and 3.12 show the calibration results for the two viscometers.

As far as the performance of the Fann viscometer is concerned, the measured values were generally in good agreement with the expected values, and the errors were usually no more than 3% in both cases. However, when operating at the lowest rotational speed of $3\text{r}/\text{min}$ (5.1s^{-1}), the viscometer gave a rather large error of about 7% again in both cases. This is probably due to the fact that the dial readings are only accurate to $1.0\text{lb}/100\text{ft}^2$ (0.4788Pa) even though additional effort was made to read them to the nearest $0.5\text{lb}/100\text{ft}^2$. Due to this limitation of the instrument, at low shear rates the errors of the calculated viscosity could become large since both τ and $\dot{\gamma}$ are low under such conditions (note $\eta = \tau/\dot{\gamma}$).

With regard to performance of the Brookfield viscometer, the results are slightly different between the two plots. For the 100-cS calibration, the errors were generally within the 3% error band when the measured torques were higher than 10% of the full-scale torque, which is $0.0673 \text{ mN} \cdot \text{m}$ for this particular viscometer. However, when the torque went below 10% at low shear rates, the error then became significant. This sort of behaviour is actually expected as it ties in with the manufacturer's own suggestion that the torque should be above 10% (Brookfield Engineering Laboratories, 2005). However, the 1000-cS calibration result seems to suggest that this condition could be relaxed for high-viscosity fluids as the errors remained below 3% even when the torque was below 10% of full scale. Despite this apparent improvement with high-viscosity fluids, during the actual tests the torque was always kept at above 10%. Where necessary a larger geometry (spindle) was used to improve the signal.

The Bohlin rheometer was loaned from the EPSRC Engineering Instrument Pool for a total period of six months. It was delivered to the author's laboratory and installed by a trained EPSRC technician. Upon installation, the technician carried out a procedure known as 'torque mapping' to calibrate the torque at multiple positions around the air bearing. This was done to ensure the accuracy of the viscosity measurements even at very low torques. Since the rheometer also came with a full calibration history by the manufacturer (Malvern Instruments Ltd), no further checks were deemed necessary.

3.4 Flow Behaviour

3.4.1 Literature Review

A literature review is given here to show the current understanding and practice related to the viscosity measurement of polymer support fluids in civil engineering. In terms of the amount of information given, the literature can be divided into three main categories: *a*) no information about the fluid viscosity is given; *b*) a Marsh funnel was used to determine the 'viscosity'; *c*) a Fann viscometer was used to generate a flow curve. There are only a small number of studies in the first category, such as Hu *et al.* (2006) and Thasnanipan *et al.* (2006). Although it is not known why these authors did not present any viscosity data, it is probably because they did not think it was important for the subject matter

which was, rather ironically, the effect of the support fluid on soil or pile behaviour. In contrast, a large number of studies, up to about 70% of the total, fall under the second category. Some of the examples are Holmes (1993), Ho & Tan (1996), Littlechild & Plumbridge (1998), Camp *et al.* (2002), and Brown (2002). These studies are usually field-based and this explains why the Marsh funnel was the preferred instrument. The last category, on the other hand, consist of studies which are mainly laboratory-based and hence detailed investigations of the fluid behaviour could be performed. These studies include Beresford *et al.* (1989), Howsam & Hollamby (1990), Kheng *et al.* (1991), Majano & O'Neill (1993), Majano *et al.* (1994), Ata & O'Neill (1997a), and Ata & O'Neill (1997b). Despite what has been found, it should be noted that testing support fluids using the Fann viscometer is perfectly practicable in the field, as it has traditionally been done for oil and gas drilling.

Amongst those studies which included measurements using the Fann viscometer, treatment of the test data was found to be quite different. Authors such as Beresford *et al.* (1989) and Howsam & Hollamby (1990) believed that the power-law model was the most suitable for polymer fluids, and hence the flow behaviour can be best described by the two power-law parameters n , and K . On the other hand, the research group led by the late Prof Michael O'Neill at the University of Houston adopted a more moderate view and interpreted their viscometer test data using both power-law and Bingham plastic models (Majano & O'Neill, 1993; Ata & O'Neill, 1997a). However, when it came to the actual presentation of the results, it becomes apparent that they preferred the Bingham plastic parameters as can be seen in Majano *et al.* (1994) and Ata & O'Neill (2000). This approach to data presentation seems to have been carried over from these publications to the last two versions of the FHWA (Federal Highway Administration) drilled shaft manual (O'Neill & Reese, 1999; Brown *et al.*, 2010). Since the original thinking process was not given in the manual, it would appear to a typical reader that the correct way of interpreting the Fann viscometer data is to assume the Bingham plastic model (i. e., to calculate the so-called PV, YP, and AV) even though the example curve given in the manual clearly shows strong shear-thinning behaviour, with a negligible yield stress.

3.4.2 Preliminary Analysis

As shown in the previous section, so far there is no consensus in the civil engineering literature as to which mathematical model is best suited to interpret the viscometer data of polymer fluids. Therefore, a set of simple test data was used to assess the suitability of the different models. The fluid sample used was obtained from a construction project in the UK where the polymer ‘SlurryPro CDP’ was used as the support fluid. The polymer concentration was 0.8 kg/m^3 and the pH was 11. This particular project will be discussed further in Section 3.4.7. Figure 3.4.2a shows the Fann viscometer test results presented as a shear stress versus strain rate (τ – $\dot{\gamma}$) plot. The viscometer readings were taken at 3, 6, 100, 200, 300, and 600 r/min ($\times 1.703$ for s^{-1}) as these are the speed settings on the common six-speed model. In addition to the power-law model (Equation 3.2) and the Bingham plastic model (Equation 3.4), the Herschel-Bulkley model (Equation 3.5) was also included in the comparison as it is effectively a combination of the two. From the figure, it can be seen that all three models are able to represent the test data reasonably well, with the power-law and the Herschel-Bulkley curves effectively lying on top of each other. Given that the Bingham plastic line was only drawn through the two data points at 300 and 600 r/min without going through any regression analysis (this is a practice inherited from the oil industry and a feature of the instrument as discussed previously), it may appear to be the preferable choice of curve-fitting model in many situations. However, as it is the viscosity of the fluid that actually controls the flow behaviour, it may be more appropriate to evaluate the test data on a viscosity-shear rate (η – $\dot{\gamma}$) plot as shown in Figure 3.15b.

When the results are expressed in terms of viscosity rather than shear stress, the comparison looks very different. It can be seen that the Bingham plastic model significantly overestimates the viscosity of the fluid at low shear rates. For instance, at the shear rate of 5.1 s^{-1} (3 r/min on the Fann viscometer), the measured viscosity is 47 cP (mPa·s) whilst the Bingham plastic model gives 195 cP. Given that it is the viscosity at low shear that governs many of the soil-fluid interactions (e. g. settling of soil particles, fluid flow through soil pores), the Bingham plastic model therefore does not appear to be a suitable model for polymer fluids. On the other hand, the power-law model shows a very good

fit to the test data over this range of shear rates. Although the data also can be fitted to the Herschel-Bulkley model, this is at the expense of an extra yield stress parameter. The choice between the Herschel-Bulkley and the power-law models therefore hinges on whether polymer fluids have a yield stress – this topic will be explored in the following sections. On the plot, the approximate shear rates of the Marsh funnel and typical pumping (as on a construction site) are also given as additional information (see Pitt, 2000 for the estimation of shear rates for the Marsh funnel). It can be seen that the condition of the fluid in a Marsh funnel is actually closer to that during pumping than any soil-fluid interactions that are of interest. This raises the question as to whether the Marsh funnel is a suitable tool for measuring the viscosity of support fluids, whether that be polymer or bentonite. Further discussions on this topic will also be given in the following sections.

The foregoing analysis has shown that the Bingham plastic model is clearly unsuitable for polymer fluids if one considers the fitting of data on a viscosity plot. On the other hand, the power-law model has been found to be a suitable model over the shear-rate range measurable on a Fann viscometer. In the following section, the use of the Brookfield viscometer and the Bohlin rheometer to assess the validity of this conclusion over a wider range of shear rates and on other polymer products is discussed.

3.4.3 Further Analyses and Difference between Polymer Products

Background

To provide further insights into the flow behaviour of polymer fluids and to assess the difference between polymer products, work was carried out using the Bohlin rheometer to extend the low shear-rate range from the Fann's lower limit of 5.1 s^{-1} down to about 10^{-3} s^{-1} . Since the shear-rate range of the Brookfield viscometer (0.07 to 100 s^{-1}) is well within that of the Bohlin rheometer, the former was only used to check the results of the latter. In a similar fashion to the work presented in the previous chapter, selected polymer products from each category (dry HPAM, emulsion HPAM, natural polymer, blended polymer) were tested to assess the difference between them. To facilitate a comparison

between them and yet adhering to what would be expected in real practice, polymers from each category were mixed at a concentration typical of that category, i. e., 0.8 kg/m^3 for dry HPAMs, 2.0 kg/m^3 for emulsion HPAMs, 3.0 kg/m^3 for natural polymers, and 2.0 kg/m^3 for blended polymers. Since natural polymers are biodegradable and the testing period for each fluid could last for several days, a tiny amount of biocide (sodium azide) was added to the natural polymer fluids to prevent bacteria growth.

Test equipment and sample preparation

With regard to sample preparation, all the polymers were mixed by a laboratory mixer (Lightnin model L1U08F) equipped with a low-shear axial-flow impeller (Figures 3.13 and 3.14). The mix water was deionised water. To prevent clumping of the granules during addition, the mixing speed was initially set at 600 r/min to help disperse the granules in the water during the first minute. The speed was reduced to 200 r/min immediately afterwards and the mixer was left running for another 40 min to help dissolve the polymers. As shown in Figure 3.13, to achieve strong top-to-bottom turnover, the rotating shaft was tilted at an angle of about 30° from vertical. Although the energy consumption readings on the mixer showed that a mixing time of 20 min was probably enough for most polymers (i. e., no change in power), a longer mixing time was used to ensure complete dissolution of the granules and hence better consistency between the different samples. As a precaution, all the fluids were also used within three days to prevent the possible effect of longer ageing.

With regard to the rheometer settings, a cone-and-plate geometry was used (Figure 3.9) as it produces uniform shear rate across the width of the specimen. The cone used has a diameter of 40 mm and an angle of 4° . The viscosity readings were taken under the steady-state equilibrium condition which may require up to 20 min to achieve at very low shear rates. As viscosity is known to be temperature dependent, all the tests were carried out at a common temperature of 25°C .

Overview of test results

Figure 3.16 shows the viscosity results plotted against shear rate on a log-log scale, which is the standard way for presenting this kind of data. To evaluate the test data based on the existing rheological framework, the Carreau model (Equation 3.3) was used to fit the

data and the best-fit curves are also shown on the plot. Since none of the flow curves clearly entered the infinite-shear region, the value of the infinite-shear viscosity could not be determined automatically and hence the initial curve-fitting results (from a commercial non-linear solver) were manually adjusted to give the best overall visual fit. Table 3.1 presents the four Carreau parameters as well as the Marsh funnel and Fann viscometer readings for comparison. As an additional element of information, the useable shear-rate ranges of the three test instruments are also annotated on the plot.

The results are very interesting. Compared to what previously has been seen in the Section 3.4.2, these curves now show the ‘bigger picture’ and tell a lot more about the overall flow behaviour of the fluids. For example, it can be said that all polymer products show signs of approaching to their limiting viscosities at very low (10^{-3} s^{-1}) and at very high (10^3 s^{-1}) shear rates. This behaviour, although not revealed previously by the Fann viscometer, is actually expected of most shear-thinning fluids as already shown in Figure 3.2. It also becomes clear that at 600 r/min the shear rate of the Fann viscometer (1022 s^{-1}) was so high that most of the fluids were already approaching their infinite-shear viscosity. Capillary viscometers, which are not described in this thesis, could have been used to extend these curves to shear rates of above 10^4 s^{-1} . This was not done because it is the low to middle shear viscosity that is relevant to most soil–fluid interaction problems. The overall shape of the flow curves have also shown that these polymer fluids, regardless of their type, do not possess a yield stress (i. e., $\eta = \infty$) as otherwise they would have shown an increasingly upward trend towards low shear rates and would not have shown the Newtonian plateau at very low shear. This again confirms the previous conclusion that the Bingham plastic model is unsuitable for polymer fluids.

Before the flow curves of the individual polymer products are compared, it is useful to discuss the reasons behind the shear-thinning behaviour exhibited by all these polymers. At the foot of Figure 3.16 is a diagram showing the schematic molecular arrangement of polymer fluids in three viscosity regions, i. e., zero-shear viscosity (η_0), shear-thinning, and infinite-shear viscosity (η_∞). When the polymer fluids are under static or near static conditions, the long-chain molecules are completely intertwined and entangled as discussed in Section 2.9. Since entanglement resists flow and viscosity represents the resistance to

flow, at this stage the viscosity is at its highest on the Newtonian plateau of η_0 . When the fluid is subject to increasing shear, the number of entanglements reduces as the chains orient along the lines of flow. The fluid therefore shows a gradual reduction in viscosity over the middle shear-thinning region. At very high shear rates, all the entanglements disappear as the flow ‘combs’ them out and hence the dependence on molecular weight (which dictates the number of entanglement) also disappears. At this stage, the viscosity is only dependent on the polymer concentration and any further increase in shear rates will not have any further effects; the viscosity is therefore at its lowest on the Newtonian plateau of η_∞ . It should be noted that the above discussion assumes that, under (near) static conditions, the polymer has an intertwined three-dimensional molecular structure as shown in Figure 2.38. This will not be the case if the pH is low and (or) the ionic strength of the solvent is high, because the electrostatic repulsion between the molecules and hence the network structure would be weakened or completely lost (see Figure 2.35*b*). For this reason, if the test polymers are dissolved in a solvent with less favourable pH and ionic conditions, the viscosity will be lower than what is shown in Figure 3.16. Discussions about the effects of pH and the ionic strength of solvent can be found in Sections 2.2.3 and 2.8.2.

Detailed discussion of test results – flow curves

With regard to the performance of individual polymer products, it can be seen that both the highest and the lowest flow curves are given by the natural polymers. This shows that products in this category can indeed behave very differently depending on their sources, e. g. plants (cellulose), seeds (guar gum), microbial (xanthan gum). The highest flow curve was found to be given by the natural polymer ‘DUO-VIS PLUS NS’ which is a xanthan gum. This result is actually not surprising because xanthan gum fluids have long been known for their high viscosity at low shear and have been utilized for a wide range of applications such as in food products (Section 1.1.1) and enhanced oil recovery (Section 2.2.1). In fact, xanthan was also proposed for foundation construction in the late 1980s – the commercial polymer ‘Viskeau’, which has now been discontinued, is believed to have been a xanthan gum although this was never made public in the relevant literature (Beresford *et al.*, 1989; White Jefferis & Associates, 1989). The exact reason why this

product failed is not known but the flow curve given in the figure suggests that this kind of fluid, if used for foundation excavation, would powerfully suspend clay and silt particles due to its extremely high η_0 which is more than a hundred thousand times that of water at the concentration used. This confirms the frequently cited but unproven suggestion that natural polymer fluids are unsuitable for foundation construction because they are very difficult to clean (Section 1.1.2).

Regarding the flow curves of the dry HPAM fluids, it can be seen that they are not significantly different from each other over the middle to high shear-rate range and only diverge from the common path at a shear rate of below 10^{-1} s^{-1} . If they are compared by their η_0 values, they can be ranked in this order starting from the highest: Shore Pac, POLY-BORE, SlurryPro CDP, HPAM standard no. 1, and EZ MUD DP. Much of this behaviour can be explained by referencing to the molecular weight and degree of hydrolysis results given in Table 2.3, as follows. Shore Pac has the highest viscosity ($\eta_0 = 47,000 \text{ cP}$) because it has the highest molecular weight of $24 \times 10^6 \text{ g/mol}$. However, following this logic would lead to the prediction that SlurryPro CDP has the second highest viscosity ($\eta_0 = 12,500 \text{ cP}$) but in fact POLY-BORE does ($\eta_0 = 34,000 \text{ cP}$). A closer examination of their fundamental properties shows that although they have very similar molecular weights, POLY-BORE has a lower degree of hydrolysis than SlurryPro CDP (28% versus 38%). A search in the literature reveals that for PAM the optimal degree of hydrolysis is probably between 20-35% and if it goes beyond this value the viscosity would drop (Caenn *et al.*, 1989; Rabiee *et al.*, 2005). The exact reason for this behaviour is not known but according to Rabiee *et al.* (2005) it may have something to do with the hydrodynamic volumes of the molecules. It is possible that if the anionicity is too high, the strong electrostatic repulsion may prevent the molecules from entangling with each other thus weakening the internal molecular structure. The reason why SlurryPro CDP (and also Shore Pac for this matter) has a higher-than-optimal degree of hydrolysis for viscosities is that it is designed to encapsulate soil cuttings and the soil on the excavation side walls, so that the fluids can stay relatively clean and softening of the side-wall soil can be minimised (see also relevant discussions in Section 2.2.1). To perform these functions effectively and to offset the effect on polymer consumption a higher degree of hydrolysis

is therefore required (G. Goodhue, personal communication, Dec 2009). Lastly, HPAM standard no. 1 takes the third place in the viscosity ranking because both its molecular weight and degree of hydrolysis are more favourable to viscosity development than those of EZ MUD DP.

For the emulsion HPAMs, it can be seen that, in addition to the molecular weight and the degree of hydrolysis, the solid content of the emulsion is an important property for viscosity development. Shore Pac XX-Poly, which has the highest solid content of 53% (Table 2.3), gives the highest viscosity curve – a curve comparable to those of dry HPAMs. This is of course also due to the fact that the concentration of the emulsion HPAM fluids (2 kg/m^3) was higher than their dry counterparts (0.8 kg/m^3). It follows that the other two emulsion HPAMs, due to their lower solid contents (33% and 38%), give lower viscosity curves.

Detailed discussion of test results – Marsh funnel viscosity

Finally, given the availability of this data set it is useful to consider if there is any relationship between the Marsh funnel viscosity (t_{MF}) and the fuller rheological parameters such as η_0 . If the dry HPAM products are ranked again but this time by their t_{MF} (given in Table 3.1), then the order starting with the highest will be: POLY-BORE, Shore Pac, SlurryPro CDP, HPAM standard no. 1, and EZ MUD DP. It can be seen that this order is actually rather similar to the previous one ranked by their η_0 . This means that t_{MF} , despite previously shown to be parameter associated with high shear (Figure 3.15), can also be indicative of the ranking of fluid viscosity at lower shear. Although this finding may seem to confirm the use of the Marsh funnel as a tool for ‘rough measurements in the field’ as originally said by Marsh himself (Marsh, 1931), it is only true of measurements made between polymers with similar chemical structures. This is because polymers of different chemical origins will have very different shear-thinning characteristics and hence a different $t_{\text{MF}} - \eta_0$ relationship. For example, the xanthan gum ‘DUO-VIS PLUS NS’ is highly shear thinning ($n = 0.26$). It therefore has the lowest t_{MF} of 37 s but the highest η_0 among all all tested polymers at 156,000 cP. On the other hand, despite also being a natural polymer, the cellulose ‘Aqualon CMC 7H3SF’ is much less shear thinning ($n = 0.65$). It therefore has a moderate t_{MF} of 47 s but a η_0 of merely 103 cP.

Summary

This section has discussed the flow behaviour of different polymer products over a wide range of shear rates. For the commonly used HPAMs, much of this behaviour can be explained by their fundamental properties such as molecular weight, degree of hydrolysis, and solid content. The flow behaviour also can be suitably described by well established mathematical models which, depending on the shear-rate range of interest, could be the four-parameter Carreau model or the two-parameter power-law model (but never the Bingham plastic model!). All these suggest that HPAM is a material that can be specified to fulfil certain physicochemical properties by engineers, much in the same way that bentonite is specified by their Bingham plastic parameters, moisture content, chemical reactivity, etc. The actual desirable range of these properties will of course need to be determined through carefully conducted case studies and laboratory work. In the following sections, other factors affecting the rheological properties of polymer fluids will be explored.

3.4.4 Effect of Polymer Concentration

To investigate the effect of polymer concentration on flow behaviour, further rheological experiments were carried out using the commercial HPAM ‘Shore Pac’. Figure 3.17 shows the flow curves for concentrations ranging from 0.2 to 1.2 kg/m³. This range was adopted to cover, and in fact slightly exceed, the range recommended by the supplier of this polymer. Table 3.2 summarises the Carreau model parameters of the fitted curves and the measured Marsh funnel viscosity for each polymer concentration.

As background information, Table 3.3 also shows the suggested polymer concentrations for different types of ground conditions. The expected Marsh funnel viscosity value for each concentration is also given in the table. It can be seen that a large polymer dosage (1.0-1.2 kg/m³) is recommended for gravels and cobbles but a much smaller dosage (0.4-0.5 kg/m³) is needed for clay and shale. This is probably due to the fact that a higher solution viscosity is necessary to minimise the amount of fluid loss in coarse soil, and hence a higher polymer concentration is also necessary. As fluid loss is usually not a concern in fine-grained soils due to their low permeability, the solution viscosity therefore

can be lower.

As expected from the supplier's data, Figure 3.17 shows that when the polymer concentration increases, the viscosity also increases. The critical shear rate ($\dot{\gamma}_{\text{crit}}$), which is where the viscosity leaves the Newtonian plateau of η_0 , tends to decrease as the polymer concentration increases. This means that the fluid becomes more sensitive to shear as the concentration increases, probably due to the increased amount of chain entanglements (i.e., weak points). This behaviour is also shown by the gradually decreasing value of n as the concentration increases (Table 3.2). To examine the rate of viscosity change due to a change in the polymer concentration, Figure 3.18 shows both η_0 and η_∞ plotted against concentration on a log-log scale. It can be seen that both η_0 and η_∞ show a power-law relationship with concentration and that they form the lower and upper boundaries of viscosity range on the plot. It is interesting to see that the slope of η_0 is much steeper than that of η_∞ , meaning that a change in the concentration has a much greater effect on the low-shear viscosity. This phenomenon can again be explained by using the chain entanglement theory, as follows. When the fluid is undisturbed, a higher concentration means that the polymer chains have more opportunities to interact with their neighbours and become entangled. However, at high shear rates when these polymer chains are all 'combed out' by the flow, the effect of extra entanglements then becomes virtually non-existent. Of course, the simple presence of the extra dissolved material also has an effect on the viscosity and this is why there is still a small increase in η_∞ as the concentration increases.

The two curves in Figure 3.18 show another interesting feature, which is that their slopes remain almost the same over the entire concentration range from 0.2 to 1.2 kg/m³. This is worth noting because in theory the slope should change dramatically once the critical polymer concentration (c_p^*) has been reached (see e.g. Barnes, 2000). In molecular network theory, c_p^* is commonly defined as the point at which chain entanglements take place. Williams & Phillips (2003) mention that c_p^* is usually the concentration level where the behaviour of polymer fluids changes from Newtonian to Non-Newtonian. Although this value is not known in this case as there is no obvious change in the slopes of the curves, it can be deduced based on the results already seen. First, it has already been shown

in Section 2.9 that this HPAM contains numerous entanglements in its microstructure at a concentration of 2.0 kg/m^3 (Figure 2.38), and therefore by the definition of c_p^* its value must be below 2.0 kg/m^3 . Second, since the c_p curve remains a straight line down to 0.2 kg/m^3 and non-Newtonian behaviour has been seen over this entire concentration range, the value of c_p^* for this HPAM must be less than 0.2 kg/m^3 . What this means to the engineers is that over the typical concentration range used for geotechnical construction, the fluid viscosity should be predictable using a simple power-law relationship such as those given in Figure 3.18 (note in these best-fit curves the offset was fixed at 1 cP , i.e., the viscosity of water). As an example of the use of these curves, if a user decides that to stabilise an excavation in a gravelly soil a zero-shear viscosity of 10^6 cP is required, then from Figure 3.18 one can immediately work out the required polymer concentration is 2.5 kg/m^3 provided that the water used is of similar quality. For water of different quality (e.g. hard water), for the same polymer concentration the achievable viscosity could be significantly lower. This is exemplified by the typical Marsh funnel viscosity (t_{MF}) values given by the supplier being about 30 s lower than those measured in this study (Tables 3.3 and 3.2).

To explore further the relationship between t_{MF} and other rheological parameters, Figure 3.19 plots both t_{MF} and η_∞ against polymer concentration. It can be seen that η_∞ can also be described fairly well by a linear equation, and in fact this equation is not very different from the power-law equation given on the previous plot. If the trend line is extrapolated back to zero concentration, it will give a viscosity of 1.07 cP which is very close to the solvent (water) viscosity of 1 cP . However, if the trend line for the Marsh funnel viscosity is also extrapolated back to zero concentration, the t_{MF} will be 38 s instead of the expected value of 26 s. To confirm this is true, an additional Marsh funnel test was carried out at the extremely low concentration of 0.01 kg/m^3 by dilution and the measured t_{MF} was indeed 38 s. Although it is not known why t_{MF} is not 26 s at about zero concentration, the implication of this is quite important: engineers should not have the false sense of security that a t_{MF} of 38 s is a reasonable ‘viscosity’ value as the true viscosity under such conditions could be very low.

3.4.5 Effect of Pumping and Entrained Air Bubbles

The discussion so far has focused on the flow behaviour of polymer fluids prepared and tested under ideal laboratory conditions. Testing under such conditions is useful in its own right since it allows comparisons between many different products (Section 3.4.3) and assessments of the effect of polymer concentration (Section 3.4.4) to be carried out. However, there are also many site factors that can influence the viscosity of a polymer fluid, such as water quality, mixing method, and contamination by the ground or fresh concrete. This section attempts to explore the fourth factor – the effect of pumping. O'Neill & Reese (1999) and Brown *et al.* (2010) noted in the last two versions of the FHWA drilled shaft manual that diaphragm pumps (Figure 3.20) cause less damage to polymer fluids than centrifugal pumps (Figure 3.21). However, they did not provide any evidence to support their claim nor mention the possible degree of damage caused by these pumps. A survey of product literature produced by leading polymer suppliers reveals contradictory advice: one supplier strongly discourages the use of centrifugal pumps whilst others either allow their use or provide no guidance on the matter.

It should be noted that from a contractor's point of view, centrifugal pumps are a better choice since their capacity is much higher. For example, a 'typical' medium-sized centrifugal pump such as the one shown in Figure 3.21 has a maximum flow rate of $320 \text{ m}^3/\text{h}$ whilst the capacity of some of the largest diaphragm pumps can rarely exceed $65 \text{ m}^3/\text{h}$. This means that to fill or empty a pile bore of, say, 1.2 m wide and 20 m deep with polymer fluids¹¹, it would take just 4 min with a centrifugal pump but about 20 min with a diaphragm pump. Most contractors will find it unacceptable to wait that long as schedules are usually tight. To achieve the same result, it would mean the contractors will have to replace one centrifugal pump with four diaphragm pumps (see e.g. KB International, 2007). This is not something they would do unless there is a good reason.

To investigate whether centrifugal pumps are indeed harmful to polymer fluids, a field trial was carried out on a construction site in central Glasgow to assess their effect on the viscosity of the fluid. A photograph of the site is shown at Figure 1.8. Since the trial

¹¹At sites where the ground conditions consist of firm to stiff clay overlying sand, it is common to dig through the clay dry and then fill the bore with support fluids just before hitting the underlying sand layer or when instability occurs (this latter condition to be avoided), whichever happens first. London is an example of this kind of ground condition and piling practice; an example can be found in Section 3.4.7.

was carried out alongside a real bored piling project and the contractor Balfour Beatty Ground Engineering (BBGE) were assessing the difference between SlurryPro CDP and Shore Pac, this offered an opportunity to evaluate the effect of pumping on both of these products. However, since they were provided by different suppliers, the recommended dosage were slightly different. For the Glasgow silty fine sand conditions, the recommended dosage of SlurryPro CDP is about 1.0 kg/m^3 whereas it is 0.6 kg/m^3 for Shore Pac (KB International, 2007; Table 3.3). The reason why they are different could be due to the fact that Shore Pac is capable of producing a higher viscosity than SlurryPro CDP at the same concentration (Section 3.4.3), although non-technical reasons such as sales strategy may also have a role to play. Because BBGE have been using the SlurryPro CDP polymer in Glasgow since 1999 (Section 1.2.2), they have learned that a slightly lower dosage of 0.9 kg/m^3 is enough for piling in the area and hence this was the actual dosage used for SlurryPro CDP. Figure 1.9 shows the mixing process.

Figure 3.22 presents a schematic diagram of the plant setup at the site. As can be seen, the polymer fluids were mixed for about 1.5 h in a 20 m^3 tank by aeration (Figure 3.23) and then fed into the pile bore by a centrifugal pump when excavation started. The pipe work was laid out so that the polymer fluid was recirculated back to the tank when the ball valve in the feed line to the excavation was shut. This meant that the pump did not need not be turned repeatedly on and off during the excavation – an important aspect of plant operation as the pump and the storage tank may be at some distance from the excavation. With regard to the testing strategy, polymer fluids were taken from the tank immediately after mixing using a fluid sampler (Figure 3.24) and then during excavation when the pump was turned on. Only the Marsh funnel was used to give an indication of the viscosity.

Figure 3.25 shows the monitoring results. Firstly, it can be seen that the starting viscosities of the two fluids are very different, being 103 and 71 s for SlurryPro CDP and Shore Pac respectively. Given the dosage of Shore Pac was 0.3 kg/m^3 less than that of SlurryPro CDP, the difference is considered normal. Secondly, during the initial standing period between the end of mixing at 18:00 and the start of pumping on the following day at 08:00, the viscosity dropped from 103 to 89 s for SlurryPro CDP and from 71 to 65 s

for Shore Pac. This initial drop was unexpected as it was originally thought that the extra time would have allowed any undissolved granules to break up and thus increase the viscosity. Although the tank was open at the top, the weather during the monitoring period was fine so that it could not have been caused by rainwater. Two possibilities remain: 1) ageing of the fluid overnight; 2) drop in the viscosity due to the escape of the entrained air bubbles in the fluid. Although ageing is a known feature of HPAM fluids (Henderson & Wheatley, 1987), it is usually associated with a much longer time scale and was therefore unlikely to be fully responsible for the amount of viscosity reduction seen over just one night. The second effect was much more probable since entrained air bubbles are known to increase the viscosity of a fluid due to flow distortion (see e.g. Taylor, 1954), and the polymer fluids prepared for the trial did contain numerous bubbles immediately after mixing (Figure 3.23). Therefore, it is reasonable to believe that the viscosity measured at the start of the monitoring period (i.e., at time zero on Figure 3.25) was significantly affected by the presence of the air bubbles. Overnight, some of the bubbles escaped and the viscosity measured on the next day was therefore slightly lower. Since the concentration of SlurryPro CDP was higher than that of Shore Pac, this also explained why SlurryPro CDP had a larger initial drop in viscosity than Shore Pac (14 s versus 6 s) because it was capable of holding more bubbles initially due to the higher viscosity.

As soon as pumping started for the excavation, it can be seen that the viscosity of both fluids dropped significantly. After the excavation was complete, the viscosity of SlurryPro CDP had dropped from 89 to 55 s over period of 5.5 h and that of Shore Pac had dropped from 65 to 35 s over a period of 7.8 h. These results confirmed that centrifugal pumps, especially when used in a loop system, can significantly reduce the viscosity of the polymer fluids. Since both polymers are synthetic HPAMs, a search in the polymer science literature has revealed that this kind of polymer is particularly vulnerable to high shear which can cause scission of the polymer chains (e.g. Caulfield *et al.*, 2002) although the relatively high degree of hydrolysis should help increase their resistance to mechanical degradation (Henderson & Wheatley, 1987; Scott *et al.*, 1996). Referring back to the figure, it can be seen that towards the end of the monitoring period the rate of

viscosity reduction appeared to accelerate. This was probably due to the fact that for each batch of fluids a significant portion was drawn off to the excavation ($\approx 9\text{m}^3$) thus increasing the rate of recirculation of the fluid remaining in the tank.

This simple field trial has revealed several interesting aspects of polymer fluids when used in a real construction project. The temporary effect of entrained air bubbles, although discussed in the science literature before, has never been mentioned in the civil engineering literature. Therefore, foundation engineers are currently unaware of this effect and could be misled into thinking that the viscosity of their freshly mixed fluids is high when in fact it is only a temporary effect of the air bubbles. The trial also has confirmed the caution given in O'Neill & Reese (1999) and Brown *et al.* (2010) that HPAM fluids are susceptible to damage by centrifugal pumps, and in fact any pumping equipment which induces high shear stresses on the long-chain polymer molecules. Although this also includes diaphragm pumps, they are much less likely to cause damage as they can automatically switch themselves off when the outlet flow is stopped and, therefore, do not need to be used in a loop system (see e.g. KB International, 2007). This feature of diaphragm pumps also offers the advantage of reduced on-site energy consumption and carbon footprint.

Finally, it should be noted that, despite the evidence produced by this field study, more rigorous laboratory studies will still be needed should one want to accurately quantify the effects of air bubbles and high-speed shearing. For the investigation of the latter, laboratory-scale centrifugal and (or) diaphragm pumps could be used in a loop system to study the effect of mechanical degradation, though it would be necessary to scale the pumping rig appropriately to maintain field shear rates.

3.4.6 Effect of Ageing

In the previous section it has been shown that the viscosity of a polymer fluid is not a constant with time even without external influences. Although it was concluded that much of the initial reduction seen at the Glasgow site was due to the escape of the air bubbles, it would still be useful to examine the effect of ageing so that this potential effect is not overlooked. The discussion will also help justify the mixing and testing strategy employed for the previous laboratory analyses, i.e., the polymer fluids were mixed for 40 min by a laboratory mixer and then used within 3 days (Section 3.4.3).

A survey of the literature has revealed a wide range of results concerning the viscosity development and ageing characteristics of (H)PAM fluids. These results may seem contradictory at first sight but in fact they can usually be explained by the properties of the materials and the mixing method used. For instance, both Gardner *et al.* (1978) and Likos *et al.* (2004) reported that it took a minimum of 5 h after mixing for their polymers to fully develop the viscosity. However, upon examination of their test methods it becomes clear that their results are the effect of insufficient mixing. For instance, in the case of Likos *et al.* the polymer fluids were mixed with a large spoon for merely five minutes as opposed to 40 min chosen for this study and over 1 h commonly found on construction sites. In terms of the ageing characteristics, Mächtle (1982) found no ageing effect in their non-ionic PAM solution but Henderson & Wheatley (1987) found a gradual reduction in the viscosity with time in their 30% hydrolysed HPAM solution. Kulicke & Kniewske (1981) also found that, for the same degree of polymerization (molecular weight), the higher the degree of hydrolysis the greater the loss of viscosity with time. This difference in ageing behaviour can be explained by referring to the molecular network theory as follows. For polymers with a high degree of hydrolysis, the polymer chains are entangled with each other and the solution has a high viscosity (see e.g. Figure 2.38). Given time, some of the entanglement would be lost and this would result in a reduction in the solution viscosity. For polymers which are only slightly hydrolysed or are completely non-ionic, the polymer chains would exist as random coils with little interaction with each other. Therefore, the opportunity for further reduction in the entanglement number is very limited and so is the potential for ageing. Finally, It should be noted that there are also ageing characteristics which are not readily explicable by the material properties or mixing method. For example, both Shyluk & Stow (1969) and Bruce & Schwarz (1969) found a rapid decrease in viscosity followed by a much slower gradual reduction in their PAM solutions. They attributed their findings to chemical reactions and chain disentanglement.

As these previous studies involved many different (H)PAMs and the interpretation methods were all slightly different, their results are not readily transferable to the HPAMs that are currently used for geotechnical construction. It was therefore decided to carry out an ageing test on the commercial HPAM ‘Shore Pac’ which, as discussed before, is a

product sold to the foundation industry. This product was chosen because its fundamental properties had been fully characterised and could be used to explain the ageing test results if required.

The testing procedures involved first preparing a solution of the material at a moderate concentration of 0.8 kg/m^3 (see Table 3.3) using both deionised water and Oxford tap water pre-treated using soda ash (sodium carbonate). The reason why Oxford tap water was used was that both Henderson & Wheatley (1987) and Kulicke & Kniewske (1981) found less ageing in polymer fluids which were prepared with impure water. Therefore, for comparison it was decided to include Oxford tap water as it is known to have a high concentration of calcium and magnesium ions amounting to a total hardness of about 250 ppm. Following the supplier's advice, soda ash was used to pre-treat the water to pH 10. It was found that a dosage of 0.9 kg/m^3 was sufficient to raise the pH to 10 and remove about 90% of the hardness. The remaining 10% was probably due to the magnesium ions which could not be removed by this method.

Immediately after 40 min of mixing by the Lightnin laboratory mixer, the solutions were transferred to a 600-mL beaker and then tested using the Brookfield viscometer. The Brookfield viscometer (Figure 3.6) was ideally suited to this test since the sample volume was large and the sample could be sealed by several layers of plastic film. This allowed continuous measurements of the viscosity over a long period without any noticeable moisture loss. Since a single rotational speed had to be chosen for the test, a moderately high speed of 20 r/min (equivalent to 4.4 s^{-1} when used with the spindle LV1) was used to keep the measured torque at a reasonable level for both samples. Finally, to prevent photo-degradation of the material (Caulfield *et al.*, 2002) and the effect of temperature variation, the sample was wrapped by a layer of tin foil and the laboratory room temperature was controlled at 25°C .

Figure 3.26 shows the ageing test results over a 30-day period. First of all, it can be seen that water quality had a significant effect on the viscosity of the fluid. The starting viscosity of the sample mixed with deionised water (190 cP) was four times as high as that of the sample mixed with Oxford tap water (48 cP). This means that, despite the soda ash treatment, the Oxford tap water was not a good solvent for the HPAM probably due

to the magnesium ions that remained and/or the extra sodium ions that were introduced. The lower viscosity can be explained by the theory that the polymer chains in deionised water were fully expanded due to the electrostatic repulsion between the anionic charges along the chains, whereas in Oxford tap water they were more compact since some of the charges were shielded by the positive ions (see e. g. Barnes, 2000).

In terms of the actual ageing characteristics, it can be seen that none of the samples showed any initial rapid decrease of viscosity as reported by some of the past researchers. However, as expected the sample mixed with deionised water did show a larger reduction in viscosity than the one mixed with Oxford tap water. This confirmed the findings of Henderson & Wheatley (1987) and Kulicke & Kniewske (1981) that HPAM fluids mixed with pure water aged the most. This phenomenon can perhaps be explained by the theory that the polymer chains were fully stretched out in the deionised water, so that there was more room for them to return to their natural random configuration, whereas the polymer coils in the Oxford tap water were already compact and hence fewer conformational changes could take place. Kulicke & Kniewske (1981) put forward the theory that conformational changes of a polymer molecule occur because certain types of intramolecular hydrogen bonds are lost, thus leading to a more stable and compact coil over time.

Finally, it can be seen from the figure that none of the samples showed any noticeable ageing during the first five to six days. This reinforces the conclusion of the previous section that the initial drop in viscosity seen at the Glasgow site was due to the escape of the air bubbles but not the ageing of the fluid.

3.4.7 Effect of Repeated Use

The discussion so far has only focused on the properties of clean polymer fluids, but in practice this condition can only be maintained for a short period of time. This is because as soon as the fluid is pumped into the excavation it will start to interact with the suspended soil particles due to mixing action of the excavation tools. Although the coarse particles (sand and gravel) will fall out of suspension relatively fast, the fine particles (clay and silt) will stay in suspension for considerably longer due to the high viscosity of the fluid at low shear rates (Figures 3.16 and 3.17). These suspended particles will change

the properties of the polymer fluids both physically (e. g. flow behaviour) and chemically (e. g. chemical reactivity). This section examines the effect of suspended soil particles on the density and flow behaviour of polymer fluids due to repeated use.

Site details

To achieve this objective, field monitoring work was carried out during the piling work for the construction of the Castle House project at Elephant and Castle, East London. The contractor was BBGE and the polymer system used was SlurryPro CDP. The work involved the construction of 37 bored piles which were 40 m deep and 0.9 to 1.5 m in diameter over a period of 52 days. The ground conditions consisted of made ground (0 to 4.5 m), River Terrace Gravel (4.5 to 5.9 m), London Clay (5.9 to 20.5 m), Lambeth Group (20.5 to 40.0 m), and Thanet Sand (40.0 to >50.0 m). Following common piling practice in that area, a steel casing was sunk to the top of the London Clay to support the loose soil at the top and to stop the perched groundwater from entering into pile bores. At this site, the piles were excavated dry until about 18 m deep and then the polymer fluid was introduced to counterbalance the groundwater pressure. Because of this arrangement, the polymer fluid had the most interaction with the Lambeth Group which, according to the site investigation, was a very stiff fissured sandy clay interbedded with some green glauconitic sand.

Since up to two piles could be excavated at any one time at the site, the tankage had to be large enough to cope with the large amount of fluids required for and unwanted fluid from the excavation. As a result, six 45 m³ tanks and one 25 m³ tank were used to handle the fluids for the project. Figures 3.27 and 3.28 show a schematic layout and a photograph of the tanks, respectively. As can be seen from the first figure, the tanks were assigned for different purposes: two for mixing and storage, one for primary sedimentation tank, three for secondary sedimentation, and one for fluid reconditioning (polymer replenishment). The mixing and storage tanks, as the name suggests, were used to prepare and store fresh polymer fluids, which had a concentration of 0.8 ± 0.1 kg/m³ for this project. After use, the exhausted fluids went through two stages of sedimentation before being rejuvenated by either adding more dry polymer granules or by mixing with the fresh fluids, depending on the total fluid volume in the system and the expected demand at the time. After

rejuvenation the fluids were sent back to the excavation again. Figure 3.29 shows a photograph of the reconditioned fluid being pumped into a pile bore at Castle House. If this photograph is compared with the one shown in Figure 1.2, then it can be seen that the colour of the reconditioned fluid was significantly darker. This was due to the suspended clay and silt particles which accumulated in the fluid after repeated use.

Results

The effect of suspended soil particles on fluid density is illustrated in Figure 3.30. The measurements were made by a hand-held digital density meter which had a resolution of 1 kg/m^3 (Anton Paar, model DMA 35n). Overall, it can be seen that the density of the used fluids gradually increased during the first ten days and then stayed at the density level of around $1015 \pm 5 \text{ kg/m}^3$ before going up again near the end of the project. This pattern of density change actually had a lot to do with the site-specific conditions, the polymer product used, and most importantly the way the contractor dealt with the fluids. For example, since 1015 kg/m^3 was the upper density limit suggested by the supplier, following the initial increase to this level after the 10 days, the contractor increased the mixing ratio of fresh to exhausted fluids to about 1:4 in the reconditioned fluid tank. This successfully kept the fluid density at around that level for the next 40 days. However, near the end of the project the contractor decided to stop mixing fresh fluids in order to reduce the amount of fluid to be treated at the end of the project. This led to the second phase of increase as shown on the plot. In terms of the fluid density between the different sampling locations, it can be seen that they take the following order starting from the lowest: clean, reconditioned, secondary sedimentation, primary sedimentation, and pile bores. This order confirms the effectiveness of the sedimentation system that was put in place.

The flow properties of the fluid were measured by both the Marsh funnel and the Fann viscometer. The results given by the former are first discussed, as follows. Figure 3.31 *a* shows t_{MF} plotted against time. To aid the discussion the average t_{MF} for each sampling location has also been calculated and drawn on the plot. It can be seen that the average t_{MF} was the highest when the fluid was clean. It then dropped (annotated by the arrows on the right of the plot) as the fluids went through various parts of the system before going

back to almost where it started after rejuvenation. The reduction in viscosity is believed to be caused by a wide range of factors, including: 1) ongoing escape of the entrained air bubbles; 2) shear degradation by the centrifugal pumps; 3) charge neutralisation of the molecules by the soil particles; 4) ageing of the fluid; 5) dilution by rainwater (the tanks were open top); 6) sedimentation of the coarse particles. Since Pitt (2000) suggests that t_{MF} has a direct relationship with the density of the fluid, Figure 3.31*b* also plots t_{MF} against density. Although a general increasing trend in t_{MF} can indeed be seen with an increasing density, the data are quite scattered. It can also be argued that an increase in the fluid density does not cause an increase in t_{MF} , since some of the highest t_{MF} actually correspond to the clean fluids as already shown in Figure 3.31*a*. This apparent contradiction may be explained by the fact that although the presence of the suspended particles would increase the viscosity (Einstein, 1906), the finer ones would also reduce the viscosity at the same time due to neutralisation of the anionic charges along the polymer molecules.

Although t_{MF} plots have highlighted some interesting properties of the fluids during the construction period, it should be remembered that t_{MF} is only a single-point, time measurement intended for ‘rough measurements in the field’ (Marsh, 1931). To investigate the effects of suspended soil particles on the actual flow behaviour, the Fann viscometer data were also analysed and fitted with a power-law model following the procedure discussed in Section 3.4.2. The two parameters, n and K , were plotted against time and are shown in Figure 3.32. It can be seen that, despite the large scatter, as time increased n appeared to decrease slightly but K appeared to increase. This suggests that the fluids were becoming more shear-thinning while at the same time acquiring extra viscosity (consistency) due to the repeated reuse. It is interesting to see that K increased almost in the same way as the density did as shown in Figure 3.30. This suggests that the power-law parameters may have a direct correlation with the density of the fluid. To investigate this, the two parameters were replotted against density and the results are shown in Figure 3.33. Since the fluid behaviour is now represented by two parameters instead of one (i.e., t_{MF}), the inconclusive result previously seen in Figure 3.31*b* has disappeared. It can now be said with confidence that the suspended particles caused the

fluids to become more shear-thinning and at the same time to make them more viscous. To illustrate this concept more clearly, Figure 3.34 shows the flow curves of selected fluid samples taken from the pile bores. It can be seen that the overall effect of the suspended particles was to raise the viscosity to up to five times the original values. Since suspended particles had a greater effect on the viscosity at low shear, it made the fluids to appear more shear-thinning. The reason why it had a larger effect at low shear was probably due to the solids making it more difficult for the polymer chains to disengage from each other, thus creating a synergy effect on the viscosity. At high shear, when the effect of chain entanglement became nonexistent, this extra effect therefore also disappeared.

3.5 Viscoelasticity

3.5.1 Background

The shear-dependent fluid behaviour considered so far is actually only a subset of the types of non-Newtonian behaviour observed in polymer fluids. In the previous discussions, the fluids were assumed to be inelastic when in fact they also exhibit some degree of elasticity. Materials of this kind are known as viscoelastic. Most fluids that we encounter in our daily life actually fall under this category. Some examples are paint, glue, yogurt, saliva, blood, and shampoo, to just to name a few. Perfectly elastic solids and perfectly viscous fluids are, in effect, the limiting cases of viscoelasticity, but for most materials it is only these limiting conditions that are observed. For example, the elasticity of rock is a lot more obvious than its viscosity although the latter does exist. The fact that nothing remains still was even known by people in biblical times, and this is reflected by the line ‘the mountains flow before the Lord’ in the Bible (Judges 5:5) and by the quote ‘everything flows’ believed to be said by the ancient Greek philosopher Heraclitus (Reiner, 1964).

For polymer fluids, the study of their viscoelastic properties can help engineers understand their flow through soil and rock (Sorbie, 1991), the sedimentation of suspended particles (Chhabra, 2006), and their effect on soil and foundation behaviour. Some researchers have shown that fluids such as bentonite slurries, silicone oil, and polyacrylamide hydrogels can significantly improve the cyclic resistance of soil (Ellis *et al.*,

2000; El Mohtar *et al.*, 2008; Graham & Elton, 2011). It follows that polymer fluids may also have a similar effect. Polymer fluids may also have a ‘soft grouting’ effect on soil as suggested by some suppliers (e.g. KB Technologies, 2002). This effect, although not yet proven under controlled laboratory conditions, is possible since polyacrylamides (PAMs) have been used as a soil conditioning agent (Wallace & Terry, 1998) and as the key ingredient of a chemical grout before (Berry, 1982).

The viscoelasticity of a polymer fluid can actually be shown without doing any rheological experiments because it can manifest itself very visually in a number of ways (Collyer, 1973). For example, anyone who has handled HPAM fluids must have experienced the self-siphoning effect which is shown in Figure 3.35. The fluid showing the effect was a 1-kg/m^3 ‘Shore Pac’ HPAM fluid. The figure shows that, when the fluid was tipped from a beaker, the hanging fluid was able to ‘pull’ more fluids out from the beaker due to its viscoelasticity, resulting in an upward flow over the wall of the beaker. Note that the surface of the liquid was well below the rim of the beaker and yet the stream reached up and flowed over the lip. This would not have happened if the fluid was not viscoelastic.

Another manifestation of viscoelasticity can be seen during the mixing of polymer fluids in a laboratory. When water is stirred rapidly in a container, a deep vortex is formed. However, soon after the addition of polymer granules into the water, the vortex will disappear and the fluid will climb up the stirring rod as shown in Figure 3.36. This phenomenon is known as the Weissenberg effect, which is named after Karl Weissenberg who first observed it (Weissenberg, 1947). The fluid appears to climb up the rod because when it is sheared along the x axis (in the direction of the horizontal flow), two perpendicular stresses, P_{yy} and P_{zz} , are also set up along the y and z axes. Since in a viscoelastic fluid P_{yy} and P_{zz} are not equal, the unbalanced force then pushes the fluid up the rod. Figure 3.37 illustrates the concept of non-zero normal stresses diagrammatically.

3.5.2 Fundamentals

The viscoelastic properties of materials have been a subject of investigation for many decades by chemical engineers and polymer scientists, and hence a vast amount of background knowledge already exists in the literature (e.g. Ferry, 1980; Bird *et al.*, 1987; Shaw & MacKnight, 2005). However, as this is not a subject area civil engineers are

familiar with, this section presents some of the basic concepts which are essential for the understanding of the experimental results to be presented later.

As the viscoelasticity of a material is commonly investigated by oscillatory shear experiments, this topic is discussed first as follows. In the classical theory of elasticity, Hooke's law states that in a sheared body the stress¹², τ , is directly proportional to the strain, γ , and hence:

$$\tau = G\gamma \quad (3.6)$$

where G is the shear modulus of the material. If this perfectly elastic (Hookean) solid is now subjected to an oscillatory shear stress, the deformation will be instant and hence the stress and the strain are exactly in phase as shown in Figure 3.38. However, for a perfectly viscous (Newtonian) liquid, the strain will be 90° out of phase and this is also shown in the figure. For a viscoelastic material, the phase angle, δ , will be in between 0° and 90° . δ is therefore a measure of the viscoelasticity, but for interpretation of oscillatory test results, this parameter is usually used to separate the measured stiffness, G^* , into two components:

$$G' = G^* \cos(\delta) \quad (3.7)$$

$$G'' = G^* \sin(\delta) \quad (3.8)$$

where G^* is known as the complex modulus, G' is the storage (elastic) modulus, and G'' is the loss (viscous) modulus. They all have the unit of pascal (Pa). Therefore, the assessment of viscoelasticity can be carried out just by comparing their relative values. If $G' > G''$ the material is said to be solid-like. Similarly, if $G'' > G'$ it is said to be liquid-like. By the definition of Equations 3.7 and 3.8, the relationship between G^* , G' , and G'' is:

$$G^* = \sqrt{G'^2 + G''^2} \quad (3.9)$$

Another popular way of investigating viscoelasticity is by carrying out creep tests over a range of times, i.e., by applying a constant shear stress and measuring the resulting strain thereafter. In rheology, creep is typically expressed as compliance, J , which is defined in the following equation:

¹²In some rheology textbooks, the Greek letter ' σ ' is also used to denote shear stress (e.g. Ferry, 1980; Barnes, 2000). In this thesis, this letter is reserved for normal stress.

$$J(t) = \frac{\gamma(t)}{\tau} \quad (3.10)$$

where $\gamma(t)$ is the time-dependent strain and τ is the applied shear stress. Essentially, J is the normalised strain of the material and has the unit of Pa^{-1} . The value of J is independent of the applied stress so long as the material is within its linear viscoelastic region (LVR). Within this region, the internal elastic structure of the material is not damaged by the applied stress and hence Hooke's law applies.

To interpret the result of a creep test, mechanical models consisting of dash pots (viscous components) and springs (elastic components) are commonly used. The two most well-known models are the Maxwell and the Kelvin-Voigt models. They are shown in Figure 3.39 and their respective equations are:

$$\gamma = \frac{\tau}{G} + \frac{\tau}{\eta} \cdot t \quad (3.11)$$

$$\gamma = \frac{\tau}{G}(1 - e^{-t/\Lambda}) \quad (3.12)$$

where t is time, and Λ is the retardation time defined as η/G . Since these simple models often cannot accurately represent the behaviour of real materials, they are usually used in combination to form the four-element Burgers model as shown in Figure 3.39. The equation of this combined model is:

$$\gamma = \tau \left[\frac{1}{G_1} + \frac{1}{G_2}(1 - e^{-t/\Lambda}) + \frac{t}{\eta_1} \right] \quad (3.13)$$

where the subscripts '1' and '2' indicate the parameters of the original Maxwell and Kelvin-Voigt elements respectively, and Λ is now given by η_2/G_2 . In fact, any number of Kelvin-Voigt elements can be added in series to the Burgers model to improve its ability to describe most practical creep curves. The equation of the extended multi-element Burgers model is therefore:

$$\gamma(t) = \tau \left[\frac{1}{G_1} + \frac{1}{G_2}(1 - e^{-t/\Lambda_2}) + \frac{1}{G_3}(1 - e^{-t/\Lambda_3}) + (\dots) \dots + \frac{t}{\eta_1} \right] \quad (3.14)$$

which can be rewritten as

$$J(t) = J_0 + \sum J_i(1 - e^{-t/\Lambda_i}) + \frac{t}{\eta} \quad (3.15)$$

where i represents the i th Kelvin-Voigt element. The three terms on the right-hand side

of this equation all have different physical meanings: the first term J_0 , which is commonly known as the instantaneous compliance, represents the immediate elastic response; the middle term represents the delayed or retarded elastic response; the last term represents the steady-state viscous response. Note that a viscoelastic material does not necessarily have all these three terms. For example, for a viscoelastic solid (e.g. a cross-linked polymer) the last term is zero if the applied stress is smaller than the yield stress, and hence $\eta = \infty$. For a viscoelastic liquid (e.g. a non-cross-linked polymer), the first term may be zero. In addition, if the magnitude of τ is small enough, the steady-state viscosity, η , will be the zero-shear viscosity, η_0 . Therefore, a creep test can offer an alternative way to measure η_0 if a suitable small stress is used.

As it is useful to present the elastic component of a material as a single value, it is customary for rheologists to combine the individual compliances (J_i) to give the equilibrium compliance, J_e . This can be done by extrapolating the long-term steady-state curve back to time zero on a creep plot. Mathematically, when $t \gg \Lambda_i$ and the viscous compliance due to t/η is removed, Equation 3.15 then becomes:

$$J_e = J_1 + J_2 + J_3 \dots \quad (3.16)$$

To further the understanding of the different material models discussed above, Figure 3.40 shows their corresponding creep compliance curves.

3.5.3 Previous work

To the best of the author's knowledge, the viscoelasticity of support fluids has not been studied before, and this is true for both bentonite and polymer support fluids. This is perhaps not surprising given that other researchers are still only using the Marsh funnel and the Fann viscometer as their measurement tools (see Section 3.4.1). However, if the search is extended to include other branches of studies in geotechnical engineering, then a few publications can be found on the study of the viscoelasticity of bentonite fluids. For example, Santagata *et al.* (2008) studied G' and G'' of a Wyoming bentonite fluid over a range of shear strains using a commercial rheometer. The tests carried out are commonly known as the amplitude sweep test and is the standard rheological test to establish the range of LVR of a material at a given frequency. Figure 3.41 shows the waveform of a

typical amplitude test. From the results, Santagata *et al.* found that at strains smaller than about 10% the fluid was solid-like, and within the LVR (< 1 to 2% strain) G' was significantly greater than G'' thus indicating strong elastic behaviour. They also found that both the range of the LVR and the flow point ($G' = G''$) of the fluids increased with decreasing bentonite concentration. In a different study, El Mohtar *et al.* (2008) carried out an amplitude sweep test on a 10% bentonite fluid, and cyclic triaxial and resonant column tests on some sand–bentonite mixtures. They found that the presence of bentonite increased the number of cycles required to cause liquefaction at a given cyclic stress ratio (CSR). They attributed the observed behaviour to the delayed generation of excess pore pressure and the thixotropic (gel-forming) property of the bentonite fluid.

With regard to the viscoelasticity of HPAM polymer fluids, several references have been found in the polymer science literature. For example, Zhao *et al.* (2005) carried out some stress relaxation experiments (rheologically opposite of the creep test described previously) on a HPAM with a molecular weight of about 16×10^6 g/mol and a degree of hydrolysis of 26%. They found that the degree of viscoelasticity increased with increasing polymer concentrations and that the addition of NaCl can reduce the viscoelasticity. In another study, Zhao *et al.* (2005) carried out some amplitude and frequency sweep tests (Figure 3.41) on a HPAM with a molecular weight of 5×10^6 g/mol and a degree of hydrolysis of 23%. The concentration used was 5 kg/m³. From the amplitude sweep test, they found that the stress at the end of the LVR, i. e., the yield stress, was about 4 Pa. From the frequency sweep test (a test used to determine G' and G'' at different time scales), they found that the HPAM fluids were solid-like between a frequency range of 0.1 and 10 Hz indicating strong elastic behaviour at short time scales.

Since these previous studies on HPAMs were carried out using materials with different properties from those used for foundation construction (Table 2.3) and only a limited amount of data was provided by each study, the commercial HPAM ‘Shore Pac’ was used again for further oscillatory and creep measurements, described as follows.

3.5.4 Amplitude Sweep Tests

As discussed before, viscoelasticity is the property of materials that show both viscous and elastic characteristics when undergoing deformation. The amplitude sweep test is

a standard rheological test which can be used to determine the strain range over which linear elasticity (Hooke's law) applies. This range is known as the linear viscoelastic region (LVR). Because the stress-strain curve of the material is linear within the LVR, the values of G' and G'' are constant within this region. An understanding of the LVR is a prerequisite for the frequency sweep test since the latter requires an input strain value that is within the LVR.

For this series of tests, the Bohlin Gemini rheometer equipped with the parallel-plate geometry was used (Figures 3.10). As mentioned in Section 3.3.1, the parallel-plate geometry was used because it is the standard measurement geometry for oscillatory tests. The diameter of the top plate was 40 mm and the gap between the plates was 150 μm . A common frequency of 0.5 Hz was used for the tests, but other frequencies could also have been used since in most cases the strain at which non-linearity occurs is independent of frequency (Barnes, 2000). As in other types of oscillatory tests, the results of an amplitude sweep test can be interpreted by assessing the relative values of the two shear moduli and the phase angle.

Figure 3.42 shows the amplitude test results for polymer concentrations ranging from 0.2 to 1.2 kg/m³, which is the same range used for the viscometry tests discussed in Section 3.4.4. The results show that, at small strains, G' is higher than G'' and δ is smaller than 45°. Therefore the polymer fluids were showing solid-like behaviour. It can be seen that as the strain continues to increase, it is G' which breaks down first. This is a typical behaviour of viscoelastic materials as the elastic structure is more prone to damage by excessive deformation. The end of the LVR, that is, the location of the yield point, is therefore determined using G' and is defined as when its value has dropped by 5% from the plateau value. It can also be seen that the polymer fluids yield at strain values of just below unity (100%), compared to only 1-2% strain in bentonite fluids as reported by Santagata *et al.* (2008). This means that, when compared to bentonite, polymer fluids can maintain their internal elastic structure over a much larger range of strains. As the strain increases beyond the yield point, G' falls dramatically. The point where G' equals G'' or when δ hits 45° is known as the flow point. After this point, viscous behaviour dominates. As shown by the two dotted lines in Figure 3.42a, the strain values of both

yield and flow points increase as the polymer concentration increases. This is the opposite of what Santagata *et al.* (2008) found for bentonite fluids. As an additional remark, it is interesting to see that the overall shape of the modulus degradation curves is very similar to those commonly found on soil at small strains, despite the fact that they are obviously very different types of material.

To illustrate further the effect of polymer concentration on the modulus values, Figure 3.43 plots G' and G'' against polymer concentration, c_p . It can be seen that the two moduli do not increase linearly with the concentration but follow a parabolic curve. Figure 3.44 also plots the yield stress, τ_y , and the flow stress, τ_f , against concentration. Again these curves show a parabolic shape. The non-linear relationships between these material parameters and polymer concentration can be explained by reference to Figure 3.45, which shows the effect of increasing polymer concentration on the number of chain entanglements. It can be seen that when a new polymer molecule is added to the network it can form more than one additional entanglement and this ability increases as the number of existing molecules increases. Given that the viscoelastic properties of a polymer fluid are due to the chain entanglements in the solution microstructure, an increase in polymer concentration therefore has a greater than linear effect on these properties. Finally, it should be noted that the values given in these figures are dependent on the frequency of the oscillation, which is 0.5 Hz in this case. If other frequencies are used the values will be different as will be shown later.

To conclude this section on amplitude sweep tests, it is perhaps worth noting that although the polymer fluid has a measurable yield stress from the oscillatory experiments, it does not mean that it is a yield-stress fluid, that is, fluids that can be modelled by the Bingham plastic or Herschel-Bulkley equations (Equations 3.4 and 3.5). Although this may seem confusing to non-rheologists, it should be borne in mind that the values of the yield stress given in Figure 3.44 are frequency (time) dependent and they will become smaller at a lower frequency. This is the reason why polymer fluids can be regarded as inelastic in the study of their flow behaviour in the Section 3.4, because in those analyses only the steady-state behaviour was considered and any transient effect ignored. Another way to understand this issue about yield stress is by referring to the Maxwell model

(Figure 3.39) – although this model has a spring, the dash pot will always flow no matter how small the stress is. On the other hand, the dash pot in the Kelvin-Voigt model can only flow when the applied stress is large enough to overcome the yield stress of the spring. This is why the Kelvin-Voigt model is commonly used to represent materials which have a finite yield stress.

3.5.5 Frequency Sweep Tests

Based on the results of the amplitude sweep tests, further tests were run to characterise the response of the polymer fluids at different frequencies, f . In these experiments, a constant strain value of 0.1 was used as the target strain, and the corresponding initial stress was taken from the amplitude sweep results. As can be seen in Figure 3.42, at this strain level the polymer fluids are well within the LVR at all concentrations, and hence it can be ascertained that results of the frequency sweep tests are independent of the chosen amplitude of the oscillation. To ensure the actual strain value does not deviate from the target value, the ‘auto stress’ function of the rheometer was turned on to allow the control software to automatically adjust the applied stress based on the response of the previous oscillation. Since it is known from preliminary experiments that the stiffness increases with increasing frequency, based on the advice of the manufacturer the rheometer was programmed to start from highest frequency (10 Hz) to the lowest (0.001 Hz) to allow the computer to better control the applied stress. As for the amplitude sweep tests, the parallel-plate geometry was used and the gap was set at 150 μm . As this series of tests involved measurements at very low frequencies, the total testing period was over 4 h and hence the vapour hood was used to prevent evaporation of the specimen during the testing period. The underside of the vapour hood was also sprayed with some water to keep the air surrounding the specimen moist. This last step was found very important for tests which lasted for longer than about an hour because the specimen volume was very small and any evaporation could have a huge impact on the test results.

Figure 3.46 shows the frequency sweep test results. First of all, it can be seen that none of the curves reaches the highest target frequency of 10 Hz, and that, the lower the polymer concentration is, the shorter the frequency range becomes. This is due to the effect of inertia. Although the inertia of the rheometer was known and was automatically

accounted for by the control software, for low-viscosity samples the inertia of the rheometer could become significantly larger than the true inertia response of the sample at high frequencies. Therefore, some of the data at high frequencies became unusable and had to be disregarded. Although in theory a smaller gap would increase the sample's inertia response and hence the upper frequency limit, as the chosen gap was already close to the minimum practical limit of 100 μm , no adjustment was made.

Regarding the effect of frequency, it can be seen that at low frequencies G' is larger than G'' but not by a large amount. This means that over a long time range the viscous behaviour is more pronounced and the fluids are more liquid-like. As the frequency increases, the two curves cross over at about 0.01 Hz. Beyond this point elastic behaviour clearly dominates as can be seen by the higher G' values. The reason for the observed time-dependent behaviour can once again be explained by the chain entanglement theory, as follows. Because the polymer chains are entangled but constantly sliding over each other like 'snakes in a snake pit' (Barnes, 2000), they form a three-dimensional network structure such as the one shown in Figure 2.38. If a stress is suddenly applied to this transient network, initially all the long-chain molecules are stretched and the fluid will behave more elastically. However, given time the molecules can relax and disengage from each other so the fluid will show more viscous behaviour.

To properly show the effect of polymer concentration, Figure 3.47 plots G' and G'' against c_p at a common frequency of 1 Hz. Similar to that which was seen for the amplitude sweep tests (Figures 3.43), these curves have a parabolic shape and the rate of increase for G' is faster than that for G'' . This means that although both moduli increase as a result of higher concentrations, the fluid gradually becomes more solid-like due to the increasingly stronger molecular structure. This transition is also shown in Figure 3.46 by the gradually reducing frequency at which crossover ($G' = G''$) occurs.

To complete the discussion, it will be also useful to compare the HPAM results with those of other polymer fluids. Figure 3.48 shows the frequency response of a CMC fluid which has a molecular weight of 7×10^5 g/mol and a degree of substitution (DS) of 0.65 to 0.85 (Benchabane & Bekkour, 2008). It can be seen that, over the whole frequency range, G'' is larger than G' at low concentrations but the opposite is true at

high concentrations. Benchabane & Bekkour attributed this behaviour to the change of the solution microstructure from ‘semi-diluted entangled’ to ‘concentrated entangled’ as defined by Graessley (1980). This behaviour was not seen in the HPAM fluid because the concentration range investigated is much narrower. However, based on the trend line of the crossover point ($G' = G''$) shown in Figure 3.46, it would appear that if a much higher concentration of, say, 5 kg/m^3 (0.5% wt.) is used, then the G' curve would be expected to plot completely above the G'' curve over the frequency range down to 0.001 Hz.

3.5.6 Creep Tests

As discussed previously in Section 3.5.2, the measurement of creep is another way to characterise the viscoelasticity of a material. However, unlike the frequency sweep test which characterises the material’s behaviour over a wide range of frequencies (time scales), the creep test only examines the long-term deformation behaviour and hence it is more suitable for applications involving a very long time scale such as the sedimentation of small particles.

Creep tests were carried out using the same batch of HPAM fluids that were used for the oscillatory tests. A cone-and-plate geometry was used, where the cone had a diameter of 40 mm and an angle of 4° . The input shear stress was 0.006 Pa and the test period was 8000 s. These settings were chosen such that steady-state response can be reached well before the end of the test, and that the resulting steady-state viscosity (η) is also the zero-shear viscosity (η_0). The test results were fitted with the multi-element Burgers model (Equation 3.15) and the calculated parameters are given in Table 3.4. For comparison, the creep curves were also analysed by the Ninomiya method for the steady-state viscosity. A description of this method was not given previously but can be found in Ninomiya (1963) and Ferry (1980).

Figure 3.49 shows the creep test results and the fitted curves of the Burgers model. As can be seen, the compliance (normalised strain) is highly dependent on the polymer concentrations, and in all cases the multi-element Burgers model was able to track the response very well. The equilibrium compliance, J_e , which represents the (lack of) elasticity of the materials, was also calculated. This was done by adding the calculated discrete compliances J_i together according to Equation 3.16, although this could also have been

done graphically by extrapolating the linear portion of the curve back to time zero as illustrated in Figure 3.40. The first method was chosen since the individual J_i values were already available from the results and a higher accuracy could also be achieved.

To illustrate the effect of polymer concentration, Figure 3.50 plots the equilibrium, J_e against concentration, c_p . It can be seen that J_e decreases as c_p increases and that the pattern can be modelled with a power law. When interpreting creep test results, it should be borne in mind that the compliance represents the normalised strain or the reciprocal of the material stiffness (Equation 3.10). Therefore, a reduction in compliance shown in this figure can be interpreted as an increase of the material stiffness, which is caused by the increasing number of chain entanglement as previously illustrated in Figure 3.45.

Figure 3.51 shows the calculated steady-state viscosity, η , against concentration, c_p . It can be seen that the η values given by the two methods are quite similar, but those of the Burgers model are generally higher. Again, the results can be said to follow a power-law relationship. It can also be seen that the η values given in this figure and in Table 3.4 are similar to the η_0 values obtained previously from the viscometry tests (Figure 3.18 and Table 3.2). This is expected because as mentioned before the stress used in the creep tests is very small (0.006 Pa) and hence η should be the same as η_0 .

Lastly, it is also interesting to see the values of the instantaneous compliance, J_0 , being zero for all polymer concentrations (Table 3.4). This means that, in the analogy of the mechanical model, the uncoupled spring in the Burgers model has no role in the modelling of the polymer fluids since the elastic response is always delayed. The amount of the delay is shown by the relative values of the individual J_i each corresponding to a different retardation time. The finding of a zero J_0 is actually not surprising, because more than half a century ago Ferry (1948) already noted that, as a first approximation, the behaviour of a polymer fluid can be represented by the retarded Maxwell model which is basically the Burgers model with the uncoupled spring removed.

3.6 Conclusions

This chapter has examined the rheological properties of polymer fluids used for foundation construction. In particular, the properties of high-molecular-weight HPAM fluids were investigated in greater detail since they are more commonly used than other fluid types.

The work has been divided into two main parts, that is, the study of flow behaviour under equilibrium conditions and the study of viscoelasticity at different time scales. In each part, the current state of the knowledge was reviewed and the fundamental concepts were given. These were followed by the specific test results and analyses. Three main conclusions can be drawn from the results:

- The rheological behaviour of a polymer fluid is highly dependent on the composition and fundamental properties of the polymers. The difference in rheological properties can be huge for different types of polymer, but also can be significant for materials of the same type. For HPAMs, the two key controlling factors are the molecular weight and the degree of hydrolysis because together they dictate the properties of the solution microstructure. Other influencing factors include the supplied polymer form (granular or emulsion), particle size, solubility, solid content, and, of course, the dosage of the material. For polymers which carry charges along their backbones (e.g. HPAM and CMC), the ionic strength of the solvent also can play a significant role in the rheological properties because the charge repulsion effect can be greatly reduced or completely removed by other available ionic species.
- The rheological properties of polymer fluids are highly dependent on the governing time scale. In terms of steady-state flow, the viscosity at low shear rates can be several orders of magnitude higher than that at medium to high shear. Similarly, the viscoelastic properties are also highly dependent on the governing frequency of the oscillation. In general, polymer fluids exhibit more elastic behaviour at high frequencies and more viscous behaviour at low frequencies.
- The rheological properties can be significantly affected by several site factors. These include the presence of air bubbles due to aeration mixing, mechanical degradation due to high-shear pumping, and the accumulation of soil particles in suspension due to repeated re-use. These effects should not be overlooked as they are frequently encountered on site. The effect of polymer ageing is also another consideration if the polymer fluids are to be kept for a long period of time.

As has been shown in this chapter, the rheological behaviour of polymer fluids is very complex but can be explained if analysed carefully. It is hoped that the results presented

in this and in last chapters will form the basis for the understanding of the different available polymer systems. Such an understanding is essential for analyses of a wide range of practical problems including trench stability analysis, settling rate of suspended soil particles, fluid maintenance strategy, etc.

Chapter 4

Application of Polymer Fluids to the Construction of Bored Piles

4.1 Introduction

It was discussed in Section 1.2 that the driving force behind polymer fluids has traditionally been the operational and environmental benefits, such as smaller site footprint, savings in mobilisation time, and reduction of construction waste. As these are all site related issues, the move from bentonite to polymer fluids has mainly been driven by the foundation contractors. For example, when the author was working on the West Rail project in Hong Kong in 2001, it was the foundation (piling) contractor Trevi who proposed to use polymer fluids to reduce construction time and costs. This request was approved by the design consultant Maunsell (now AECOM), since the same polymer system (SlurryPro CDP) was also being used for the Taiwan High Speed Rail Link project (Contracts 260 & 270) where Maunsell was also the consultant (Corbet *et al.*, 2005). Although this pattern of ‘technology transfer’ was very common in those days, it is about to change as design consultants are getting more aware of the potential benefits that polymers can bring to the performance of the foundations. However, should a design engineer want to take advantage of the polymer fluids in the design phase, especially without the luxury of a site-specific load test, (s)he will need to know the answers to the following questions:

- What is the amount of increase in the strength and stiffness of the soil when compared to other stabilisation methods, such as water, bentonite or full-length casing;
- If there is an increase in the soil properties or foundation performance, what are the reasons and what are the necessary fluid properties to guarantee such improvement. Similarly, if there is a reduction, what are the reasons and how to prevent it;
- Are there any adverse effects on the structural integrity of the foundation, such as the compressive strength of concrete;

- Whether the existing pile construction specifications apply to polymers, such as the 12-h maximum pile bore open time as specified by SPERW in the UK (Institution of Civil Engineers, 2007). Note that according to SPERW the time period starts when excavation below the temporary surface casing commences and stops when concrete is placed.

With these questions in mind, this section first examines the available case histories from the literature for the current state of the knowledge. The case histories are grouped by their geographical locations to highlight the effects of local practice and geological conditions. To specifically answer some of the above questions, the results and the full analysis of a pile load test programme carried out at an East London site are presented.

4.2 Literature Review

4.2.1 Europe

Corbet *et al.* (1991) reported one of the first case histories of piles constructed using polymer fluids. The site was in Norfolk and the ground conditions consist of very weak chalk. To assist in the design and in the selection of a suitable construction technique, three test piles were constructed using three types of support fluids: water, polymer, and bentonite. Since no information about the polymer was mentioned in the paper, its type and properties are not known. However, since these test piles were constructed in 1989 and during that period a commercial natural polymer known as ‘Viskeau’ was being promoted in the UK (Beresford *et al.*, 1989; White Jefferis & Associates, 1989), it is possible that the polymer used was Viskeau. From the load test results, it was found that the polymer pile significantly outperformed the other two in terms of their load–settlement characteristics, even though the polymer pile was fitted with a ‘soft toe’ of polystyrene when the other two were not. Note that the term ‘soft toe’ is used to describe a layer of soft sediment at the base (toe) of a pile often caused by insufficient base or fluid cleaning prior to concreting. A ‘soft toe’ is almost always undesirable but in this case it is artificially fitted to the pile to remove the effect of base resistance on the overall pile response.

It is interesting to note from the results that although the water-supported pile bore

had the most irregular sidewall profile and the largest actual diameter as measured by a caliper, it showed the largest settlement. In the author's view, this indicates that the water was unable to stabilise the pile bore thus causing significant stress relief in the soil and sloughing of the sidewall. It was also noted in the article that a temporary casing of at least 15 m was necessary to ensure the stability of the very soft top soil (SPT $N < 10$ blows/0.3 m). This highlights the fact that support fluids, regardless of their types, simply cannot adequately support very soft soil and temporary casings have to be used or collapses will occur.

Wheeler (2003b) reported another case history in the UK in the form of a professional news article. In the trial, three polymer piles and one bentonite pile were tested at a site in Canary Wharf in East London. The aim of the trial was to see whether the pile design, based on the assumption of using bentonite fluids, could be adopted for piles formed using polymer fluids. Although the load test results were not presented in the article, it was reported that both the shaft and base resistances of the polymer piles were very good. As a result, a 4-m reduction in the pile length was allowed in the design of the working piles. It was also reported that the polymer fluids did not produce significant shaft degradation with extended pile bore open time (it seems that the three polymer piles had different soil–fluid exposure times but this was not made clear in the article). This led to the conclusion that the bores of the working polymer piles could be left open overnight to allow better use of site resources. Several other practical issues were also mentioned in this article, such as the need for the contractors to change their production methods when using polymers, the effect of polymers on rebar–concrete bond, and the potential adverse effects of some proprietary additives on the piles.

In the discussion of BBGE's experience of using polymer fluids in the Glasgow area, Lennon *et al.* (2006) presented the load–settlement curves of two test piles from two different sites. The ground conditions at Glasgow typically consist of very loose becoming very dense silty sand for the top 25 m and then the bedrock. The groundwater table is associated with the River Clyde and is typically at between 4 m and 5 m depth. The polymer used for the test piles was SlurryPro CDP from KB International. Since the load–settlement curves of both piles showed a near linear relationship up to the proof

load, it was concluded that the piles had very good contact with the soil both along the shaft and at the base. Similar load–settlement response was also seen internally by BBGE at several other sites, such as the Anderston site in Glasgow (SlurryShield) and the University College Hospital site in central London (SlurryPro CDP).

Relatively little information can be found about the use of polymer support fluids in other European countries. This is probably because the experience tends to get reported in the local languages and this literature review is mainly confined to publications in English. However, from the review, it was found that several papers have been written about the testing of the polymer piles constructed for the Vasco da Gama Bridge in Lisbon (Bustamante *et al.*, 1998; Manuel Correia & Sêco e Pinto, 1999; Guadagnini, 2001). The polymer fluid used for this project was ‘Geomud-15’. Note that this polymer product was not discussed in the previous chapters, because very little information can be found in the literature apart from the description that it is a high-molecular-weight polymer (Bustamante *et al.*, 1998). In summary, it was found that all the pre-construction test piles performed reasonably well, exceeding the predicted resistance in two out of three cases. Therefore, the working piles were drilled using the same polymer fluid. However, it was noted by Bustamante *et al.* (1998) that two of the working piles had to be re-drilled using another polymer SlurryPro CDP since the Geomud-15 fluids became contaminated by the ground. KB international, in one of their advertising documents, explain that the fluids were contaminated by the large amount of calcium and magnesium ions in the ground due to the nearby salt flats (KB Technologies, 2000). This incident highlights the importance of fluid maintenance and also polymer selection to the success of a piling project.

Another European case history of the performance of polymer piles was reported in Bustamante *et al.* (1998). The project was the Naples–Rome section of the Italian High Speed Train (TAV) project; the polymer used was Geomud-15. The results of two pile tests carried out at two different sites (Peccia and Cassino Sud) were reported. Based on the measured load–settlement and unit shaft resistance curves, it was concluded that the piles performed satisfactorily. However, despite the promising test results it was reported that polymer fluids were only used at sites where the ground conditions consist of

pyroclastic soil (pumice and lapilli) but not at sites underlain by alluvial deposits, which included silty clay, silty sand, clayey sand, sand and gravel. At those sites bentonite fluids were used instead. Although no reasons were given as to why polymer fluids could not be used to support the alluvial deposits, it is possibly due to the common perception that polymer fluids are less useful in stabilising loose coarse soil because unlike bentonite they do not possess a gel strength.

4.2.2 North America

Since polymer support fluids were first developed and used in the United States, it is not surprising to find a larger number of case histories from this country. Holmes (1993) reported one of the first uses of SlurryPro CDP fluids in the US for the construction of a new bridge over the Ohio River in Kentucky. The ground conditions consist of sand and gravel overlying water-sensitive shale. It was reported that the original design was actually driven piles but was later changed to bored piles (drilled shafts) due to the ability of the SlurryPro CDP polymer to preserve the integrity of the shale as shown by the results of some laboratory immersion tests. To verify the design assumptions, at each of the two main pier locations a test pile was constructed under the conditions of actual construction. At the base of each pile an Osterberg cell (O-cell) with a capacity of 26.5 MN was fitted for the proposed bi-directional load test. In two related articles, Goodwin (1993a, 1993b) presented the test results and the equivalent surface load–settlement curve (as if the pile was loaded from the surface) for one of the tests. Although the performance of the test pile was not discussed in detail, it was reported that the results of the first load test led to the decision to reduce the rock socket length of the second test pile, and that in light of the test results the original design could have used fewer piles. From these descriptions, it is evident that the performance of the polymer fluids exceeded the expectation of the foundation engineers.

It is interesting to note that the Ohio River case history is not the only one which mentions the preserving effect of polymer fluids on shale. Axtell *et al.* (2009) and Brown & Axtell (2010) described the use of the HPAM polymer ‘POLY-BORE’ for another bridge project in the Missouri River in Kansas City. From the results of slake durability tests, the polymer fluid was found to be able to preserve the integrity of the shale better than

river water alone. From the test pile results, it was found that, even though the pile bore was deliberately left open for a period of 4 days, the mobilised unit shaft resistance was still similar to those developed by piles in other western Missouri shale. The preserving effect of HPAM polymers (EZ Mud, EZ Mud DP, Super Mud, and Super Mud Dry) on Missouri shale was also studied by Likos *et al.* (2005) through a series of laboratory tests involving jar slake, drum slake, linear swell, and hardness tests. It was found that the inhibitive effects of the polymers on the slaking and softening behaviour of shale were very sensitive to polymer concentration but not so much to the polymer type. It is worth noting that although these studies confirm the usefulness of polymer fluids for drilling in shale, the results are actually not surprising at all, since HPAM has long been known to be a shale inhibitor in the petroleum industry and has been used extensively for oil and gas drilling (e.g. Scheuerman, 1973; Clark *et al.*, 1976; Liao & Siems, 1990). Other chemicals known to inhibit the degradation of water-sensitive shale also include potassium chloride (KCl) and xanthan gum polymer (Darley & Gray, 1988). A brief discussion of the various polymers used for oil well drilling has been given in Section 2.2.1.

The performance of polymer piles in other soil types was investigated in several studies, including Brown *et al.* (2002) (silty sand), Brown (2002) (silty and clayey sand), Camp *et al.* (2002) (sand and clay), Frizzi *et al.* (2004) (sand and sandstone), and Ata & O'Neill (1997a, 1998) (sand and clay). In summary, these studies show that the measured unit shaft resistance of a polymer pile is generally greater than that of a bentonite pile, and the back-calculated α and β values¹³ often exceed those recommended by the local design guidelines. For sand and silt, the improvement is attributed to the absence of a soft bentonite filter cake at the pile–soil interface (Brown, 2002), whereas for clay, it has been suggested that the improvement is due to a rougher pile concrete surface caused by a chemical interaction between the polymer and the concrete (Ata & O'Neill, 2000; O'Neill, 2001). Since filter cakes cannot form on the surface of low-permeability clay, the amount of improvement in shaft resistance is believed by Brown (2004) to be greater in sand than in clay, based on the results of Camp *et al.* (2002).

Despite the clear evidence of improvement in shaft resistance as shown by these studies,

¹³For readers with a non-geotechnical background: α and β are empirical pile design parameters based on the soil mechanics principles of total stress and effective stress respectively.

the use of polymers is not without problems. There is field evidence to suggest that polymer fluids cannot hold any dislodged soil particles in suspension for long periods. This could cause contamination at the top of the concrete column and (or) a ‘soft toe’ at the completed pile (Brown, 2004). For example, Frizzi *et al.* (2004) reported the construction and load test details of two polymer piles and one bentonite pile. It was found that there was more sediment at the base of the polymer-supported bores than in the bentonite-supported bore. As a result, the base resistances of the polymer piles were significantly lower than that of the bentonite pile. The sediment at the base of the polymer piles caused larger settlement at the pile head under large loads when the shaft resistance was fully mobilised. This common issue of ‘soft toe’ with polymer piles is also probably the reason why Texas Department of Transportation (2003) found excessive settlements at those bridges which were founded on polymer piles in their state.

To complete this brief discussion of the North American case histories, it should be mentioned that polymer fluids also have been used in Canada but apparently at a much smaller scale. At the time of writing this thesis, only one documented case history has been found, i. e., the Golden Ears Bridge across the Fraser River in British Columbia, Canada (Amini *et al.*, 2008). The polymer type was not mentioned in the article but is believed to have been a product from KB International according to their website. In summary, it was found that the back-calculated α value of the polymer test pile was significantly higher than that recommended by O’Neill & Reese (1999) for bored piles, and it was close to that recommended by the American Petroleum Institute (2000) for driven piles. This finding is similar to those of the other studies already discussed.

4.2.3 Asia

The Asian case histories available in the literature are currently concentrated in three countries: Thailand, Singapore, and Taiwan. In Thailand, the case histories are all located in Bangkok and can be found in Littlechild & Plumbridge (1998), Thasnanipan *et al.* (2002), and Thasnanipan *et al.* (2006). The common and unique feature about these projects is that the polymer fluids used were not mixed in water but in dilute bentonite slurries. According to Littlechild & Plumbridge (1998), this was done because high fluid loss into the sand layers was observed at the polymer-supported trial bores excavated for

a local railway project known as BERTS, but later it was found that the amount of fluid loss could be significantly reduced if the polymer fluid was mixed with water pre-loaded with bentonite at a concentration of 5 kg/m³. Because of this finding, the practice of using dilute bentonite slurries as the mix ‘water’ for polymers was adopted for the construction of the working piles for the BERTS project. This practice apparently spread to other projects as well. For instance, Thasnanipan *et al.* (2002) reported the same fluid loss problem and the same solution for another polymer piling project in Bangkok.

In terms of pile performance, all the studies reported that the use of polymer-bentonite fluids would lead to improvements in shaft resistance when compared to pure bentonite fluids. In the BERTS project, Littlechild & Plumbridge (1998) also found that the shaft resistance of the piles did not seem to be affected by the construction time which ranged between 14 and 41 h. This finding is cited in the well respected foundation design reference book by Tomlinson (2001). Despite the apparent advantages of using the mixed polymer–bentonite fluids, the author has reservations about this practice since, although the bentonite dosage is small, filter cakes can still form on the excavation sidewalls and reduce the shaft resistance. In fact, the adverse effect of adding bentonite to polymer fluids was also noted by a local study reported by Thasnanipan *et al.* (2003). In this study, a series of laboratory interface shear box tests were carried out. The results showed that the interface resistance between the cement mortar (simulating the pile concrete) and the sand was reduced by both pure bentonite and polymer-bentonite fluids and that there is little difference between them. To overcome the high fluid loss problem in areas such as Bangkok, the author believes that the correct method is to use the existing fluid loss control additives (Table 2.1) as they can plug the pores of the sidewalls without forming a filter cake at the surface. Why this was not done in Thailand is unclear.

It is worth noting that similar polymer–bentonite fluid systems are also used in the petroleum industry but for very different reasons. For deep oil well drilling, it is often found that drilling fluids with a high solids content cause reductions in the penetration rate, and hence part of the bentonite is often replaced by polymers (usually PAC, or xanthan gum) in order to reduce the amount of solids in the fluid and yet to be able to keep the cuttings-carrying capacity. This kind of fluid system is commonly known as the

‘low-solids mud’ by the petroleum engineers (Darley & Gray, 1988; Schlumberger, 2011).

In Singapore, two case histories of polymer use were given in Ho & Tan (1996) and Ho (2002). The first one was the construction of barrettes in cemented dense sand and the second one was the construction of bored piles in residual granitic soil. The common feature of these projects is that sediment was found at the base of the excavation at both sites. This problem was reported to be the cause of the poor pile base response at the first site and of the contaminated concrete at the pile base at the second site. An examination of the fluid test records of the first project reveals that the Marsh funnel viscosity of the polymer fluid was only between 28 and 32 s. These values are extremely low by any standard because water alone can give a Marsh funnel viscosity of around 26 ± 0.5 s. Because the viscosity of the polymer fluids was very low, they would not have the ability to hold any dislodged soil particles in suspension. Under this fluid condition, the accumulation of soft sediment at the pile base was therefore inevitable unless rigorous base cleaning procedure was followed at the end of the excavation. It is interesting to note that, despite the low fluid viscosity, no stability problem was reported at either of the sites, and that the back-calculated unit shaft resistance was comparable to those obtained from other construction techniques.

In Taiwan, SlurryPro CDP polymer fluids were used on Contracts 260 and 270 of the Taiwan High Speed Rail Link project as noted in Section 4.1. Some of the load test results were given in Bustamante & Boato (2005) and Corbet *et al.* (2005). In summary, the test results showed that the unit shaft resistance in clay was higher than the original design value, but no improvement in shaft resistance was found for the sand and gravel. This finding differs from the suggestion of Brown (2004) that the improvement in shaft resistance is more pronounced in sand than in clay. Once again, a weaker-than-expected unit base resistance (end-bearing pressure) was concluded from the back-analyses, and this prompted a revision of the design equations and more careful base cleaning of the working piles. The repeated occurrence of ‘soft toe’ in different countries suggests that it is indeed a very wide spread problem and should not be ignored.

4.2.4 Africa

Only two case histories of polymer use in Africa can be found. They are the construction of a new bridge over the Lake Nokoué (a lagoon) in Benin (Schwarz & Lange, 2004; Bustamante & Boato, 2005) and the João Landim Bridge in Mansôa, Guinea-Bissau (Bustamante & Boato, 2005). In the Benin case, 56 to 72 m long, 1.2 m diameter bored piles were needed to support the proposed bridge structure. The ground conditions consisted of loose to medium dense sand (0 – 15 m), soft becoming stiff silty clay (15 – 55 m), and fine to medium sand (> 55 m). Following a series of field trials, it was found that collapses repeatedly occurred at about 48 m to 66 m depth when the pile bores were supported with Geomud-15 or Novagel polymer fluids (see Schwarz & Lange, 2004 for details about these failures). Further laboratory investigations revealed that the soil was very rich in sodium, potassium, chloride, and sulfate (note that the site was a lagoon). Therefore, it was believed that the collapses were caused by the contamination of the polymer fluids by the ions from ground (note: see Figure 2.35 for the effect of added sodium ions from NaCl on the viscosity of some HPAM fluids). Because of the poor performance of the polymer fluids, polymer-enhanced bentonite slurries were used instead. After the switch, no more stability problems were reported and the project was completed successfully. It is interesting to note that this case study is actually rather similar to the Vasco da Gama Bridge case (Section 4.2.1) where Geomud-15 also failed due to contamination by the salts in the ground. These two case histories highlight the importance of choosing a suitable support fluid for the specific ground conditions of a site.

In the Guinea-Bissau case, Geomud-15 was proposed for the construction of some 55 to 75 m long, 1.8 to 2.0 m diameter bored piles for the bridge. The site was underlain by about 50 to 70 m of clay and then the limestone bedrock. The strength of the clay was described as soft at the top, gradually becoming hard near the bottom. A trial undertaken on the river bank showed that the polymer fluid was unable to support the pile bore at 64.5 m depth, and this was believed to be due to the presence of numerous medium to coarse sand bands found in the hard clay. Other attempts to drill using different dosages of Geomud-15 also failed. As a result, the contractor decided to support the pile bore with a full-length casing, and only used the polymer fluids as a lubricant to aid the installation

of the casing through the clay.

4.2.5 Summary

Based on the above literature review, the following conclusions can be drawn:

- Support fluids, regardless of their types, are unlikely to be able to adequately support very soft top soil and a temporary casing has to be used;
- Polymer fluids are deemed by some practitioners to be unsuitable for use in coarse-grained soil, although it has never been formally demonstrated to be the case;
- If no stability problems are encountered during the excavation and the ground does not contain high levels of salts, polymer fluids should be able to continue to maintain the stability of the pile bore for extended periods of time, and the resulting shaft resistance should not be severely affected by this;
- Polymer fluids are good at preserving the integrity of water-sensitive shales but this ability is dependent on the concentration used – a polymer-specific minimum concentration must be respected;
- Back-calculated α and β values of polymer piles often, though not always, exceed those currently recommended by design guidelines for bored piles. The author has not identified a single case history where the unit shaft resistance of a polymer pile was reported to be lower than that of a bentonite pile;
- For coarse-grained soil, the improvement in shaft resistance is probably due to the absence of a soft filter cake on the excavation sidewall. For fine-grained soil, the improvement is sometimes attributed to a rougher concrete surface caused by the chemical interaction between the concrete and the polymer;
- In the case histories, despite the reported improvements in shaft resistance and settlement with the polymer fluids, the effect on soil stiffness was rarely mentioned;
- ‘Soft toe’ is a very wide spread problem and often results in poor response from the pile base. The inability of polymer fluids to hold particles in suspension is believed to be the cause of this problem;
- Failures of polymer fluids are often associated with contamination by the salts in the ground;

- High fluid loss can be experienced under certain ground conditions. Mixed polymer–bentonite fluids have been successfully used to tackle this problem, but probably at the cost of a lower shaft resistance compared to pure polymer fluids;
- It is not known whether polymer fluids affect the structural integrity of the pile as there is little information about this in the literature.

4.3 Background to Stratford Field Trial

4.3.1 Purpose of the Trial

As mentioned in the literature review, there are only a few published case histories of polymer piles in UK, and some of them are only descriptive rather than giving full test details and analyses. Therefore, some British engineers and their clients are still sceptical about the performance of polymer fluids and, when they are used, prefer to stick to the existing design rules which were largely developed based on the case histories with bentonite. For example, during the construction of the bored piles at the Castle House site (Section 3.4.7), BBGE had to reduce the design α value from 0.5 to 0.35 for any polymer-supported bores that were left open overnight, effectively increasing the length of those piles. To address this issue and to provide further evidence of the effect of polymer fluids, it was decided to carry out a field trial comprising three bored piles at a construction site located in Stratford, East London. The specific objectives were as follows:

- To assess the difference between a polymer pile and a bentonite pile in terms of their settlement characteristics and the unit shaft and base resistances;
- If the use of polymer fluids cause improvements in the above properties, why and whether this could be lost by an extended open time of the pile bore;
- To back-calculate α and β values from the measured shaft resistance and compare them with the current design practice;
- To assess the effect of fluid intermixing on the strength and stiffness of the hardened concrete.

To achieve these objectives, three full-scale bored piles were constructed side by side at the test site, the details were as follows:

- Polymer pile P1: To be excavated using polymer support fluid (SlurryPro CDP) and concreted on the same day;
- Polymer pile P2: To be excavated using polymer support fluid (SlurryPro CDP) and concreted on the next day;
- Bentonite pile B1: To be excavated using bentonite support fluid (Berkbent 163) and concreted on the same day.

4.3.2 Works Arrangements

As the construction and testing of piles is a highly specialised task, several companies were commissioned by BBGE to carry out the actual pile load and material testing. The companies involved in the trial and their roles are:

- Balfour Beatty Ground Engineering Ltd (BBGE) – construction of test piles;
- Precision Monitoring & Control Ltd (now Environmental Scientifics Group) – pile load tests and cross-hole sonic logging tests;
- Sandberg LLP – concrete compression tests;
- Testing and Analysis Ltd (Testal) – small-strain pile integrity tests.

During the trial, the author was on site to observe and document the various construction activities and also to carry out fluid testing. After the field trial, the test results from each companies were given to the author for analysis and interpretation. During this stage, help from BBGE was given to the author but this was limited to the provision of site information and general discussion of the results. No engineers from BBGE were involved in the analysis, and all the results given in this thesis are the work of the author unless otherwise stated.

4.4 Site Conditions

4.4.1 Site Location and Background

The test site is located at about 0.5 km south-east of the Stratford International station ‘box’ in East London. Figures 4.1 (a) and (b) show two aerial photographs of the site.

This site sits on the surrounding land that was raised by about 7 m using the excavated materials from the Channel Tunnel Rail Link (CTRL) tunnel and the station box (Dyson & Blight, 2007). Since the site was being developed for the Stratford City project, a site investigation consisting of exploratory boreholes (BHs) and cone penetration tests (CPTs) had already been carried out near the test piles (Norwest Holst Ltd, 2008). The locations of the BHs and CPTs are shown in Figure 4.1c.

4.4.2 Soil Profiles

From the borehole logs and by examination of the soil cuttings during the excavation of the test piles, the site was found to be underlain by the made ground/land raise, Lambeth Group, Thanet Sand and then Chalk. The Lambeth Group is a stratigraphic group consisting of Woolwich, Reading, and Upnor Formations. At this site, the Woolwich Formation consists of two members, namely the Laminated Beds and the Lower Shelly Clay. The depths and geotechnical descriptions of these soil layers according to British Standards Institution (1999) are given as follows:

Made Ground/Land Raise

5.5 mOD to -0.5 mOD (0 m to 6.0 m b.g.l.)¹⁴

MADE GROUND: Dark brown gravelly fine to coarse sand with occasional shell fragments and pockets of stiff clay. Gravel sized fragments are angular to subrounded fine to coarse flint, chalk, brick, concrete, and wood.

River Terrace Gravel

-0.5 mOD to -1.0 mOD (6.0 m to 6.5 m b.g.l.)

Brown fine to coarse SAND and rounded to angular fine to coarse GRAVEL of flint.

Lambeth Group – Woolwich Formation – Laminated Beds

-1.0 mOD to -4.0 mOD (6.5 m to 9.5 m b.g.l.)

Firm greyish brown slightly sandy thinly laminated CLAY with closely spaced thin to thick laminations of silt and occasional shell fragments

Lambeth Group – Woolwich Formation – Lower Shelly Clay

-4.0 mOD to -5.5 mOD (9.5 m to 11.0 m b.g.l.)

¹⁴OD stands for Ordnance Datum. b.g.l. stands for below ground level.

Stiff dark greyish green sandy CLAY with many shells and shell fragments

Lambeth Group – Reading Formation – Lower Mottled Clay

–5.5 mOD to –8.0 mOD (11.0 m to 13.5 m b.g.l.)

Stiff to very stiff green very sandy slightly gravelly CLAY with occasional shell fragments. Gravel is subrounded to rounded fine to coarse flint. From 12.5 m to 13.5 m b.g.l., light green limestone hardpan (calcrete duricrust)

Lambeth Group – Upnor Formation

–8.0 mOD to –14.0 mOD (13.5 m to 19.5 m b.g.l.)

Very stiff dark green very sandy gravelly CLAY with occasional shell fragments at top. Gravel is rounded to subrounded flint.

Thanet Sand

–14.0 mOD to –27.5 mOD (19.5 m to 33.0 m b.g.l.)

Very dense grey slightly silty fine to medium SAND. From –25.75 mOD, becoming silty.

Chalk

–27.5 mOD to end of borehole at –29.5 mOD (33.0 m to 35.0 m b.g.l.)

Structureless CHALK (Grade Dm, Mundford Grade VI)

4.4.3 Groundwater

Due to the geological conditions in the London Basin, two aquifers exist at the test site. The shallow aquifer is in the made ground and River Terrace Gravel, and the deep aquifer is in the Thanet Sand and the underlying Chalk. These two aquifers are separated by an aquiclude which is the cohesive Lambeth Group at the test site and the London Clay in other parts of London. According to Whitaker (2004), the groundwater table in the deep aquifer near the Stratford station box had been steadily rising since the late 1950s, and in 1997 the piezometric surface had reached about –11 mOD. However, for the construction and operation of the CTRL Stratford box, a permanent dewatering scheme was required to depress the piezometric level back to a minimum of 10 m below the base slab level of the box (the slab level is approximately –10 mOD at the West end and –15 mOD at the East end). As a result, dewatering wells were installed at both ends of the box

(Figure 4.1*b*) and they have been pumping continuously since late 2001.

To investigate the actual location of the groundwater table in the deep aquifer, measurements were made during a site investigation in 2007 and the data were collated by Arup who was the design consultant of the Stratford City project (Ove Arup & Partners Ltd, 2008). Figure 4.2 shows the measurement results against the distance from the dewatering wells. A probable draw-down profile is also shown in the figure. It can be seen that the groundwater table stood at about -25 mOD at the location of the test pile, which is about 500 m from the wells at the East end of the box.

To investigate the pore water pressure (u) in the different soil layers, measurement data were also collated by Arup and are plotted in Figure 4.3. It can be seen that the groundwater table in the shallow aquifer stood at between 1.5 and 3.0 mOD, and that of the deep aquifer stood at between -20 and -26 mOD. Because of the under-drainage, the pore pressure in the Lambeth Group was found to be below hydrostatic. Similar pore pressure conditions in the Lambeth Group at other sites were also reported by Hight *et al.* (2004). As the groundwater table in the deep aquifer at the test pile location has already been established, a possible continuous profile can therefore be drawn. In this idealised profile, the pore water pressure between -14 and -25 mOD, i.e., the top 11 m of the Thanet Sand above the groundwater table, is assumed to be zero. This assumption is different from the one adopted by Georgiadis *et al.* (2003) for their finite element analysis of pile behavior in the similar Canary Wharf ground conditions. Georgiadis *et al.* assumed that the pore water pressure in the Thanet Sand is hydrostatically negative above the groundwater table in the deep aquifer. While this may be a reasonable assumption in some cases, the presence of many possible routes (e.g. groundwater monitoring wells, exploratory boreholes) for air entry into the Thanet Sand at this site means that the negative pore water pressure is likely to be short-lived. In a discussion of the numerical modelling of groundwater pumping from the deep London aquifer, Marvroulidou *et al.* (2005) also expressed a similar view on the likelihood of air entry into the Thanet Sand. Finally, it is also worth noting that, for the actual pile design, Arup adopted a more conservative pore pressure profile in which the piezometric level of the deep aquifer stood at -10 mOD instead of -25 mOD (Ove Arup & Partners Ltd, 2008). This was done to

ensure any future rise in the groundwater regime would not invalidate the pile design.

4.4.4 Engineering Properties

As part of the year 2007-2008 site investigation, in-situ tests were performed at the test site and laboratory tests were carried out on the recovered soil samples. This section briefly discusses some of the results, as they are essential to the back-analysis and understanding of the load test results.

Moisture Content: Figure 4.4 shows the moisture contents of the soil samples recovered from the nearby boreholes. It can be seen that the moisture contents of the Lambeth Group appear to decrease from about 30% at top (-1 mOD) to about 15% near the bottom (-14 mOD). For the Thanet Sand (-14 mOD to -27.5 mOD), the moisture content typically ranges between 30% and 35% but there are also a few data points lying below 20%. These low values are probably caused by air entry into the system and hence partial desaturation of the sand.

Bulk Density: Figure 4.5 shows the bulk density of the soil, ρ_b . Three types of data are given in the plot: 1) density of the Lambeth Group samples measured as part of a triaxial test, 2) density inferred from the moisture content, and 3) density inferred from the specific volumes of intact Thanet Sand samples obtained from the CTRL Stratford site and reported by Ventouras (2005). Note that the last two types of data are calculated using phase relationships and assuming complete soil saturation, and hence this assumption may not be true for the Thanet Sand samples for the reasons described. From the plot, it can be seen that the bulk density of the Lambeth Group appears to increase from about 1.95 Mg/m^3 at the top of the layer (-1 mOD) to 2.25 Mg/m^3 at the bottom (-14 mOD). With regard to the Thanet Sand, the two types of inferred density data correspond reasonably well with an average value of around 1.90 Mg/m^3 . Note that this density is likely to be very close to the maximum density of the soil, since the highest achievable dry density, ρ_d , measured by both the author and the site investigation contractor, is about 1.44 Mg/m^3 which corresponds to a saturated density of 1.90 Mg/m^3 .

Cone penetration test (CPT): Figure 4.6 presents the results of the cone penetration tests carried out in the vicinity of the test piles. In these plots, zero depth corresponds to the ground level at the test pile location (5.5 mOD) and hence the top level of the

Lambeth Group is around 6.5 m b.g.l. (-1 mOD). Two curves are shown in each plot, i.e., the cone resistance, q_c , in MPa and the sleeve friction, f_s , in kPa. It can be seen from the plots that the made ground and the River Terrace Gravel at the top 6.5 m are highly variable in terms of q_c . This is not surprising since a wide range of soil types was reported to have been used for the made ground (Dyson & Blight, 2007) and man made materials such as bricks and wood fragments were also found from the exploratory holes. In the Lambeth Group, it can be seen that the Woolwich Formation (6.5 to 11.0 m b.g.l.) is noticeably weaker than the underlying Reading Formation (11.0 to 13.5 m b.g.l.). As indicated by the higher sleeve friction, the Reading Formation also appears to have a higher clay content than the Woolwich Formation, although both have been described as clay in the borehole logs. All the tests hit refusal at between 12.5 and 13.5 m, b.g.l., and this is believed to have been caused by the presence of a calcrete (duricrust) layer which was found at this depth during the excavation of the test piles. As a result, no information about the Upnor Formation and the Thanet Sand is available from the CPT results. Nicholson *et al.* (2002) also reported consistent CPT refusal before hitting the Thanet Sand at an Isle of Dogs site in East London.

Standard penetration test (SPT): Figure 4.7 shows the standard penetration test results. The N values, i.e., the number of blows per 300 mm of penetration, are taken from the site investigation records without any corrections applied. It should be noted that, in the Upnor Formation and in the underlying Thanet Sand, some of the tests were terminated at 50 or 100 blows without reaching the full 300-mm penetration. It can be seen from the figure that the N values are highly variable in the made ground and the River Terrace Gravel, but show a gradually increasing trend in the Woolwich and Reading Formations (-1 and -8 mOD). The actual trend in the Upnor Formation is less certain since most of the tests were terminated at 50 blows and hence the apparent vertical trend. The same can be said about the results obtained in the Thanet Sand until -25 mOD except that the tests were stopped at 100 blows instead. Between -25 mOD and the top level of Chalk at -27.5 mOD, the N values show a decreasing trend and this is believed to be due to the increasing amount of silt as mentioned in the soil descriptions (Section 4.4.2). Based on the test results, an idealised profile has been drawn and the portion in the

Lambeth Group (-1 to -14 mOD) was used for the derivation of the undrained shear strength, as follows.

Undrained shear strength: Figure 4.8 shows the undrained shear strength (S_u) of the cohesive Lambeth Group. Three types of data are given on the plot, 1) the results of unconsolidated-undrained (UU) triaxial tests on selected ‘undisturbed’ U100 samples, 2) a strength profile inferred from the idealised profile of the SPT N values using a correlation factor of 4.5 ($S_u = 4.5N$), and 3) strength profiles inferred from the CPT q_c profiles using a correlation (cone) factor of 20 ($S_u = q_c/20$). Note that the values of these correlation factors are only the author’s best estimates, since empirical correlations between the results of these in-situ tests and design parameters (e. g. S_u) have not yet been established for the Lambeth Group (Hight *et al.*, 2004). Nevertheless, it can be seen from the figure that the inferred S_u profile from the SPT plots nicely between the possible lower and upper bounds which are given by the UU test results and the inferred profiles from the CPTs respectively. Since the so-called ‘undisturbed’ U100 samples are commonly known to suffer some damage, this may explain why the UU test results form the lower bound of the plot. The reason why the CPT results form the upper bound is not known, but is possibly due to the high sand content of the clays giving higher values of q_c . Given that the inferred S_u profile from the SPT also corresponds reasonably well to the field description of the undrained strength of the Lambeth Group (see Section 4.4.2), it was decided to use this profile for back-analyses.

Effective stress strength parameters: Since only a few consolidated-undrained triaxial tests were included in the site investigation, the literature was consulted for the effective stress strength parameters of the Lambeth Group. Table 4.1 shows the peak angle of shearing resistance, ϕ'_p , and effective cohesion, c' , of the various units of the Lambeth Group as given by Hight *et al.* (2004). It can be seen that the value of ϕ'_p appears to increase with depth but it is also significantly affected by the sampling and testing techniques as well as the soil plasticity and clay content of the samples. Except for the Upper and Lower Mottled Clays, all the other units have zero effective cohesion. With regard to the properties of the Thanet Sand, the value of ϕ'_p is typically taken as 36° for pile design as this is the value commonly given by small (direct) shear box tests

on recompacted samples (e.g. Troughton & Platis, 1989; Troughton, 1992). It is also worth noting that Ventouras (2005) carried out some triaxial tests on intact Thanet Sand samples taken from the CTRL Stratford site. The value of ϕ'_p was found to be 38.7° from the undrained compression test and 34.1° from the drained compression test. From the drained extension tests, the value of ϕ'_p was found to be 29.9° . The critical state angle, ϕ'_{cs} , was found to be 32.3° independent of the silt content of the samples.

In-situ horizontal stress: Although several pressuremeter tests were carried out during the site investigation, they are of the pre-bored Ménard pressuremeter type and hence the in-situ horizontal stress cannot be reliably estimated from the test results. The literature was therefore again consulted for the likely values of the coefficient of earth pressure at-rest, K_0 , of the various soil layers. According to Hight *et al.* (2004), due to the recent stress history and heavy overconsolidation, the K_0 values in the clay units of the Lambeth Group would be expected to be greater than unity. The reported K_0 values (measured by self-boring pressuremeter tests) of several case histories were quoted, such as the average value of 1.5 from the Jubilee Line Extension project and a range of between 1.7 and 2.5 from several others. In a numerical study of pile behaviour in the Canary Wharf ground conditions, Georgiadis *et al.* (2003) used a K_0 of 1.15 for all the units of the Lambeth Group. The same K_0 value of 1.15 was also used by Georgiadis *et al.* and Troughton & Platis (1989) for the underlying Thanet Sand. Based on these results, it seems reasonable to assume a K_0 of 1.5 for the Lambeth Group and of 1.15 for the Thanet Sand.

Horizontal stress acting on pile shaft: Since this case study is about the performance of bored piles, it is just as important to know the value of the coefficient of earth pressure for the shaft, K_s , for the back-analyses. Based on the results of a pile load test carried out at the British Library site, O'Riordan (1982) found that K_s ($= \sigma'_h/\sigma'_v$) ranges between 0.85 and 1.06 in Lambeth Group. According to Hight *et al.* (2004), Chapman *et al.* (1999) also reported a back-calculated K_s value of 1.05 in the Lambeth Sand (not present at this site). These findings confirm the belief of Burland (1973) that the initial at-rest horizontal stress in the soil cannot be fully re-established at the shaft face. Considering all the available information, it was decided to use a single K_s

value of 1.15 for both Lambeth Group and Thanet Sand for the purpose of back-analysis. This equates to a 23% reduction in the estimated horizontal stress in the Lambeth Group, which is not far from the 20% reduction suggested for heavily overconsolidated clay by Fleming *et al.* (2009).

4.5 Test Pile and Construction Details

4.5.1 Test Pile and Instrumentation Details

To achieve the objectives set out in Section 4.3, three bored piles (P1, P2, and B1) were constructed side by side at the test location between 23th and 29th July 2008. Figure 4.9 shows a schematic cross section of the setup as well as a graphical summary of the ground conditions. Note that for the purpose of load testing and back-analysis, the River Terrace Gravel (-0.5 to -1.0 mOD) has been merged with the made ground, and the Woolwich and Reading Formations (-1.0 and -8 mOD) are now treated as one single unit. Figure 4.10 shows the setting out of the test and reaction piles. It shows that the distance between each test pile is 6.0 m, and that the distance between the test and reaction piles (4.2 m) is larger than three pile diameters (3.6 m). Figure 4.11 shows the details of the pile head in a cross-section schematically.

As shown in these figures, the test piles had an embedded length of 27.0 m and a stick-up length of 0.5 m. The nominal diameter was 1.20 m but the actual outside diameter for the top 8 m was 1.26 m due to the use of a temporary casing during excavation. The piles were reinforced axially with twelve 32 mm ribbed steel bars in the upper half of the pile and four bars in the bottom half. To create a pile head suitable for the load test, the upper 1.2 m was encased in a permanent steel casing of 1.2 m long. The outside diameter of this casing was 1.26 m and the inside diameter was 1.23 m. To provide the reaction force necessary for the load tests, eight continuous-flight-auger (CFA) piles were installed at the four corners of each test pile. These piles have a diameter of 1.05 m and an embedded length of 23.5 m.

In terms of pile instrumentation, ‘sister bar’ strain gauges (model VWS-4000, MGS Geosense, UK) and retrievable extensometers (model A-9, Geokon, USA) were installed near the boundaries between the different soil layers to measure the load distribution

and compression of the piles, respectively. The strain gauges were installed at 0.75, 6.2, 14.9, 19.4, and 26.5 m from the pile top. The retrievable extensometers were also installed at these levels plus at the pile base at 27.5 m. Figure 4.11 schematically shows the instrumentation details near the pile head, and Figure 4.12 shows the ‘sister bar’ strain gauge and the steel pipe (50 mm I.D., 60 mm O.D.) used for cross-hole sonic logging and for hosting the retrievable extensometers.

4.5.2 Support Fluid: Basic Information

Water

The mix water used for preparing the support fluid was potable water delivered by a tanker. The pH was about 7.5, the electrical conductivity was $670\ \mu\text{S}/\text{cm}$, and the total hardness was about 250 mg/L (ppm). These properties are typical of fresh water in London and across South East England.

Polymer

The polymer used in this trial is the SlurryPro CDP system supplied by KB International in the USA. As shown in Table 2.1, this polymer system consists of a base polymer (SlurryPro CDP) and several additives for enhanced performance and dealing with difficult ground conditions. Since the soil at the test site was relatively stable (mainly stiff clay and dense sand), only a few additives were needed for the trial. A list of the chemicals used, their dosages, and their key functions are given as follows:

ProTek 100: Concentrated potassium hydroxide solution. Prior to adding the base polymer, ProTek was added to the mix water to raise the pH to about 11.0. The key function of this additive was to treat the water hardness and to create a suitable alkaline environment for the base polymer to generate viscosity (see Section 2.2.3 for a discussion of the effect of pH on the degree of dissociation of polymers).

SlurryPro CDP: Base polymer supplied as dry granules. This product is a modified HPAM although it is also known to be a ‘Vinyl’ polymer (see footnote no. 3 in Section 2.3.3). It was mixed by aeration at a concentration of $1.0\ \text{kg}/\text{m}^3$ for the trial. The key function of this product was to generate fluid viscosity and be the carrier of other additives. Figure 4.13 shows a 25-kg bag of SlurryPro CDP and a container of ProTek 100 on site.

SlurryPro LA-1-D: Additive supplied as fine powder. The chemistry of this product is not published. It was mixed at a concentration of 0.08 kg/m^3 for the trial. According to the supplier, this additive helps the formation of a fluid membrane on the sidewall.

SlurryPro MPA: Additive supplied in liquid form. The chemistry of this product is not published but the molecules are known to carry a positive charge and to have a hydrophobic tail. When added to a HPAM fluid, this chemical would be expected to attach itself to the carboxyl group of the HPAM and also create a tighter molecular structure due to the mutual attraction (association) of the hydrophobes. According to the supplier, the key function of this additive is to accelerate the settlement of suspended soil particles before concreting. In terms of its application, as soon as the excavation was complete, 10 L of dilute solution (1% in water) were dropped to the bottom of the pile bore in a plastic bag which was weighted with a brick or stone. This process was colloquially known as dropping an MPA ‘bomb’ on site and is shown in Figure 4.14. If added during the excavation (note this was not done during the trial), this additive also will reduce the fluid loss in coarse soil since it reacts with the base polymer (CDP) to form a soft gel mass which can plug the pores in the sidewalls. Figure 4.15 shows the cleaning and gel-forming effects of this additive in a bucket trial, but note that the effect shown in the photographs was greatly exaggerated since undiluted solution was used.

In terms of mixing, the base polymer SlurryPro CDP and the additive SlurryPro LA-1-D were sprinkled into the ProTek 100-treated water in a mixing tank such as the one previously shown in Figure 1.9. A photograph of the polymer fluid can be seen in Figure 1.7. Note that the milky colour of the fluid was entirely due to the additive SlurryPro LA-1-D as a solution of SlurryPro CDP is transparent. This therefore suggests that SlurryPro LA-1-D did not dissolve in the water but existed as fine colloidal particles in the fluid.

Bentonite

The bentonite used for the trial was Berkbent 163 supplied by Tolsa (formerly Steetley Bentonite and Absorbents Ltd). This product is a sodium carbonate (soda ash) activated bentonite and is commonly used in the UK for the construction of bored piles, cut-off walls, and diaphragm walls. According to the supplier, the moisture content of the bentonite

powder ranges from 10% to 14%, and the cation exchange capacity (CEC) is about 85 meq/100 g. The methylene blue index (MBI), determined by the author according to ASTM (2003), is about 89 meq/100 g. Methylene blue ($C_{16}H_{18}N_3SCl$) is a potent cationic dye which can be adsorbed by the negative charges of clay particles. The value of the MBI is therefore a direct indication of the negative charge on the clay. There is also often a strong correlation between MBI and other clay properties such as CEC, specific surface area, swell potential, etc. (e.g. Yukselen & Kaya, 2008).

Depending on the ground conditions, the dosage recommended by the supplier ranges from 30 to 80 kg/m³, i.e., 3% to 8% w/w. The dosage used for the trial was 40 kg/m³ since the soil at the test site was relatively stable. Although treatment of the mix water with soda ash was recommended by the supplier in hard water areas such as London, a preliminary trial showed that the residual soda ash in the product was capable of removing 90% of the water hardness, i.e., from about 250 down to 25 mg/L, with the remaining 10% believed to be caused by magnesium ions. As a result, no soda ash treatment was deemed necessary as it would have made little difference. After mixing, the fluid was left in the storage tank to hydrate and at the time of use the fluid was about 3 days old. Figures 4.16 and 4.17 show the equipment used for mixing the bentonite fluids and for storage. No bentonite cleaning equipment (desander and desilter) was required since only one bentonite pile was being constructed. A photograph of the bentonite fluid is shown at Figure 1.7.

4.5.3 Excavation and Base Cleaning Procedures

Excavation of the test piles generally followed BBGE's standard procedures for that area. The initial steps included inserting a steel casing through the made ground down to the top of the underlying clay as shown schematically in Figure 4.9. A photograph of the casing being inserted into ground can be found in Figure 4.18. Since the made ground was underlain by stiff overconsolidated clay (Lambeth Group), after the installation of the casing, the made ground and the majority of the Lambeth Group were excavated dry without any support fluids. To continue to maintain the stability of the pile bores, support fluids were introduced into the bores by a centrifugal pump when the depth was about 17 m b.g.l. (−11.5 mOD). As discussed in Section 3.4.5, centrifugal pumps can cause

damage to the polymer fluid due to shear degradation. It was used because at the time of the trial its adverse effect on the polymers was not yet proven.

Once the support fluids were in the pile bores, excavation became more technically challenging since it required the right tools and the experience to prevent suction occurring in the fluid column and in the surrounding soil during raising of the excavation tool – an issue not always appreciated by the foundation contractors. To prevent suction from developing, both a twin-flight auger (Figure 1.3) and a vented digging bucket (Figures 4.19 and 4.20) were used to allow the fluid to flow back into the bottom of the hole during tool extraction.

For the polymer piles, after the bores were excavated down to about 0.5 m above the final base level, the solids-laden fluid was cleaned by dropping an MPA ‘bomb’ to the base of the bore. The ‘bomb’ was then broken by the digging bucket and the contents dispersed by gently raising and lowering the tool several times. A waiting period of about 30 min was then required to allow the additive to react with the base polymer and the soil particles to fall out of suspension. The effect of this procedure was checked by comparing the sand content measured before and after ‘the bomb’. The results will be discussed in the next section.

For the bentonite pile, at the end of the excavation the used bentonite fluid was exchanged for fresh fluid since the sand content was found to be higher than the allowable limit – further details will be given in the following section. Figure 4.22 shows the fluid exchange operation by air lifting.

In all cases, a flat-sided cleaning bucket was used to excavate the final 0.5 m of soil and to prepare the soil surface for a base hardness/cleanliness test. This test made use of a 100 mm×100 mm×20 mm thick square metal plate on a tape measure. The impact as felt by the operator when the plate hit the pile base was graded on a scale of between 1 (hard) to 5 (very soft). Details of this test can be found in Berry (2009) and are not repeated here. All three test piles received grade 3, meaning detection of a distinct base with slight embedding of the weight into the soil. This result confirmed a similar base quality for the three test piles.

4.5.4 Support Fluid: Field Test Results

Immediately after the excavation was completed, fluid samples were taken from the middle and bottom of the pile bores for testing. The properties tested included pH, sand content, density, Marsh funnel viscosity, and the flow (viscosity) curve. Since at the time of the trial the Brookfield viscometer was not available to the author, only the Fann viscometer was used to measure the flow curve from 5 to 1022 s^{-1} . Table 4.2 summarises the test results for the first four properties, and Figure 4.24 shows the flow curves. These results are discussed individually as follows.

pH: The pH of the fluid was measured using a hand-held pH meter. The clean polymer fluid had a pH of about 11 since the pH of the mix water was raised to this level. It can be seen that the used fluids also had the same pH, suggesting little chemical reaction took place while the fluid was in the pile bores. With regard to the bentonite fluid, although the mix water was not pre-treated, the pH of the fresh fluid was still 10.5 which was believed to be caused by the residual soda ash in the bentonite powder. Again, no change in the pH was found in the used bentonite fluid.

Sand content: The sand content, defined as the fraction by bulk volume of particles larger than $75\text{ }\mu\text{m}$, was measured using the standard sand content kit from the oil industry (American Petroleum Institute, 2003). It can be seen that, immediately after excavation, the sand content of the polymer fluids was between 1.3% and 1.4% near the bottom of the pile bores. However, after dropping the MPA ‘bomb’ and a waiting period of about 30 min, the sand content of the polymer fluids in piles dropped down to about 0.1%. This indicated an effective removal of the suspended coarse particles by this method. In contrast, the sand content of the bentonite fluid was 6.8% in the middle of the bore and 10% near the bottom of the bore. Since these values exceed the upper limit of 4% as specified by Federation of Piling Specialists (2006), the used bentonite fluid was exchanged to prevent the formation of a ‘soft toe’.

Density: The density of the fluid was measured using a hand-held density meter shown in Figure 4.23 (model DMA 35n, Anton Paar). As can be seen from the table, the density of the used polymer fluid was between 1.02 and 1.03 g/m^3 . This was caused by the presence of suspended soil particles. There appeared to be significantly more soil particles

in the bentonite fluid since the density was found to be 1.10 g/m^3 in the middle of the bore and at 1.15 g/m^3 near the base of the bore. As already mentioned before, the used bentonite fluid was exchanged since it did not satisfy the current industry specifications.

Marsh funnel viscosity: The Marsh funnel viscosity of the fresh polymer fluid was found to be about 70 s in the mixing tank, and 56 s and 50 s in the bores of piles P1 and P2 respectively. As previously shown in Section 3.4.5, this level of drop is not uncommon when the aeration mixing method and a centrifugal pump are used. In contrast to the polymer fluids, the Marsh funnel viscosity of the bentonite fluid rose from 34 s in the storage tank to about 40 s in the pile bore. This was again caused by the presence of a large amount of suspended particles, especially by clay sized material, in the fluid before exchange.

Flow curve: Figure 4.24 shows the Fann viscometer test results. To show the overall trend of the curves, the power-law and Herschel-Bulkley models were used to fit the data points of the polymer and bentonite fluids respectively. For clarity, this was done only to the results for the clean fluids. In summary, the results confirm that the effect of use was to increase the viscosity of the bentonite fluid and to decrease the viscosity of polymer fluid. However, it should be noted that if the polymer fluid was repeatedly reused, then the viscosity would have risen instead as discussed in Section 3.4.7. The test results also explain why the particle-holding ability of the bentonite fluid was so much greater – at the shear rate of 1 s^{-1} the projected viscosity of the bentonite fluid was about 10 times higher than that of the polymer counterpart. This difference is expected to be even more significant at lower shear rates and when thixotropy comes into play.

4.5.5 Concreting Details

Self-compacting concrete which had a designated class C28/35 20 mm CIIB-V S4 DC-3 was used to form the piles. This designation essentially means the following: 1) characteristic compressive cylinder and cube strengths at 28 days equal to 28 MPa and 35 MPa respectively, 2) the maximum aggregate size is 20 mm, 3) the cement type is Portland cement with 21% to 35% fly ash, 4) the target slump is between 160 and 210 mm, and 5) design chemical class 3 according to BRE Construction Division (2005).

Following standard bored piling practice, the concrete was delivered through a tremie

pipe into the bores to minimise the degree of intermixing between the fresh concrete and the support fluid. Initially, when the tremie pipe was rested on the pile base, a plug having the same internal diameter as the pipe was installed and then the pipe was loaded with concrete. When the pipe was almost full, it was lifted slightly and this allowed the concrete to flow out very quickly due to the large hydrostatic head inside the pipe. This was done to ensure any soft sediments accumulated at the base could be flushed aside to ensure good contact between the concrete and the soil. The discharge rate of the first truck load of concrete was therefore about $1.2 \text{ m}^3/\text{min}$ and a lower rate of around $0.5 \text{ m}^3/\text{min}$ was used thereafter to fill the bore. Figure 4.25 shows the concreting operation.

To assess the effects of the polymer fluids on concrete quality (structural integrity will be dealt with later), during casting of the bentonite pile B1 and the polymer pile P2, samples were taken from the top of the concrete column (Figure 4.26) and from the chutes of the delivery trucks for compressive strength and stiffness tests. Figure 4.27 shows the test results. It can be seen that the strength at the top of the concrete column, i.e., the concrete–support fluid interface, was clearly affected when compared with the strength of samples taken from the delivery truck. The overall strength reduction was 26% and 29% for bentonite and polymer fluids respectively, and a similar degree of reduction can also be seen in the elastic modulus results. The reduction seen was likely to be caused by the intermixing of fresh concrete and support fluid at their interface during casting, but there does not seem to be any correlation with the support fluid type. This finding echoes the conclusion of Jefferis & Mavroulidou (2003) that polymer fluids have little effect on fresh concrete mixes, except to increase their workability and therefore reduce their strength in proportion to the volume of extra fluid. Despite this finding, it should be noted that the amount of reduction shown in Figure 4.27 was not representative of the concrete conditions of the whole pile, as the majority of the fresh concrete would not have any contact with the support fluid. Further discussions on the difference between the theoretical and actual concrete conditions will be given in Section 4.6.5.

Although no measurement of the sidewall profile was carried out at the end of the excavation, during casting the successive rises in concrete level were measured after every truck load of concrete was discharged. This information has been converted to the

equivalent radius along the pile length and is given in Figure 4.28. It is interesting to see that the profiles of the three piles were rather similar, suggesting comparable conditions of the pile bores before concreting. This finding also appears to be consistent with the results of Ata & O'Neill (1998) that the sidewall profiles of their polymer-supported bores, one in sand and one in clay, remained almost the same for up to 19 hours. Finally, it should be noted that the inferred radii given in Figure 4.28 should be considered with caution since there are many factors that can affect their accuracy, including variations in concrete volumes between deliveries, material wastage, etc.

4.5.6 Pile Integrity

The structural integrity of the pile was assessed by two non-destructive methods, i. e., cross-hole sonic logging (CSL) and pile integrity test (PIT). Since most foundation engineers are familiar with the operating principles of these two tests, they are not repeated here. In summary, the CSL results showed no anomalies in the bentonite pile but indicated a flaw in each of the polymer piles. Figure 4.29 shows the profiles of the first arrival time and attenuation of the signal energy. It can be seen that the signal energy reduced (attenuation increased) by 10 to 15 dB at around 18.0 m in pile P1 and at 22.5 m in pile P2. This suggests a flaw or defect in the pile concrete. Fortunately, since the first arrival time increased only very little at these locations, the defects were likely to be minor.

The reason why the two polymer piles had local defects is not known, but they were possibly caused by the damaged access tubes, explained as follows. When CSL was being carried out in pile P1, it was noted that water was draining out of one of the tubes. To establish what level the water would settle at, this tube was filled with water again and the level checked on the next day. It was discovered that the water found its natural level at about 23 m, i. e., 5 m below the location of the flaw. This means that there was one or possibly several leaks in the tube. Since the tube was filled with water prior to concreting, the water may have eroded the concrete locally around the leaking points.

Another possible cause of the flaws was the formation of some (water) channels inside the fresh concrete (Kog, 2009). Since the workability of the self-consolidating concrete was high, some bleeding was noticed at the top of the concrete column during the casting of the three test piles. If this also occurred within the body of the fresh concrete, it

would have locally reduced the concrete quality. No other possible reasons for the flaws can be identified since the concreting operation of the three piles went smoothly and the ground conditions were normal without any artesian water pressure in the soil or collapsing sidewalls.

Figure 4.30 shows the PIT results in the form of velocity–depth profiles. As no sudden change in the velocity can be seen in any of the plots, the cross sections of the piles were likely to be uniform with depth, i.e., no necking or bulging. This finding corresponds with the back-calculated sidewall profiles given in Section 4.5.5. Finally, it should be noted that a clear reflection of the wave at the pile base level (27.5 m from top of pile) was not detected, and it was likely to be caused by the high soil resistance along the shaft reducing the energy of the stress wave.

4.5.7 Load Test Method

Following the pile integrity tests, compressive axial load tests were carried out when the concrete was about 28 days old. Figure 4.31 shows the pile head being prepared for the load test together with some of the instrumentation equipment. Figure 4.32 also shows the reaction frame being constructed.

The test piles were loaded following the ‘slow’ maintained load (ML) method specified by the SPERW (Institution of Civil Engineers, 2007). As the name of the method suggests, following each application of an increment of load, the load was maintained until the rate of pile head settlement satisfied one of three criteria as set out in the specification. For each load increment, the duration also could not be less than a specified minimum period. In addition to the requirement for load-holding periods, a single load cycle was also included in the test to assess the pile behaviour during unloading and reloading.

With regard to the magnitude of the loads, although the specification only requires the maximum load to be 100% design verification load (DVL) plus 50% specified working load (SWL), it was decided to raise the maximum load to 100% DVL + 100% SWL as it is commonly felt that the original maximum load would not cause sufficient settlement for a detailed back-analysis (e.g. England & Fleming, 1994). Due to this change, the test piles were unloaded at 100% DVL + 50% SWL instead of 100% DVL as originally specified. Based on the preliminary calculations, the values of DVL and SWL of the test piles were

9.1 and 9.0 MN respectively. The maximum (proof) load was therefore 18.1 MN.

4.6 Load Test Results and Interpretation

4.6.1 Load–Settlement Response

Figure 4.33 shows the applied load and pile head settlement plotted against time for the three test piles. It can be seen that the three tests all lasted for about 34 h, and that the actual load-holding periods were not very different between the tests. These figures also show that, during the load-holding periods, the loads were held constant without any significant deviations. An examination of the actual data shows that the variation is typically no more than ± 5 kN. The ability to hold the load constant is a key requirement for any sensible extrapolation of the measured settlements to infinite time.

Figure 4.34 shows the load–settlement curves of the three piles. As the load–settlement response is a function of time, discrete data points with the time-dependent settlement either removed or projected to infinite time are also given on the plot. The points with zero time-dependent settlements should be close to what would have been obtained should the piles have been tested using the ‘quick’ ML method (e.g. procedure A in ASTM, 2007)¹⁵. On the other hand, the data points with the time-dependent settlement projected to infinite time should correspond to the long-term conditions as noted in England & Fleming (1994). For routine geotechnical design, it is the long-term condition that is the most useful since it is the closest to the in-service condition of the piles. As will be shown later, the data corresponding to the short-term condition are also very useful, since they allow the shear stiffness of the soil to be considered without the influence of creep and consolidation. It is also worth noting that on the plots two sets of data points are given for 13.6 MN (100% DVL + 50% SWL) – the first set was calculated for the virgin loading condition (smaller settlements) and the second set for the reloading condition (larger settlements). According to England & Fleming, the result of the initial virgin loading is the more useful for load test interpretation, and hence it was used for the following analyses.

In terms of the pile performance, it can be seen from the figure that the two polymer

¹⁵ASTM (2007) specifies that in a quick ‘ML’ test the load is held constant for a period of not less than 4 min but not more than 15 min. The same period is used for all loading increments throughout the test.

piles, despite their different soil-support fluid exposure periods, behaved very similarly. The bentonite pile, on the other hand, showed significantly larger settlements at the higher loads than its polymer counterparts. At the moderate load level of 9.1 MN (100% DVL), the extrapolated settlement values of piles P1, P2, and B1 were 8.6, 6.3, and 8.2 mm respectively. However, at the maximum load of 18.1 MN (100% DVL + 100% SWL), these values then became 29, 24, and 51 mm – the settlement of the bentonite pile was almost twice as much as that of the polymer piles. These results confirm the findings of the literature review (Section 4.2) that the load–settlement response of polymer piles is usually better than their bentonite counterparts, and that extended pile bore open time does not have an adverse effect on the piles.

The reason for the superior behaviour of the polymer piles is probably due to their higher shaft resistances. Based on the findings of literature review (Section 4.2), it is possible that the polymer fluids gave the piles a cleaner concrete-soil interface at the shaft because they do not form filter cakes (clean fluids) or form thinner cakes (solids-laden fluids). The vigorous base cleaning procedure adopted for the construction may also contribute to a clean pile base. To gain further insight into the behaviour of the piles, the load test results will be subjected to further analyses which will be discussed below.

4.6.2 Analyses Using Chin’s Method

As none of the three test piles was taken to failure during the load tests, their load–settlement curves were analysed using the method proposed by Chin (1970) to obtain an estimate of the ultimate pile resistance. This method is also known as the Chin-Kondner method since it was R. L. Kondner who first proposed the underlying hyperbolic stress–strain (or load–settlement) principle (Kondner, 1963). The author prefers this method since the interpreted ultimate resistance is not based on some arbitrary definitions of pile failure, e. g. settlement equals 10% of pile diameter (ISSMFE Subcommittee on Field and Laboratory Testing, 1985), but corresponds to a state when the pile is displaced indefinitely with negligible increase in resistance. This method is also very popular among practising engineers in the UK and is suggested by the Institution of Civil Engineers (2007) as a possible interpretation method.

To use this method, one only needs to divide the settlement (Δ) at each loading stage

by the applied head load (P_h) at that stage and plot it against Δ . For an end-bearing pile, the data points will plot along a straight line with the slope of the line being the ultimate base resistance. For a combined friction and end-bearing pile, two straight lines will result. Chin & Vail (1973) suggest using the slope of the first line to obtain an estimate of the ultimate shaft resistance, and the slope of the second line to obtain the ultimate total resistance. Fleming (1992) noted that this cannot be strictly true because of the nature of hyperbolic functions. Nonetheless, it is still a good starting point for any load test interpretation of piles not taken to failure.

Figure 4.35 shows the analysis results of the three test piles. To remove the influence of time from the load test results, the analyses were carried out using data points corresponding to both short- and long-term conditions as shown in Figure 4.34. As expected, the interpreted resistances are higher for the short-term condition than for the long-term condition. Since the trend of the data points can be said to follow two different straight lines, the ultimate shaft and total resistances can be estimated from the slopes of the lines and the numerical values are given on the plot.

In terms of the difference between the test piles, it can be seen that the ultimate shaft and total resistances of the bentonite pile are significantly lower than those of the polymer piles. This is as expected since the bentonite pile showed a much larger settlement. It should be noted that, since the Chin analysis is essentially a curve-fitting exercise to a hyperbolic curve, a pile with larger settlement will always be interpreted as having lower ultimate resistance, and vice versa.

4.6.3 Analyses Using Fleming's Method

CEMSET is another method commonly used in the UK to interpret pile load test data. This method was proposed by Fleming (1992) as a predictive and an analytical tool for maintained load tests using parameters most geotechnical engineers would recognise and understand. Essentially, this method is based on the same hyperbolic function as used by Chin, but the overall load-settlement response is divided into three main parts: shaft resistance and settlement, base resistance and settlement, and elastic shortening. Since the underlying principles and the equations can be found in the original paper, they are not repeated here. Only the key parameters are described below to aid the discussion of

the results.

Geometry: There are five parameters which define the basic geometry and the load transfer length of the piles. They are pile shaft diameter (D_s), pile base diameter (D_b), upper length of a pile carrying no load or low loads by shaft resistance (L_0), length of a pile transferring load to the soil by shaft resistance (L_f), and the effective column length of the pile transferring shaft resistance, divided by L_f (K_e). For the test piles, D_s and D_b equal 1.2 m, and L_0 can be taken as the thickness of the made ground which is 6 m. It follows that L_f is 21 m. K_e was taken as 0.45 as this is the approximate value suggested by Fleming for piles in stiff overconsolidated clays.

Resistance: There are two resistance parameters in the formulation, i. e., ultimate shaft resistance (U_s) and ultimate base resistance (U_b). The values of these two parameters were directly taken from the Chin analysis since the the definition of ‘ultimate resistance’ is exactly the same for both methods. The values corresponding to the long-term condition were used to remove the influence of time as suggested by Fleming.

Stiffness: The first stiffness parameter is the elastic modulus of pile material in compression (E_p)¹⁶. During the back-analysis, this parameter was adjusted until the computed values of elastic shortening at various loading stages matched the measured values as recorded by the extensometers. The second stiffness parameter is the secant deformation modulus for soil beneath the pile base at 25% of ultimate stress (E_b). This parameter was adjusted until the computed base settlement matched the actual response. The final stiffness parameter is a dimensionless flexibility factor which represents the movement of a pile relative to the soil when transferring load by friction (M_s). Fleming noted that this parameter is the tangent slope at the origin of the hyperbolic curve representing shaft resistance. This parameter was also noted to be the equivalent of $\zeta\tau_0/2G$ in the notation of Randolph & Wroth (1978), where ζ is $\ln(r_m/r_0)$, r_m is the maximum radius of the zone of influence, r_0 is the pile radius, τ_0 is the shear stress at the pile surface, and G is the shear modulus. From this relationship, it follows that M_s is also the equivalent of $w_s/2r_0$, where w_s is the deflection of the pile shaft (Randolph & Wroth, 1978). According to Fleming, the value of M_s typically ranges from ≈ 0.0005

¹⁶In the original paper, Fleming denotes the elastic modulus of the pile material as E_c . In this thesis, E_c is used to represent the elastic modulus of pile concrete only, i. e., not including the contribution of the reinforcing steel.

for very stiff soils or soft rocks, to 0.004 for soft to firm or relatively loose soils. For stiff overconsolidated clays, this parameter typically ranges between 0.001 and 0.002, but the actual value is also known to be affected by construction practice and several other factors. During back-analyses, this parameter is the last to adjust since it is the least known.

Figure 4.36 shows the back-analysis results of the three test piles. The actual values of the parameters used are also given on the plots. Since the geometry and resistance parameters are either known or have already been determined, the following discussion is focused on the stiffness parameters, especially E_b and M_s . As can be seen, the estimated value of E_b for polymer pile P2 was the lowest at 65000 kN/m², although the U_s of this pile was previously estimated to be the highest. The values of E_b for the other two piles (P1 and B1) were both estimated to be 85000 kN/m². The difference between these back-analysed E_b values may be caused by the different pile bore open times between P2 (7.5 h) and both P1 and B1 (26 h). However, as shown in the figure, since the actual mobilised base resistances of the two polymer piles were less than 10% of their predicted ultimate values, the estimated values of E_b and U_b should be treated with caution as the uncertainty is large.

The estimated values of M_s , on the other hand, can perhaps be accepted with more confidence, as a significant proportion of shaft resistance was mobilised in all test piles. It is interesting to find that the back-calculated values of M_s are 0.0006 for the two polymer piles, and only 0.003 for the bentonite pile. This means that, in terms of the shaft response, the polymer piles were comparable to piles in very stiff soils or soft rocks, and the bentonite pile was similar to a pile in a firm soil. Since the value of ζ is about 4 for typical pile geometries (Baguelin & Frank, 1980; Randolph, 2003), using the preceding relationship M_s can also be expressed in terms of the average local shear strain along the pile shaft $\gamma_0 (= \tau_0/G)$, which is 0.03% for the two polymer piles and 0.15% for the bentonite pile. As M_s is the initial tangent slope of the shaft resistance curve, these strain levels can be seen as an indication of the effect of pile installation. For soil which experiences little disturbance, the strain level will be low, otherwise the strain level will be high. Finally, it should be borne in mind that, as the back-analyses were carried out using the long-term projected settlements, the stiffness parameters presented in this section

are only the effective values including the effects of consolidation and creep. The ‘true’ stiffness values without the influences of these effects should therefore be higher.

The CEMSET analysis marks an end to the first phase of load test interpretation using data mainly collected externally, i.e., the applied head load (P_h) and the measured pile head settlement (Δ). In the following sections, the instrumentation data will be examined in detail before being used for the construction of the load transfer curves. This two-step approach is also recommended by Institution of Civil Engineers (2007). Since the piles were tested by the ‘slow’ ML method, special emphasis was also given to the effect of concrete creep and of the pile modulus determination methods on the accuracy of the interpreted loads. Although these two effects are seldom discussed in the literature, careful consideration of them was found to be extremely important for interpretation of the instrumentation results.

4.6.4 Stress–Strain Curves of Pile Materials and the Treatment of Concrete Creep Strains

Figure 4.37 shows the stress–strain curves at the different instrumentation levels for the three test piles as recorded by the ‘sister bar’ strain gauges. The strain gauge results are expressed in microstrain ($\mu\epsilon$). To remove the effects of installation and load eccentricity, the strain readings are referenced to their values recorded at the start of the load test, and the values given in the plots are the average gauge readings at each level. Note that the stress values are only correct for the uppermost level where the shaft resistance over the top 0.75 m of the pile can be ignored. At the other levels the stress (load) values in the pile would be smaller but they are not yet known at this stage. Nonetheless, it can be seen from the plots that the measured strain values increased significantly during the load-holding periods and appeared to be both time and stress dependent (the load-holding periods are shown in Figure 4.33). As steel does not creep, it must be the pile concrete which caused the additional strains. Since Hooke’s law is the only equation that will be used to convert the measured strains into pile loads, the creep strains have to be removed otherwise considerable errors could result. To the author’s knowledge, the need for creep strain correction has only been emphasised for long-term load measurements over a period of perhaps several weeks or months (Dunnicliff, 1993; Fleming *et al.*, 2009; Ooi *et al.*,

2010).

To examine the effect of concrete creep strains (ε_{cc}) in greater detail, the stress-strain (σ - ε) curves of the materials of pile P2 was chosen for illustration. Figure 4.38 shows the σ - ε response of the pile material at the uppermost instrumentation level (i. e., 0.75 m below pile head) as recorded by the ‘sister bar’ strain gauges. Two curves are given in the figure: the dotted line shows the actual measured response and the solid line shows the response corrected for the effect of concrete creep. For the purpose of correction, creep strains are defined here as the strains that develop during the load-holding periods. As the application of each load increment took about 5 min (calculated from the actual data points plotted in Figure 4.33), the potential error caused by this definition, using the creep prediction method given in Eurocode 2 (European Committee for Standardization, 2004), is about 3.5% of the total creep value.

In terms of the results, Figure 4.38 shows that a significant amount of concrete creep, ε_{cc} , developed during the load-holding periods, resulting in a total of about $200\ \mu\varepsilon$ at the end of all the loading stages. If these creep strains are removed, the σ - ε curve then plots as a hysteresis loop with negligible residual plastic strains. This means that the true σ - ε response was nonlinearly elastic and the corrected strains can be described as elastic strains. However, despite the correction, a tiny amount of plastic strain ($6\ \mu\varepsilon$), caused by the unload-reload loop, is still present and may just be noticed on the corrected curve at 11.4 MPa. Since the plastic strains accounted for less than 1% of the total elastic strains ($740\ \mu\varepsilon$), they are negligible in this case.

As some authors have suggested the use of retrievable extensometers for the derivation of pile modulus (Hanifah & Lee, 2006; Ali *et al.*, 2008; Ali & Lee, 2010), it is also worth examining the extensometer data which is shown in Figure 4.39. It can be seen that, unlike the measured response of the strain gauges in Figure 4.38, the extensometer did not detect any creep during the load-holding stages, but instead displayed a noticeable amount of instrument drift. At proof load ($P_h = 18.1\ \text{MN}$; $\sigma = 15.2\ \text{MPa}$), the amount of drift was about 0.02 mm, equivalent to $27\ \mu\varepsilon$ over the effective gauge length of 0.75 m. Another problem with the extensometer was that the readings became negative on unloading, a problem that became more pronounced as the maximum stress level increased. The

reasons for this behaviour is not known, but it may have been caused by the debonding and slippage at one of the interfaces between the extensometer anchors, steel tube, and the pile concrete. Debonding at the steel–concrete interface is possible because of the smooth painted surface of the steel tube (Figure 4.12) and the large difference in their modulus values leading to high shear stress to be carried at the wall. For this reason, the extensometer records will not be considered for the estimation of pile modulus.

As the treatment (removal) of creep strains directly affects the accuracy of the load test interpretation, it is also useful to compare the so-defined creep values with those determined by the standard methods to ensure they are indeed reasonable. Such a comparison would also highlight the effects of nonuniform stress (and strain) distribution and of the various steel components on the creep development behaviour, discussed as follows. Figure 4.40 shows the creep strains of the four individual strain gauges and their average values at the uppermost instrumentation level under proof load. It can be seen that, although they all start with a different strain level (given to the right of the curves), this value does not correlate strongly with the magnitude of the creep strain. For example, gauge 1 had the second lowest starting strain but recorded the second highest creep strain. This suggests that the measured creep strains were not particularly sensitive to the nonuniform strain distribution across the cross section of the pile. The curves given in this figure also justify the decision to use four gauges instead of two at each instrumentation level, as the result would be 10% higher or lower if only two were used.

Figure 4.41 shows the predictions of creep strain using the calculation method given in Eurocode 2 (European Committee for Standardization, 2004). As Eurocode only covers the creep strain of plain concrete, the age-adjusted effective modulus method illustrated in Wight & MacGregor (2009) was also used to take account of the restraining effects of the various steel components in the pile. In this exercise, the pile concrete was initially assumed to be uncontaminated, and hence the elastic modulus of the pile concrete (E_c) was taken as 29.3 GPa as shown in Figure 4.27. As Eurocode uses compressive cylinder strength (f_{cyl}) instead of compressive cube strength (f_{cube}), the value of f_{cyl} was taken as the typical 75% of f_{cube} . The predictions show that the various steel components had a strong effect not just on elastic but also on the creep behaviour of the pile. For example,

including the reinforcing bars alone would reduce the prediction at 5 h from 77 to 61 $\mu\epsilon$. If the effects of the steel logging tube and the external casing are also considered, then the predicted value will drop down to 33 $\mu\epsilon$.

If the above predictions are correct, they will offer an opportunity to assess the contribution of the various steel components to the deformation behaviour of the pile. However, because all the current predictions underestimate the measured values, it is likely that the actual E_c of the pile was lower than those of the cubes tested in the laboratory. This could be caused by the intermixing between the fresh concrete and the support fluid as discussed in Section 4.5.5, or by the different stress levels and sizes between the pile concrete and the cube samples. Although it is not known which of these is the main cause, this finding highlights the difference between the expected and actual properties of the pile concrete.

To quantify the possible reduction in the values of the parameters (E_c and f_{cyl}), the scenario where only concrete and reinforcing bars contribute (the dotted line in the plot) was matched to the measured response by manually adjusting them. It was found that a reduction of 20% was necessary for the two curves to match. An attempt was also made to match the case where all the steel components contribute (the solid line in the plot), but it proved to be impossible. The highest creep value achievable for this case was 63 $\mu\epsilon$ (17 $\mu\epsilon$ short of the measured value) even when the parameter values were reduced by 80%. Any further reduction would lower the amount of the predicted creep. This is because, as the contribution of the concrete reduces, the steel would start to dominate the overall deformation behaviour and, because steel does not creep, the predicted creep strain would reduce. Based on the finding of this ‘matching’ exercise, it would appear that the pile concrete was weaker than expected and the external steel casing (Figure 4.11) did not contribute to the deformation behaviour of the pile. This last finding is actually very important for the calculation of the pile modulus and will be discussed further in the next section.

To apply the preceding findings to other test piles, the creep correction method shown in Figure 4.38 was also used to adjust all the other strain gauge data. Figures 4.42 and 4.43 show the as recorded and the corrected strain profiles, respectively, for the three

test piles. It can be seen that, after the correction of concrete creep, the strain values have become significantly smaller especially at large loads. Some of the strain profiles also appear more reasonable than those before the correction. Although it may not be obvious from these curves, the amount of creep strain removed gradually reduces with depth, which is as expected due to the reducing stress level in the pile. The corrected strain profiles will be used for strain-to-load conversion which will be discussed in the next section.

Finally, it is important to note that the above data reduction approach assumes that no redistribution of load between the pile shaft and the pile base took place during the load-holding periods. This has shown to be untrue by Whitaker & Cooke (1966) for piles embedded purely in clay and when the shaft resistance is fully mobilised. Burland & Cooke (1974) explain that this is caused by the consolidation settlement mobilising further load at the pile base. If the load also redistributed in the test piles during the load-holding periods, the strain-distribution profiles would have become steeper as more load was transferred to the pile base (note that the strain-distribution profile for an end-bearing pile would be very steep). However, this was not seen at the test piles and, in fact, due to the large amount of concrete creep strain developed near the pile head, the slope of the strain-distribution profiles actually became flatter (see Figures 4.42 and 4.43). The reason why the load did not redistribute is probably because the shaft resistance was not fully mobilised. The amount of mobilised shaft resistance will be discussed in Section 4.6.8.

4.6.5 Estimation of Pile Modulus and Strain to Load Conversion

To derive the elastic modulus of the pile (E_p), both the secant modulus method¹⁷ (e. g. Shi, 1996; Deschamps & Richards, 2005; Geotechnical Engineering Office, 2006) and the tangent modulus method (Fellenius, 1989, 2001) were used, as they have been found to complement each other and together provide a better view of the deformation behaviour of the pile material. Note that the E_p values previously estimated in the CEMSET and in the creep strain analyses (Sections 4.6.3 and 4.6.4) are not suitable for strain-to-load conversion, since these values are the effective long-term values including the effect of concrete creep.

¹⁷This method does not have a generally accepted name. It is named by the author based on its principal characteristics.

A description of the two modulus methods is given as follows:

Secant modulus method: To use this method, one first needs to compute the E_p values from the applied stress and the corrected strain values measured at the uppermost instrumentation level (i.e., from the corrected σ - ε curve given in Figure 4.38). The modulus value determined in this way is commonly referred to as the secant modulus and hence the name of the method. The resulting E_p values are then plotted against ε and their relationship modelled with a best-fit curve or line. The fit establishes an equation for the secant modulus as a function of the measured strain, which is then applied to the other gauge levels in the pile to convert the corrected strains into loads. The assumption of this method is that the E_p - ε relation developed for the gauge level near the pile head can be applied to the remainder of the pile. In other words, the deformation behaviour of the concrete is assumed to be the same throughout the pile. This assumption can usually be validated quantitatively, at least for the top part of the pile, by the tangent modulus method.

Tangent modulus method: Unlike the secant modulus method, this method considers not only the material at the pile head but at all instrumentation levels. For gauge levels down in the soil, this method takes advantage of the fact that, after full mobilisation of the shaft resistance, the σ - ε response of the pile material is effectively that of a free-standing column, provided that the soil can sustain its peak strength (i.e., show elastic-perfectly plastic type behaviour). The σ - ε curve is assumed to follow a second-order polynomial, so that the tangent modulus, E_{pt} , has a linear relationship with strain. The secant modulus, E_p , which is still the modulus required for strain-to-load conversion, is then calculated from the slope and y -intercept of the E_{pt} line. The equations of this method are:

$$\sigma = 0.5a\varepsilon^2 + b\varepsilon \quad (4.1)$$

$$E_{pt} = d\sigma/d\varepsilon = a\varepsilon + b \quad (4.2)$$

$$E_p = 0.5a\varepsilon + b \quad (4.3)$$

where a and b are the slope and y -intercept of the tangent modulus line respectively.

To illustrate the pile modulus determination procedure, pile P2 is again used as an

example. Figures 4.44 and 4.45 show the secant modulus and tangent modulus plots respectively. It can be seen from Figure 4.44 that the back-calculated E_p values (the solid circles) decrease non-linearly with increasing strain level, and that the best-fit curve of the secant modulus method is able to represent the trend reasonably well. On the other hand, the tangent modulus line only gives a linearly decreasing trend; this is due to the assumptions of the method as noted earlier.

Although the tangent modulus method performed worse than the secant modulus method, it actually provided some very important information about the deformation behaviour and the conditions of the pile. First, it can be seen in Figure 4.45 that data points of E_{pt} at the first two instrumentation levels (0.75 and 6.2 m) sensibly followed a common trend line, despite the fact that the top 1.2 m of the pile was encased in a steel casing (Figure 4.11). This means that the casing was not contributing to the axial stiffness but was simply floating on the outside of the pile like a ring on a finger. The creep ‘matching’ exercise discussed in Section 4.6.4 also resulted in this same conclusion. Second, if one accepts the conclusion about the negligible effect of casing, then the E_{pt} values given in Figure 4.45 also indicate that the quality of the concrete was uniform over the top 6.2 m of the pile. This piece of evidence, together with the cross-hole sonic logging records given in Figure 4.29, allow the E_p - ε relationship to be applied to other instrumentation levels with greater confidence.

With the values of the E_p determined, the strain profiles given in Figure 4.43 can now be converted into pile loads using Hooke’s law written in the following form:

$$P = EA\varepsilon \quad (4.4)$$

where P is the load in the pile, E is the calculated modulus, A is the pile cross-sectional area (concrete and steel), and ε is the corrected strain. Figure 4.46 shows the interpreted load profiles for the three test piles. The profiles of the unload and reload loops are omitted from the plots for clarity. To extend the profiles to the entire length of the pile, the load level in the top 0.75 m is assumed to be constant, and the rate of load reduction over the bottom 1.05 m of the pile (26.45 to 27.5 m) is assumed to follow the same trend as above. It can be seen from the plots that the load distribution between the two polymer piles are very similar. This is not surprising since the Chin and CEMSET parameters

for these two piles are also very similar (Sections 4.6.2 and 4.6.3). The figure also shows that the shaft resistance of the bentonite pile was much lower since the mobilised base resistance was much higher. Again, this is expected from the results of the previous analyses. Finally, it is important to note that the shape of the load-distribution curves seems to suggest the presence of some residual or locked-in loads (Fellenius, 2002a), which are most commonly seen in driven and jacked piles but also in bored cast-in-place piles (see e.g. O’Riordan, 1982; Fellenius & Altaee, 1996; Siegel & McGillivray, 2009). For this reason, the potential effect of the residual load is investigated in the following section.

4.6.6 Estimation of Residual Load

Residual loads are loads that are ‘locked-in’ the pile before the start of the test. At this site, they can be caused by three mechanisms, 1) the recovery of the soil at the pile–soil interface after disturbance caused by the drilling process as well as the heat and pressure from the fresh concrete, 2) change in concrete volume during curing, and 3) the ongoing consolidation of the cohesive Lambeth Group due to the weight of the recently placed made ground (Section 4.4.1). In fact, even when the third mechanism does not come into play, some residual loads are still expected, as was found by O’Riordan (1982) who was working in a nearby site.

Since residual loads only developed between concreting and the start of the tests, it is useful to examine the change in the strain gauge readings recorded during this period. During construction, gauge readings were taken at three critical stages: in the yard, in the pile bore immediately after concreting (in practice this means about 3 h after pouring the first truck load), and just before commencement of the load test. Figure 4.47 shows the changes in the strain values referenced to the baseline values taken in the yard for pile P2. It can be seen that the gauge readings dropped immediately after concreting and then increased slightly during curing. This indicates that the gauges first experienced a compressive force, probably as a result of the internal strain distribution due to the hydration heat. As the concrete cooled, this process reversed resulting in a partial recovery of some of the compressive strains and even some tensile strains near the bottom of the pile. Fellenius *et al.* (2009) also reported similar patterns of change in the strain values for other pile types and provided a detailed discussion of this phenomenon. Since it is not

possible to separate the installation effects due to heating and subsequent cooling from those actually caused by the residual load, it had to be quantified by a different method, as follows.

Fellenius (2002b) proposed a method for the determination of residual load directly from the interpreted load-distribution curves. In essence, his method assumes that the residual load is fully mobilized along the upper portion of the pile and the shearing resistance of the soil is independent of the direction of shear. Therefore, the calculated shaft resistance from the measured load curve could be as much as twice the true value for the upper part of the pile. To use the method, one first needs to plot half of the measured load reduction (as calculated from the strain gage results) against depth and then compare this curve (known as the half curve) with the estimated shaft resistance obtained from an effective stress analysis (i.e., using K_s and δ). If the assumptions of the method are correct, these two curves should lie on top of each other over the part where the negative skin friction (NSF) is fully mobilized. The depth where these curves deviate from each other is where the NSF starts to reduce and change to positive direction. Over the ‘matched length’ between these two curves, the residual load is the same as the value of the half curve, and the true load is therefore the measured load plus the residual load. Because the residual load below the neutral plane cannot be directly determined, it needs to be estimated from the estimated shaft resistance with the help of some engineering judgment. Further details about the method can be found in Fellenius (2002b) and are not repeated here.

As an example, Figure 4.48 shows the result of the residual load analysis for pile P2. The soil parameters used are summarised in Table 4.3. Since at this stage it is not yet known how the pile installation has affected the soil properties at the pile–soil interface, for the Lambeth Group the interface angle, δ , was taken as ϕ'_p . The value of the coefficient of earth pressure for the shaft, K_s , was taken as 1.15 as discussed in Section 4.4.4. It can be seen from the figure that the shaft resistance curve indeed lies very close to the half curve until a depth of 19 m was reached, which is near the bottom level of the Upnor Formation. The estimated shaft resistance was also checked by using the total stress approach (i.e., using S_u and α ; the curve is not shown for clarity), and it

gave similar results.

As the residual load below the neutral plane cannot be determined directly by this method, it had to be estimated using prior knowledge about the interface shear strength of the soil concerned, which in this case is the Thanet Sand. Although for pile design the δ of Thanet Sand is commonly taken as 36° which is the ϕ'_p commonly found using the small shear box (Section 4.4.4), this practice is considered too approximate for this study since it does not take into account the effect of support fluid. As a result, to investigate this effect some laboratory tests have been carried out using a medium interface shear box which is 270 mm long and 170 mm wide. The initial development of the box was carried out by Bower (2009) and further work was done by the author. Only the main findings are presented here. Figure 4.49 shows the test results and the fitted failure envelopes for three cases: 1) recompacted sand, 2) concrete–sand interface after the sand was exposed to the SlurryPro CDP polymer fluid under a pressure of 230 kPa for 7.5 h, and 3) as in the second case but using the Berkbent 163 bentonite fluid instead. The filtration pressure, exposure period, and the properties of the two support fluids were chosen to mimic the actual field conditions, so that the results can be applied directly to this case study.

In summary, it was found that the sand has a ϕ'_p value of 36.5° which is essentially the same as those commonly determined using a small shear box, but that δ turns out to be highly dependent on the type of the support fluid – for polymers it has a value of 32.5° but for bentonite it only has a value of 16.5° . In the former case, the reduction from 36.5° to 32.5° is probably due to the soil disturbance caused by the casting process (e.g. penetration of the coarse aggregates into the soil surface) and the different internal stress distribution in the soil. However, in the second case, the reduction to 16.5° is actually caused by the failure plane located in the consolidated filter cake between the concrete and the sand. To help illustrate this, Figure 4.50 shows the filter cake thickness curves over a period of 24 h for two cases: 1) immediately after filtration at 230 kPa, and 2) as in the first case but after consolidation at a pressure of 360 kPa. This latter pressure level is the estimated fluid concrete pressure acting on the Thanet Sand at the test piles using the method proposed by Lings *et al.* (1994). It can be seen from the figure that the cake thickness (h) was about 9 mm immediately after filtration but reduced to about

2.5 mm after consolidation. If this layer is present between the sand and the concrete, it will become the weak plane. Therefore, during the interface shear box tests, it was the strength of this material that controlled the interface shearing resistance if the soil was formerly exposed to bentonite fluids. For comparison, it is interesting to note that Cernak *et al.* (1973) reported a similar δ value of 14° for their consolidated filter cake which was formed at a lower filtration pressure of 25 kPa.

With the laboratory test results, the estimation of the residual load for pile P2 was completed using a δ value of 32.5° . The complete curves of the residual load and of the ‘true’ pile load distribution are shown in Figure 4.48. Based on the results, it can be concluded that the pile did have a considerable amount of residual load locked-in before the start of the test. Ignoring this load would have resulted in significant error in the interpreted loads and hence in the shear strength of the various soil layers. However, it should be noted that the residual load estimated by this method is not exact as noted by Kim *et al.* (2011). However, the error introduced by not including the residual load will be far greater than the potential error of the method itself and also the errors caused by the choice of soil parameters for the estimation of the shaft resistance curve. To complete this stage of the interpretation, the residual loads have also been estimated for piles P1 and B1 using the same method. Figure 3.4 shows the corrected load-distribution curves of the three test piles. These corrected load-distribution curves were used to derive the mobilised shaft and base resistances, and the results are given in the following section.

4.6.7 Measured Shaft and Base Resistances

Figure 4.52 shows the average mobilised shear stress (τ) for each soil layer plotted against the local pile displacement (w) at the mid point of each layer. Several comments can be made about these curves, as follows.

Firstly, it can be seen that none of the τ – w curves starts from the origin except for two curves of the made ground. This is expected since the NSF in the made ground and in the two Lambeth group layers had to be balanced by a positive shaft resistance in the Thanet Sand and by the base resistance.

Secondly, it can be seen that the τ – w curves of the made ground and the Lambeth Group units plot roughly around zero shear stress with the same τ_p value in both positive

and negative directions. This is actually a direct effect of the application of the residual load correction method as discussed earlier. It is worth noting that, although not using the same correction method, O’Riordan (1982) also dealt with the issue of residual load in a similar way by manually moving the axis of zero stress such that τ_p was approximately equal in both directions. The method adopted for this study is considered better since it automatically guarantees force equilibrium in the pile.

Lastly, it can be seen that a significant amount of creep developed during each load-holding period. The creep occurred in both the soil and the pile concrete but by different amounts. An inspection of the actual extensometer data reveals that about 10% to 15% of the creep movement was actually due to the concrete. As one of the objectives of this case study is to back-analyse the actual soil stiffness values, it was decided to remove the creep from these curves before proceeding further. The corrected τ – w curves are shown in Figure 4.53. It should be noted that this step of creep removal from the pile displacement is different from the previous removal of the concrete creep strain for the purpose of strain-to-load conversion (Section 4.6.4).

Figure 4.54 shows the mobilised base stress (q_b) plotted against the base displacement (w_b). A significant amount of creep again can be seen in the measured q_b – w_b curves. As a result, the same correction procedure has been applied and the new curves are denoted as ‘short-term’ on the plots. The base displacements projected to infinite time for each loading stage are also given on the plots and are denoted as ‘long-term’. These long-term base settlements were used previously for the CEMSET analyses (Figure 4.36), although no residual load was considered at the time. Finally, it should be noted that no sign of ‘soft toe’ can be found from the results. This confirms the effectiveness of the base cleaning methods (i. e., ‘MPA’ bombs for the polymer piles, fluid exchange for the bentonite pile, and base cleanliness checks for all piles) used during the construction of the piles. Of course, the low sand content and the low fluid density also served to ensure that the base conditions would not deteriorate during the period between base cleaning and the casting of the first truck load of concrete. This period was about 3 h during the construction of the test piles.

4.6.8 Load Transfer Analyses

Background Theory

To obtain the shear modulus (G) and the peak shear stress (τ_p), load transfer analyses were carried out using the computer program RATZ (Randolph, 2003). This program was also used by other researchers such as Randolph & Jewell (1989), Yetginer *et al.* (2006), and Lehane (2009) to back-analyse pile load test results. In the program, the pile is discretised into a number of elements, each of which is connected to a linear spring which represents the pile stiffness, and to a non-linear load-transfer spring which represents the soil resistance (Figure 4.55). The adopted form of the load transfer curve provides sufficient flexibility to model most aspects of soil response, e.g. non-linear work-hardening behaviour prior to peak followed by strain-softening after peak. Figure 4.56 shows the general form of the load transfer curves for the pile shaft and pile base. These curves are discussed briefly as follows.

As can be seen in the Figure 4.56a, there are three main stages to the load transfer curve for pile shaft, namely, an initial linear stage, a parabolic stage, and a strain-softening stage. The extent of the linear stage is controlled by the parameter ξ which may be taken anywhere in the range from 0 to 1. During this initial stage, the shear stress is assumed to be proportional to the normalised pile displacement, i.e., $\tau_0 = kw/r_0$, where k is the initial gradient of the load transfer curve. The value of k may be expressed as G/ζ , where ζ equals $\ln(r_m/r_0)$ and typically has a value of about 4 (Randolph & Wroth, 1978; Baguelin & Frank, 1980). As the shear stress level reaches the maximum value that the interface between pile and soil can withstand, i.e., when $\tau_0 = \xi\tau_p$, a rupture surface will develop close to the pile and hence the shear stress level in the parabolic stage can be expected to vary with the absolute displacement of the pile, w . As for the linear stage, the shape of the parabolic curve is dependent on r_0 , G , and ζ but also on τ_p which defines the vertex of the parabola. The initial gradient, k , now equals to $d\tau/dw = G/\zeta r_0$ (Randolph & Jewell, 1989), and for a parabola starting from the origin, the displacement required to mobilise τ_p is $2\tau_p/k$. After the peak, the strain-softening stage can be modelled by specifying the residual shear stress τ_r , the displacement to residual Δw_{res} , and the parameter η which controls the shape of the strain-softening curve and has a typical range of between 0.5 and

1.5. The equation of this curve can be found in Randolph & Jewell (1989). On unloading, say from a point when $\tau_0 = \tau_f$, the yield stress is taken to occur at the same point as for loading, i.e., $-\xi\tau_p$, and the negative failure shear stress is also assumed to be the same magnitude, i.e., $-\tau_f$. Further details about the modelling of cyclic loading can be found in Randolph & Jewell (1989) and are not repeated here. Finally, it should be noted that although the G value obtained in this way will be the initial tangent value, it will be lower than the small-strain stiffness of the soil (G_0) due to construction effects. Possibly for this reason, Deeks *et al.* (2005) and Yetginer *et al.* (2006) describe this parameter as the initial operative soil stiffness (G_{oper}) in their analyses.

As shown in Figure 4.56*b*, the load–displacement (q_b – w_b) response of the pile base has also been taken as parabolic in shape, with the vertex of the parabola being the ultimate unit base resistance (q_{bf}). The properties of a parabola are such that the initial gradient of the pile response is $2q_{bf}/w_{bf}$, where w_{bf} is the base displacement at q_{bf} . After reaching q_{bf} , the base resistance is assumed to remain constant, independent of the magnitude of the base displacement. In other words, for displacements outside the range of the initial parabola, the parabola is ‘dragged’ along the x -axis without changing shape. The unloading and reloading response is assumed to be elastic, subject to the condition of no tension being developed at the pile base.

Results and Discussions

Using the load-transfer model described above, the various soil parameters were adjusted to seek an overall best-fit match to the measured load–displacement (P – Δ and P – w) and shear stress–displacement (τ – w) response. To simplify the task, the values of G and τ_p were assumed to be constant for each soil layer, as opposed to, say, a linearly increasing trend. The pile was also assumed to have a constant elastic modulus (as in the previous CEMSET analyses), although in reality it should vary both with depth and strain level as discussed in Section 4.6.5. Figures 4.57 to 4.59 show the measured and simulated load–displacement responses for the three piles at the various levels where data are available. Figures 4.60 to 4.63 show the measured and simulated shear stress–displacement response for each idealised soil layer. Table 4.4 provides a summary of the parameter values used.

Overall, it can be seen from the figures that, within the limitations of the program, a

reasonable match between the predicted and measured behaviour has been obtained for the three test piles, both in terms of their load–displacement and shear stress–displacement response. To aid the following discussion, Figures 4.64 and 4.65 plot the back-analysed τ_p and G as a function of depth for the three test piles. It can be seen from the figures that generally the values of the soil parameters for the two polymer piles are very similar between themselves, but are different from those of the bentonite pile. This finding is consistent with the results of the previous Chin and CEMSET analyses. The back-analysed values for each soil layer are compared in greater details as below (except the made ground which was supported by a temporary casing).

In the Woolwich and Reading Formations, the G and τ_p values for the three test piles can be said to show little difference given the uncertainties involved, e. g. possible variation in the ground conditions, slight difference in the actual pile dimensions. However, in the underlying Upnor Formation, a clear improvement in G can be found for the two polymer piles when compared to their bentonite counterpart. This improvement is also accompanied by a noticeable increase in τ_p , although the difference is still within the range of expected scatter. The back-analysis results obtained for these two layers appear to confirm the observation of Brown (2004) that polymer fluids have an insignificant effect on the τ_p of clay (Section 4.2). However, as the results also indicate that polymer fluids have a small but positive effect on the value of G , this may still explain why polymer piles in clay are still commonly observed to show satisfactory P – Δ response (Section 4.2).

As shown previously in the Section 4.48, the choice of support fluids can have a significant effect on the interface shearing resistance between the pile and Thanet Sand. The τ_p values shown in Figure 4.64 actually reflect the consequences of different choices for δ values for this soil layer during the estimation of residual load. The back-analysed G values, on the other hand, are less sensitive to the decisions made during the previous steps. It can be seen that the two polymer piles have virtually the same G value of about 120 GPa. It is interesting to note that at this site the small-strain stiffness, G_0 , of Thanet Sand is estimated to be about 150 GPa using the G_{vh} – p' curve given by Ventouras & Coop (2009) and the calculated depth-averaged p' of 520 kPa. This means that the back-analysed G is 80% of G_0 , thus suggesting little soil disturbance was caused in this

layer during the excavation process.

The back-analysed q_{bf} and w_{bf} values are given in Table 4.4. They also merit some discussions as follows: since the mobilised base resistance was rather small for the two polymer piles (Figure 4.54), during the back-analysis it was decided to use the ultimate base resistance values previously obtained from the Chin analyses as the input values for q_{bf} , and then adjust the w_{bf} values to obtain a best-fit match. In fact, additional trials show that even if the chosen q_{bf} values are increased or decreased by, say, 20%, the effect on the back-analyses is still relatively small and can be easily compensated by adjusting the values of w_{bf} . This is because only a relatively small amount of base resistance was mobilised both in reality and in the numerical model. Of course, if the simulated pile load is taken beyond the actual proof load of 18.1 MN, then these values will have a much greater effect. Although the chosen w_{bf} values are only rough estimates, it is still interesting to see that they correspond to 8.3%, 16.7%, and 9.2% of the base diameter for piles P1, P2, and B1 respectively. These values are not far from the expected values since, according to Fleming *et al.* (2009), the base displacement required to mobilise q_{bf} of a low-displacement pile in granular soil is likely to be larger than the typical 5% to 10% range.

To conclude this discussion, Figure 4.66 shows the simulated response of the three test piles outside the actual load range. The level of pile head settlement (Δ) at which the shaft resistance becomes fully mobilised is also marked on the plot. These displacements correspond to 1.9%, 1.6%, and 2.1% of the pile diameter for piles P1, P2, and B1 respectively. It is interesting to note that these values are all higher than the typically assumed value of 1%. This is believed to be due to the presence of the residual load, because during the initial loading stage some shaft displacements were needed to overcome the NSF and to reverse the shear stress direction to positive. Other influencing factors also include the pile dimensions, the stiffness (compressibility) of the pile material, and of course, the choice of support fluid as shown in this study.

4.6.9 Implications for Pile Design

Soil stiffness and shear strain

Although the the back-calculated soil parameters (G and τ_p) given in Table 4.4 clearly show the effect of support fluids, they only apply to the ground conditions at Stratford and cannot be applied to other sites. Therefore, these parameters have been subject to further interpretation with an aim of producing design parameters that, hopefully, can be used for other projects. To achieve this objective, both soil stiffness and strength have been considered; the former is first discussed as follows.

As it is known that the ratio of small-strain stiffness to shear strength (G_0/S_u) typically ranges between 500 and 1000 following the work of Jardine *et al.* (1986)¹⁸, it is useful to compute this stiffness ratio for the soil adjacent to the test piles, and then use the result to assess the relative level of soil disturbance caused by the pile installation process. Table 4.5 shows the results for the two Lambeth Group units. It can be seen that the G/S_u ratios (also known as the rigidity index) are always higher for the two polymer piles. For the Woolwich and Reading Formations, this ratio is about 300 for the polymer piles and 200 for the bentonite pile. For the Upnor Formation, this ratio is about 500 for the polymer piles and 160 for the bentonite pile. These ratios indicate that the polymer fluid can preserve the quality of the soil better than their bentonite counterpart (not a surprising conclusion given what has been seen). For pile design at other sites, these ratios potentially can be used to estimate the initial stiffness of the two Lambeth Group units. It is interesting to note that, if the G/S_u ratios for the polymer piles are multiplied by a factor of two, then they will lie within the typical range of G_0/S_u as mentioned above. This means that, for the design of polymer piles in these soil layers, the initial G value can, as a rough estimate, be taken as 50% of G_0 if it is known (by seismic CPT, cross-hole seismic test, bender element test, etc.). It is interesting to note that Balakrishnan *et al.* (1999) and Chang & Zhu (2004) also reported a reduction of around 50% in the G value of the Singaporean and Malaysian residual silty soil due to the use of support fluids. Finally, as discussed before, for the two polymer piles the back-calculated G of the Thanet Sand is about 80% of the the estimated G_0 .

¹⁸In the original paper, the results were presented as normalised undrained Young's modulus, E_u/S_u .

As the stiffness degradation curves of some of the soil layers are available in the public domain, it is also possible to assess the level of soil disturbance based on the shear strain (γ_0) level in the soil. To do this, the normalised soil stiffness, E_u/p' , was calculated for the two Lambeth Group layers and the corresponding γ_0 levels read from the stiffness degradation curves given by Linney & Page (1996). Similarly for the Thanet Sand, using the estimated ratio of G/G_0 , the γ_0 level was also read from the stiffness degradation curves given by Ventouras (2005). However, since the curves given by Ventouras (2005) are not normalised (because the stiffness of Thanet Sand depends on the silt content, anisotropy, etc.), the strain level estimated in the Thanet Sand is only a rough estimate and should be treated as such. Table 4.5 shows the results. Not surprisingly, it can be seen that the overall strain level is much lower for the two polymer piles. If these strain values are averaged over the length of the pile, then the average γ_0 values will be 0.02% and 0.01% for piles P1 and P2 respectively. No depth-averaged strain level was calculated for the pile B1 since the γ_0 of the consolidated filter cake is not known. Nonetheless, it is interesting to note that the γ_0 values of the two polymer piles are similar but noticeably lower than the 0.03% estimated from the previous CEMSET analyses (Section 4.6.3). This is because in the CEMSET analyses the ‘long-term’ case was considered, while in the load transfer analyses it was the ‘short-term’ case that was considered.

Design α and δ values for shaft resistance

The τ_p values given in Table 4.4 were also used to back-calculate the adhesion factor, α , and the interface friction angle, δ , for design based on the principles of total stress and effective stress respectively. The relevant equations are:

$$\tau_p = \alpha S_u \quad (4.5)$$

$$\tau_p = K_s \sigma'_v \tan \delta \quad (4.6)$$

where K_s is taken as 1.15 as mentioned in Section 4.4.4, and the depth-averaged values of σ'_v for each soil unit are given in Table 4.5. For the back-calculation of δ , it is assumed that the effect of support fluid can be expressed exclusively through this parameter (see Majano *et al.*, 1994; Article 53.5.4 in Terzaghi *et al.*, 1996). While this is a reasonable assumption for the comparison between the polymer pile P1 and the bentonite pile B1

(both of them have a soil–fluid exposure time of 7.5 h), the effect between the two polymer piles in theory should be expressed in K_s , since any difference between them should be due to the effect of stress relaxation. However, as their behaviours are very similar, it was decided to use the same K_s value for their back-analyses.

The back-calculated α and δ values are given in Table 4.5. The former is discussed first: as can be seen, the difference between the α values are rather insignificant. This is because of the small difference between τ_p values. Nonetheless, two conclusions can still be drawn from the results. First, the commonly accepted design value for α of 0.4 reported by Hight *et al.* (2004) for Lambeth Group is adequate but appears to be slightly too conservative, as the average value found in this study is 0.6 (note the chosen S_u profile was determined based on SPT N values – see Figure 4.8). Second, a pile bore open time of up to 26 h (which is the total soil–fluid exposure time for pile P2) does not seem to reduce the α value if the bore is supported with a polymer fluid. A similar conclusion about the negligible effect of pile bore open time was also reported in Wheeler (2003b) for a nearby site in Canary Wharf, but the details are not known as the load test results were not given. As noted previously in Section 4.3, BBGE had to reduce their design α value from the 0.5 to 0.35 for pile bores which were left open overnight at the Castle House site. The finding of this study should therefore help them and other contractors to deal with similar issues in the future.

The values of the back-figured interface friction angle (δ) also merit some discussions. If this parameter is calculated for the Woolwich and Reading Formations using Equation 4.6, then the resulting values will be between 34° and 35° . These values are higher than the likely ϕ'_p values for this combined layer because according to Table 4.1 the highest documented ϕ'_p value for the Reading Formation (Lower Mottled Clay) is 28° and those for the Woolwich Formation are 21.5° (Laminated beds) and 24° (Lower Shelly Clay). The reason why the back-calculated δ is higher than ϕ'_p is probably due to the K_s value used which is 1.15. As the K_0 value for the Lambeth Group could be anything between 1.5 and 2.5 (Section 4.4.4), a higher K_s of up to 1.5 is possible.

For the Upnor Formation (Table 4.1), the back-calculated δ values are 25° and 30° for polymer piles P1 and P2 respectively and 20° for the bentonite pile B1. These values

were calculated using an assumed K_s value of 1.15. Compared to the expected ϕ'_p of 33° , the δ values are about 10% to 25% lower for the two polymer piles and 40% lower for the bentonite pile. The reduction is probably due to the combined effect of stress relaxation and increased moisture content in the soil immediately adjacent to the pile shaft (e. g. Chang & Zhu, 2004).

Lastly, for the Thanet Sand it can be seen that the δ values are 32° for the polymer piles and 16° for the bentonite pile. As discussed in Section 4.6.6, these values were determined using the results of the interface shear box tests and then used for the estimation of the shaft resistance in this layer. This finding indicates that the current design practice of taking δ as 36° (ϕ'_p) is unconservative especially when bentonite fluids are used. For information, the current design practice can be found in foundation design textbooks such as Tomlinson (2001).

Base resistance

As discussed in the previous two sections, the back-calculated q_{bf} values are only approximate due to the limited amount of base resistance mobilised during the load tests. Therefore, these values have not been subjected to further analyses for design parameters such as N_q^* (Troughton & Platis, 1989; Nicholson *et al.*, 2002). Nonetheless, a useful conclusion can still be drawn from the q_{bf} and w_{bf} results, that is, the common problem of ‘soft toe’ with polymer piles can be avoided if the sand content in the fluid is low and the cleanliness of the pile base is properly inspected and if necessary re-cleaned before concreting (Sections 4.5.3 and 4.5.4).

4.7 Conclusions

This chapter has examined the application of polymer fluids for the construction of bored piles. In dealing with this issue, a comprehensive literature review of the documented case histories has been carried out. The review clearly shows that polymer fluids can offer many advantages over the conventional bentonite fluid, including stiffer load–settlement response and improved shaft resistance. However, the review also shows that piles constructed using polymer fluids commonly suffer from the problem of ‘soft toe’ which is caused by the accumulation of soft sediment at the pile base. Other issues such as the effect on soil

stiffness and the effect of extended pile bore open time are also not clearly understood and the analyses presented show that resulting designs are unnecessarily conservative in some cases.

To advance the current state of the knowledge, a field trial comprising the testing of three bored piles was carried out at a test site in Stratford, East London. The site conditions, construction details, support fluid properties, and the analysis methods have been clearly set out before the results are discussed. It was found that a pile bore open time of up to 26 h had virtually no effect on the soil if supported by a polymer fluid (SlurryPro CDP). The choice of support fluid was also found to have little effect on the peak shear stress of the cohesive Lambeth Group although a noticeable effect was found on the shear stiffness. As demonstrated in a series of supplementary interface shear box tests, a much higher peak shear stress can be achieved in the Thanet Sand if polymer fluids are used. If bentonite fluids are used then the peak stress can be expected to be close to that of the consolidated filter cake even when the exposure time was relatively short. Finally, the base resistance was found to be comparable for the three piles. This demonstrates the importance of fluid properties and base cleaning and checking method for the prevention of ‘soft toe’ when polymer fluids are used.

Chapter 5

Concluding Remarks

5.1 Introduction

This thesis has been concerned with the properties and applications of polymer support fluids in geotechnical engineering and particularly in piling. These two research objectives were identified by the author following a comprehensive review of the current situation of these ‘new’ construction materials. It was found that polymer fluids have excited many and polarised views amongst foundation engineers – some try to avoid them after hearing ‘horror stories’ about their past failures, but others believe that polymers are the future of support fluids. In the author’s view, the difference between these opinions is a result of just one thing – the vast majority of geotechnical engineers do not understand them and think ‘polymer is polymer’, that is, all polymers are the same and if there have been problems with one polymer type or application there will be problems with all polymers and applications. This may be a bold statement, but based on the observations of the author it cannot be far from the truth. There are many reasons behind this unfavourable situation; these reasons have been discussed in Chapter 1 and in the introductory sections of Chapters 2 and 3. To improve this situation and to advance the current state of the knowledge, this thesis has sought to demystify polymers, and, with the new results, to illustrate how they can affect the performance of bored piles via a comparative case study. The results have been discussed in detail in Chapters 2 to 4. In this final chapter, the key findings are summarised, the overall success of this research project is evaluated, and suggestions are given for future work.

5.2 Summary of Main Findings

5.2.1 Fundamental Properties of Polymer

The main findings presented in Chapter 2 can be summarised as follows:

- Based on a review of the literature, it was found that most polymer suppliers currently promote a base polymer plus additives system, although some pre-blended products also are available for simplified uses. Partially hydrolysed polyacrylamide (HPAM) is by far the most common type of chemical used as the base polymer, and a wide range of additives are also available for improving the performance of the base fluid in difficult ground conditions. A summary of the different polymer products has been produced.
- A scanning electron microscope (SEM) was used to examine the microscopic appearance of selected dry polymer products. It was found that under the microscope the dry HPAM granules typically show glassy surfaces and sharp edges. These features are believed to be caused by the manufacturing process. On the other hand, natural biodegradable polymers such as guar gum, xanthan gum, and carboxymethyl cellulose have very different particle size and shape. As expected, many different types of particles were found in the blended polymer products. The results confirm the use of microscopy as a first step in characterising polymers.
- X-ray spectrum analyses showed that some particles with similar appearance can have very different chemical composition. For the HPAMs, the amount of sodium can be used as an indicator of the degree of hydrolysis. The composition of the blended polymers is much more complicated than that of plain polymers.
- The solid content of the commercial dry HPAMs tested in this study all satisfy the minimum requirement of 85% as proposed by some practitioners. The solid content of emulsion HPAMs is much lower due to the presence of oil as the carrier of the actual polymer materials.
- The amount of ‘fish eyes’ a dry HPAM can produce during mixing seems to depend on its molecular weight and solid content, but not the degree of hydrolysis. The amount of cross-linking, which was not explicitly considered in the analysis, is obviously another factor since cross-linked sodium polyacrylates are commonly used as superabsorbent polymers in consumer products such as sanitary towels and baby diapers. If mixed properly, natural polymers do not form ‘fish eyes’ although they have a stronger tendency to clump together during wetting out. Emulsion HPAMs also do not form

‘fish eyes’ as they are already ‘wet’.

- The range of the degree of hydrolysis values measured on the commercial dry HPAMs corresponds reasonably well with the range given by O'Neill & Reese (1999). The degree of hydrolysis of emulsion HPAMs was found to be much lower than those of their dry counterparts.
- The range of molecular weights measured on the commercial dry HPAMs is wider than the range given by O'Neill & Reese (1999). The molecular weights of emulsion HPAMs were found to be much lower than those of their dry counterparts.
- By using an environmental SEM, a three-dimensional network structure was directly observed in an aqueous solution of a HPAM. The microstructure seen is believed to be due to entanglements between the long-chain polymer molecules. Similar results were also reported by Feng *et al.* (2002) who was working on this imaging technique for liquid polymers but not by the few geotechnical engineering researchers who attempted this work.

5.2.2 Rheological Properties of Polymer

The main findings presented in Chapter 3 can be summarised as follows:

- Based on a review of the civil engineering literature, it was found that there is currently no consensus as to which mathematical model to use to interpret the Fann viscometer results. A simple assessment showed that the commonly used Bingham plastic model is clearly unsuitable for polymer fluids, should one consider the fitting of data on a viscosity plot. On the other hand, the power-law model was found to be able to represent the results very well over the shear-rate range measurable with this viscometer (5 to 1022 s⁻¹).
- By using the Bohlin rheometer, the flow curves of different polymer products were obtained over a much wider range of shear rates. It was found that all polymers tested showed signs of approaching limiting viscosities, η_0 and η_∞ , at very low (10⁻³ s⁻¹) and at very high (10³ s⁻¹) shear rates, respectively. This behaviour can be explained using the chain entanglement theory, and the results can be fitted with the four-parameter Carreau model. The difference between the flow curves of the various HPAM products

can be explained by reference to their fundamental properties such as molecular weight, degree of hydrolysis, and solid content. This shows that HPAM is a material that can be specified using these basic parameters to meet requirements of the users.

- By varying the polymer concentration of an HPAM solution, it was found that both η_0 and η_∞ display a power-law relationship with the concentration. It was found that a change in the polymer concentration has a larger effect on the viscosity at low shear rates. This phenomenon again can be explained by using the chain entanglement theory. From the test results, it was deduced that the critical polymer concentration (c_p^*) of the HPAM tested is likely to be very low. By comparing the Marsh funnel results with the rheometer results, it was found that, for the same type of polymer, the Marsh funnel ‘viscosity’ does show some relationships with the true viscosity parameters such as η_∞ .
- From a field trial in central Glasgow, it was found that the HPAM fluids showed an initial drop in their viscosity even without any external influences. This phenomenon is believed to be caused by the escape of the entrained air bubbles since the fluids were mixed using compressed air. A further drop was also found when the fluids were transported to the pile bores using a centrifugal pump and circulated within a loop system when not required. The results confirm the caution given in O’Neill & Reese (1999) and Brown *et al.* (2010) that HPAM fluids are susceptible to damage by shear in centrifugal pumps.
- Laboratory tests showed that the ageing characteristics of HPAM fluids largely depend on the quality of the mix water. Fluids mixed using deionised water showed a larger reduction in viscosity than those mixed using the relatively hard Oxford tap water, although for the same concentration the starting viscosity of the former was much higher. This is probably because the polymer chains in the deionised water were fully expanded due to charge repulsion and hence there was more room for them to return to their natural state. By the same token, since the polymer chains in the Oxford tap water were already compact (more coiled) due to some charge neutralisation, fewer conformational changes could take place.
- From the field monitoring results obtained from the Castle House site, it was found that the changing pattern of the polymer fluid density was highly dependent on the

practice of the contractor. It was also found that the Marsh funnel viscosity tended to drop as the fluids went through various parts of the system. However, this parameter could not clearly reflect the effect of increased fluid density. On the other hand, the two power-law parameters, n and K , showed much better correlations with the fluid density. This finding highlights the weakness of the Marsh funnel and demonstrates the usefulness of the Fann viscometer as a quality control instrument for polymer fluids on site.

- Amplitude sweep tests in the Bohlin rheometer showed that high-molecular-weight HPAM fluids yield at a shear strain level of about 1, which may be compared with the only 1% to 2% strain in bentonite fluids as reported by Santagata *et al.* (2008). It was also found that both storage and loss moduli, G' and G'' , increase with the polymer concentration and their curves have the shape of a parabola. Similarly, the yield and the flow stresses also showed a parabolic relationship with the polymer concentration. The amplitude sweep test results provided the necessary background information for other viscoelastic experiments.
- Frequency sweep tests showed that at low frequencies the rheological properties of HPAM fluids are dominated by their viscous behaviour, whereas at high frequencies the elastic behaviour dominates. This sort of behaviour is not unique to HPAMs as it also has been seen on other types of polymer (Carnali, 1991; Benchabane & Bekkour, 2008). Once again, the chain entanglement theory can be used to explain the behaviour observed.
- Creep tests showed that the compliance (normalised strain) of a HPAM fluid is highly dependent on the polymer concentration, and that the multi-element Burgers model can represent the results very well. It was also found that for the polymers tested, the instantaneous compliance, J_0 , is zero for all polymer concentrations, meaning that the elastic response of a polymer fluid is always delayed.

5.2.3 Application of Polymer Fluids to Bored Pile Construction

The main findings presented in Chapter 4 are summarised as follows:

- Based on a literature review, it was found that the shaft resistance of a polymer pile is often better than that of bentonite pile. However, the problem of ‘soft toe’ is very

widespread probably due to the inability of the polymer fluid to hold the soil particles in suspension. Pile bore stability is usually not a problem if the ground does not contain a high level of salts and the polymer is used properly. To tackle the problem of high fluid loss into the ground, mixed polymer-bentonite fluid systems were used in some cases but probably at the cost of a lower shaft resistance. The effects of polymer fluids on the structural integrity of piles and on soil stiffness are not yet properly understood.

- To address some of the unresolved issues and to provide further evidence of the effect of polymer fluids in a UK setting, three bored piles have been constructed using both polymer (SlurryPro CDP) and bentonite (Berkbent 163) fluids. The site was located in Stratford, East London. The ground conditions, construction details, and support fluid properties were clearly set out to aid the interpretation and understanding of the load test results.
- The structural integrity of the test piles were investigated by both cross-hole sonic logging (CSL) and pile integrity tests (PIT). The quality of the concrete was also compared by testing samples taken from the concrete–fluid interface. From the CSL results, it was found that each of the polymer piles had a flaw in concrete. Although the actual cause was not known, the flaws were possibly due to the damage of the access tubes or the formation of some water channels inside the fresh concrete. The PIT found no anomalies. Laboratory tests on the hardened concrete samples showed that both polymer and bentonite fluids have similar adverse effects on the compressive strength and the elastic modulus.
- From the results of maintained load (ML) tests, the two polymer piles were found to show stiffer load–settlement ($P-\Delta$) response than the bentonite pile especially at the higher loads. Using the Chin’s analysis method, the ultimate shaft and total resistances of the piles were estimated. As expected, the two polymer piles were predicted to have higher ultimate resistances than the bentonite pile due to their lower settlement values. From the results of the CEMSET analyses, the shaft response of the two polymer piles were found to be comparable to piles in very stiff soils or soft rocks, and that of the bentonite pile was typical of piles in firm soil. This suggests that the level of soil disturbance due to pile installation was much lower when polymer fluids were used.

- An examination of the strain gauge results showed that a significant amount of concrete creep (ε_{cc}) developed during the load-holding periods of the ML tests. Therefore, the creep strain was removed from the measured values. In addition, a comparison between the measured and predicted ε_{cc} values showed that the pile concrete was weaker than expected and the external steel casing at the pile head did not contribute to the deformation behaviour of the pile. This latter finding allowed the back-calculated pile modulus curve to be applied to other instrumentation levels without any adjustments.
- Based on the site conditions and the shape of the load-distribution curves, it was concluded that there were some residual loads in the piles before the start of the load tests. The residual loads were quantified using the analysis method proposed by Fellenius (2002b). Due to the limitation of the method, the interface shear resistance in the Thanet Sand had to be determined by another method. Some interface shear box tests were therefore carried out under conditions similar to those at the test site. The results showed that the angle of shearing resistance, ϕ'_p , is 36.5° , and the interface friction angles, δ , are 32.5° and 16.5° for polymer and bentonite fluids respectively. The difference between the δ values can be attributed to the presence of a consolidated filter cake between the concrete and the sand when bentonite fluids are used.
- By using the load transfer program RATZ, the peak shear stress, τ_p , and the shear modulus, G , have been back-calculated for each soil layer. For the two Lambeth Group layers, it was found that, overall, polymer fluids have an insignificant effect on τ_p but a more noticeable positive effect on G . For the Thanet Sand, the τ_p values simply reflect the choice of the δ values used for the determination of the residual load. The back-calculated G value was found to be about 80% of the predicted small-strain shear modulus, G_0 . This latter finding suggests that little disturbance was caused in the sand during the excavation process, and hence the stabilising power of the polymer fluids (note that water cannot stabilise Thanet Sand as noted by Troughton, 1992). Due to the relatively small proportion of base resistance mobilised during the load tests, the chosen values of the ultimate unit base resistance, q_{bf} , and the corresponding base displacement, w_{bf} , were found to have little effect on the results of the load transfer analyses unless the simulated load is taken beyond the actual proof load.

- The back-calculated G have been used to assess the level of soil disturbance caused by the pile installation process. It was found that, for the design of polymer piles in the Lambeth Group, the initial G value can be taken as 50% of G_0 if this latter value is known from in-situ or laboratory tests. By considering the stiffness degradation behaviour of the various soil layers, the overall local shear strain, γ_0 , was estimated to be between 0.01% and 0.02% for the two polymer piles. These values correspond reasonably well with the estimated γ_0 of 0.03% from the CEMSET analyses which include the full effect of consolidation and creep. Also from the CEMSET analyses, the γ_0 of the bentonite pile was estimated to be 0.15%, which is five times the γ_0 of the polymer piles.
- The back-calculated τ_p values have been analysed to deduce the adhesion factor, α , and the interface friction angle, δ , for pile design based on the principles of total and effective stress respectively. For the Lambeth Group, the average α value is about 0.6 compared to the current design value of 0.4 as reported by Hight *et al.* (2004). Since the α values of the two polymer piles are very similar, this indicates that a pile bore open time of up to 26 h is unlikely to reduce the shaft resistance if the bore is supported by a polymer fluid. This finding should be useful for foundation contractors when dealing with this issue in the future. The back-calculated δ of the Woolwich and Reading Formations was found to be higher than the estimated ϕ'_p if the effective cohesion, c' , is assumed to be zero. For the results to make more sense, a c' of 35 kPa had to be used in the calculations – not an unreasonable assumption considering the range of c' values reported in the literature. For the Upnor Formation, the back-calculated δ was found to be 10% to 15% lower than ϕ'_p (33°) for the two polymer piles and 40% lower for the bentonite pile. Finally, the results obtained for the Thanet Sand indicates that the current design practice of taking δ as 36° (ϕ'_p) is not conservative especially when bentonite fluids are used.

5.3 Evaluation

This thesis has been concerned with the properties of polymer support fluids and their application to foundation construction. Equal emphasis has been placed on these two parts since they are considered equally important for the future development of this construction

technology. From the very early stage of this project, the author has come to believe that, to maximise the many benefits that polymers can bring and to prevent future failures, foundation engineers should start to appreciate the difference between the various polymer types that are available in the marketplace and to stop thinking ‘polymer is polymer’. Unfortunately, this understanding process is complicated by several factors. First, after years of product development by polymer suppliers, there are currently many different polymer systems available in the market and each system may also include a number of additives for enhanced fluid performance. As practising engineers are all busy professionals, very often they simply do not have the time to appreciate the differences. Second, because of the common lack of chemistry education for civil engineering students at universities, very few practising engineers would have the training to understand complex polymer chemistry without any guidance or help. Third, most polymer suppliers currently do not disclose the chemical properties of their products for commercial reasons. Although this is perfectly normal business behaviour, this practice makes it the more difficult for engineers to embrace polymer technology. For these reasons, polymer technology is currently seen by many in the industry as a ‘black box’.

To help practising engineers and site technicians to understand the properties of polymers, in the first part of this thesis (Chapters 2 and 3) the author has carried out several tasks to demystify the technology by making the existing information more understandable to them and also by revealing new information. These tasks are summarised briefly as follows to highlight the author’s contribution to the field.

To deal with the issue of a large number of polymer products which are currently only identifiable by their trade names, the author has categorised them based on their intended functions and chemical composition (Table 2.1 and Appendix A). Dave Ritchie (personal communication, January 2008), otherwise known as the polymer man at BBGE, once told the author that some inexperienced site operatives tend to ‘throw everything down the hole’ in the case of borehole instability or high fluid loss. The author believes that Table 2.1 and Appendix A will help them make reasoned decisions when selecting and using polymer products in the future. The information provided will also be a good starting point for anyone who wishes to understand this technology but does not know

where to start.

To compare the performance of the various polymers and to identify the material parameters that are the most relevant for product specification, the author has tested the chemical and physical properties of some of the more commonly used products. For HPAMs, the molecular weight and the degree of hydrolysis of the molecules have been identified as the key parameters since together they have been shown to dictate the viscosity of the fluids. The effect of polymer types (HPAM, CMC, xanthan gum, guar gum) on solution viscosity has been highlighted to show the importance of polymer selection. In addition to material properties, the rheological test results have also shown many other factors that can affect the properties of polymer fluids. These include water quality, mixing and pumping method, ageing, governing time scale (shear rate or frequency), suspended soil particles due to repeated re-use, and fluid maintenance strategy. The results presented will be useful for anyone who deals with polymer fluids whether they are the engineers in the office trying to select a suitable polymer product for a particular project or the site operatives trying to control and maintain the properties of polymer fluids on a construction site.

In the second part of this thesis (Chapter 4), a case history where both polymer and bentonite fluids were used for the construction of bored piles has been discussed. Although similar studies have been carried out in other countries before (Section 4.2), in the author's opinion this study is still much needed by the industry for two main reasons. First, despite the growing use of and interest in polymer fluids in the UK, comparative studies of this kind are still extremely rare in this country. For instance, a similar case history was reported by Corbet *et al.* (1991) using a polymer that was not identified¹⁹. Wheeler (2003b) also mentioned a comparative study at the Canary Wharf but did not present any test results. Therefore, the results presented in this study should be very useful for foundation engineers both in the UK and elsewhere. Second, compared to the other case studies, the one presented in this thesis is one of the very few that includes a detailed description of the fluid properties including a presentation of the flow curves. The advantage of this practice is that, if another pile is constructed using polymer fluids

¹⁹An enquiry was made to Mr S. Corbert about this, but unfortunately he cannot remember details about the polymer fluid type since the actual work was carried out more than 20 years ago in 1989.

with similar properties, similar results should be expected. If the pile turns out to be of poor structural quality or shows large settlement, then provided good polymer practice has been followed, construction practice or ground conditions can be considered as the more likely cause(s) of the problem than the polymer.

Although the second part of this thesis is about the application of polymer fluids, this study has by no means covered all their possible uses. For instance, another major use of polymer fluids, although still very limited in the UK, is for the excavation of structural diaphragm walls. This thesis has not made any attempt to cover this use since the considerations could be very different. For example, if a hydromill is used to excavate the soil, then the chemical demand on the polymers will be much higher since significantly more soil surfaces will be exposed to the polymers. This could turn the polymer fluids literally into muddy water within a very short period of time. A blended polymer product specifically designed for use with the hydromill has recently become available, but further testing both in the field and in a laboratory is still required to confirm its performance.

Wheeler (2003a), in a magazine article, quoted a practising foundation engineer saying “we’re probably where we were with bentonite thirty years ago” when commenting on the then current situation with polymers. The author shares this same view, and believes that much more work still needs to be done to bring us to a level where most engineers will feel comfortable with these ‘new’ materials. For the reference of other researchers, some suggestions are given for future work in the following section.

5.4 Suggestions for Further Work

5.4.1 Fluid Penetration into Soil and Stabilising Mechanism

In the twenty years or so after the first use of bentonite slurry as a support fluid in the 1950s (Veder, 1953), a considerable amount of work was put into the study of fluid penetration into soil (e.g. Jefferis, 1972; Hutchinson *et al.*, 1975; Fleming & Sliwinski, 1977). These works led to a much better understanding of the stabilising mechanism of bentonite fluids and, consequently, their acceptance in the industry. For polymer fluids, an understanding of this issue is even more important since they do not form a filter cake on the soil surface and hence any resistance to fluid loss into the ground must be due to

the non-Newtonian properties of the fluids. An understanding of this soil–fluid interaction problem would help explain why water cannot stabilise Thanet Sand but polymer fluids evidently can. The results also would be very useful for assessing the suitability of polymer fluids for use in very coarse soil.

To the author’s knowledge, so far the filtration behaviour of polymer fluids through soil particles has only been investigated by a few researchers, such as Majano & O’Neill (1993) and Lesemann (2010). However, if the search is extended to include other branches of engineering, then a considerable amount of work also can be found in the oil and gas literature, although those studies were usually carried out using rocks and under much higher filtration pressure (e.g. Sorbie, 1991). Nonetheless, in the author’s opinion, much more work is still required to understand the stabilising mechanism of polymer fluids. It is expected that any future work could be either experimental or numerical, or both. If it is numerical, then the discrete element method (DEM) appears to be a promising approach since it is known that soil–fluid interaction problems at particle level can be analysed by coupling with computational fluid dynamics (CFD) which can consider various types of non-Newtonian fluid properties. Regardless of the study method, it is anticipated that the rheological test results presented in this thesis would be central to the interpretation of the results.

5.4.2 Settling of Suspended Particles and the Formation of ‘Soft Toe’

As discussed in Section 4.2 and elsewhere, the problem of ‘sot toe’ is very widespread with the use of polymer fluids, due to their lower particle-holding ability than bentonites. Although this study has shown that this problem can be avoided by following rigorous fluid cleaning and base checking procedures, it is appreciated that it may not be always possible to bring the sand content down to what was achieved at the Stratford test site (0.1% to 0.2%) when the ground conditions consist entirely of sand, especially when the piles are excavated and cast in the same day. Therefore, it would be useful to carry out a series of laboratory tests to investigate the settling rate of sand particles in polymer fluids at different viscosity and sand content combinations, so that the likely thickness of soft sediment at the base of a pile bore or diaphragm wall trench can be predicted for

any given period of time. In fact, some small scale laboratory work has already been carried out at The University of Manchester using the ‘Turbiscan’ machine, and some useful results have been obtained. It is hoped that these results can be analysed in the near future for publication.

5.4.3 Study of Case Histories for Improved Pile Design Parameters

As a result of the literature review carried out for this study (Section 4.2), data from a substantial number of case histories have been collated. As noted previously, the author has not identified a single case where the unit shaft resistance of a polymer pile was reported to be lower than that of a similar bentonite pile. Therefore, if these case histories are analysed in a more systematic way, then it would be possible to refine the values of the various empirical parameters (α , β , δ , etc.) for the design of polymer piles. In doing so, the effect of different polymer types (HPAMs, blended, etc.) may also be evaluated. The results would enable engineers to take advantage of polymer fluids at the design stage as well as in the field. It is worth noting that, to the best of the author’s knowledge, it is very rare for any differentiation to be made between bentonite and polymer fluids by pile designers since no refined design parameter values are available, although many are certainly aware of the potential increase in shaft resistance when polymers are used.

5.4.4 Effect of HPAMs and Additives on Interface Shear Strength

Although the Stratford field tests reported in this thesis have made use of only a small number of polymer additives (ProTek 100, Slurry Pro LA-1-D, SlurryPro MPA), as shown in Table 2.1 many other additives are available. Of particular technical interest are those designed for weighting and increased borehole stability, since a few of them are believed to be chemical grouts with the chemistry of sodium silicate and sodium aluminate²⁰. Silicate–aluminate grouts have been used extensively in the past for seepage control and soil strengthening (e.g. Jefferis & Sheikh Bahai, 1995; Karol, 2003). If used in conjunction with an HPAM support fluid, before setting these chemicals could penetrate into the soil under the hydrostatic pressure of the fluid column. Since it is known that the shaft

²⁰Note that this information is not specifically disclosed by the polymer suppliers. It is an educated guess by the author and his colleagues based on the properties of the materials.

resistance/deflection of a pile is dominated by the soil immediately adjacent to the pile wall, even a small penetration (grouting) distance of, say, 10 mm, could potentially have a beneficial effect on the load-settlement behaviour of the pile. This may be the reason why some polymer suppliers claim that their products can have a ‘soft grouting’ effect on the soil.

In fact, (H)PAMs also have been used as chemical grouts and as soil conditioners (Berry, 1982; Wallace & Terry, 1998; Karol, 2003) – Figure 5.1 shows the grouting effect on some dry Leighton Buzzard sand. The sample on the left of the photograph was previously saturated with a high-molecular-weight HPAM fluid in a filtration cell and then left to dry in the air while seated on a porous stone base (the black disc in the photograph). The sample on the right was saturated only with water. The strengthening effect was evident when these samples were inverted. The one previously saturated with polymers was examined under a scanning electron microscope and the micrographs are shown in Figure 5.2. It can be seen that the dried polymer effectively glued the sand grains together. However, it was also found that if this sample was re-wet again, the effect would disappear due to the dissolution of the polymer. This explains why this effect was not noted previously during the interface shear box tests on Thanet Sand (Section 4.48) since the sand was kept fully saturated.

To summarise, in the author’s opinion the effects of HPAMs and silicate-based grouts on soil merit more research. An interface shear box or a pull-out device similar to the one used by Majano *et al.* (1994) may be employed for this purpose.

5.4.5 Effect of Polymer Fluids on Hardened Concrete and on the Bond Strength with Reinforcing Bars

Although this study has shown that both bentonite and polymer fluids appear to have the same detrimental effect on the strength and stiffness of hardened concrete (Figure 4.27), the results are only indicative since the actual degree of intermixing at the fluid-concrete interface is not known. To confirm the field results, a series of laboratory tests with a range of intermixing (contamination) ratios would be required. Once this issue was addressed, the next step would be to see if there is any reduction in the bond strength between concrete and reinforcing bars (rebars) when polymers are used (compared to

bentonites). In fact, some work has already been carried out by BRE (BRE, 2001) for the CTRL project and the results presented by Jones & Holt (2004) and Jones (2005). From their analyses, it was concluded that, when polymer fluids are used, the design bond condition should be assumed as poor until further information becomes available. It also was concluded that low-viscosity fluids ($t_{MF} = 60$ s) have a less detrimental effect on the bond strength than high-viscosity fluids ($t_{MF} = 100$ s). However, a closer examination of the original BRE test results reveals that the difference between the various tests is actually statistically insignificant and hence these conclusions may not be fully valid. Therefore, work is still required to investigate the effect of polymer fluids on rebar–concrete bond strength.

5.4.6 Development of a Set of Polymer Specifications

Unlike bentonite fluids for which a set of well-defined performance criteria are available (Federation of Piling Specialists, 1975; 2006), currently there is very little guidance on the required or desired properties of polymer fluids. This is not surprising given how little engineers currently know about them. Polymer fluids are currently specified using the same parameters as bentonites, i. e., pH, density, sand content, and Marsh funnel viscosity. The acceptable range of these parameters is usually provided by the polymer suppliers and sometimes modified by consulting engineers, local authorities, or even contractors who self-certify their work. This approach has three potential pitfalls. First, hypothetically, polymer suppliers may claim that their products can work at lower viscosities or at higher densities purely to attract more customers. Second, site staff may treat polymers as if they were bentonite and use inappropriate solution concentrations. For example, Marsh funnel times for polymer fluids tend to be higher than for bentonite slurries and if bentonite values are used for polymers problems can occur. Third, engineers may, with the laudable intention of improving quality, specify performance limits that are inappropriate or unnecessarily difficult or even impossible to achieve. For example, the Missouri Department of Transportation in the USA currently specifies that the density of a polymer fluid should not be higher than 63 lb/ft³ (1.009 g/cm³) (Missouri Department of Transportation, 2011). This performance criterion can be very difficult to meet, since the density of a used polymer fluid can easily exceed this level especially when sampled

from the bottom of a pile bore where the sand content is usually high. However, it is interesting to see that they allow a pile bore to be left open for up to 72 h if supported by a polymer fluid, probably due to the findings of Likos *et al.* (2004) and Axtell *et al.* (2009). Nonetheless, in the author's opinion, the lack of a set of common specifications is certainly hindering the wider acceptance of polymer fluids. Its development should be based on scientific research and should break away from the influence of bentonite which is a completely different type of material.

5.5 Final Words

In this thesis the properties and applications of polymer fluids for excavation support have been discussed with the ultimate aim of helping practising engineers and technicians to understand this technology better. The author believes that he has achieved this goal though much remains to be done to advance the knowledge further. However, one key question still remains: should we be using bentonite or polymer fluids from now on? Unfortunately there is no simple answer to this question because the decision will depend on a wide range of technical and non-technical factors, including the surface and subsurface conditions of the site, experience of the local contractors, environmental regulations, disposal costs, etc. To make it easier for practising engineers to compare the two fluid systems, their pros and cons have been set out in Table 5.1. As can be seen, an advantage of one fluid is often a disadvantage of the other, so probably both of them will continue to be used for the foreseeable future. However, with everything considered, polymers should have a bright future because they are more environment friendly and their properties are more controllable than bentonite. It is hoped that the work presented in this thesis will make it easier for the practising engineers to apply this promising technology in the future.

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- Zhao, D., Zhang, M., Zhang, D., Xu, H., Xu, P., Wu, R., Gao, G., & Liu, F. 2005. Studies on the viscoelastic properties of ultrahigh molecular weight partially hydrolyzed polyacrylamide. *Polym. Prepr.*, **46**(1), 777–778.
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Figures

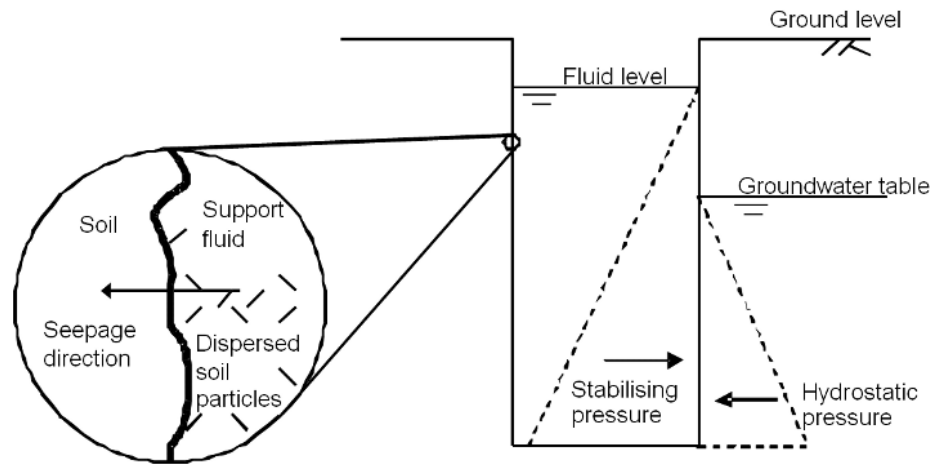


Figure 1.1: Schematic diagram of a fluid-supported excavation.



Figure 1.2: Polymer fluids being pumped into a pile bore.



Figure 1.3: Piling auger immediately after extraction from a polymer-supported bore.

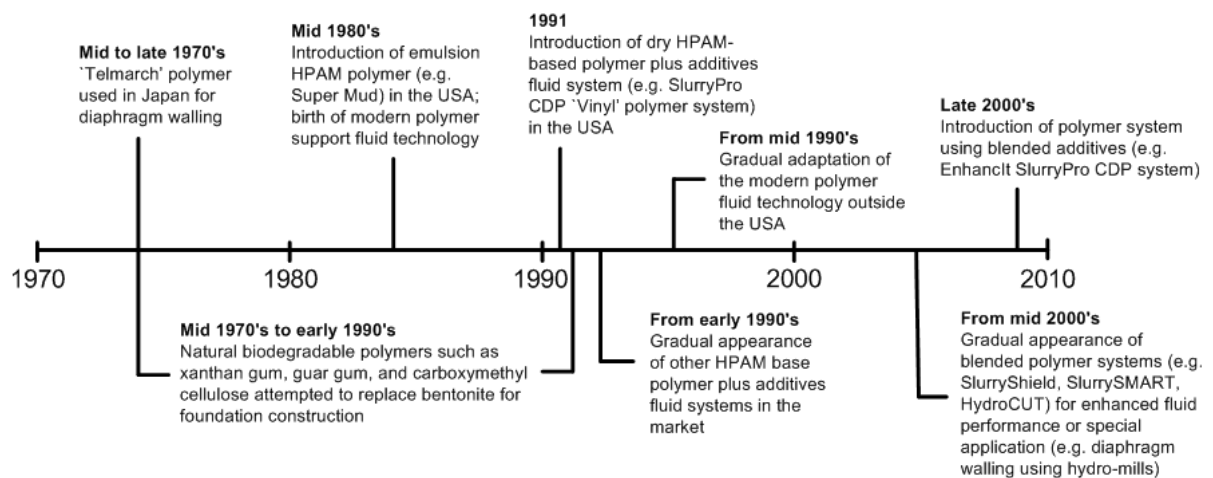
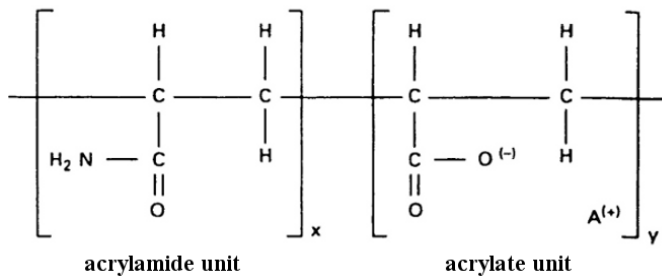
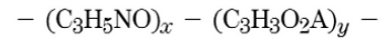


Figure 1.4: Timeline of the development of polymer support fluid.

Chemical formula:



Empirical formula:



where:

A is a positive ion, typically sodium;

x and y are variables depending on the product.

Figure 1.5: Chemical and empirical formulae of HPAMs. (Modified after British Standards Institution, 1998.)



Figure 1.6: Photograph of HPAM granules.

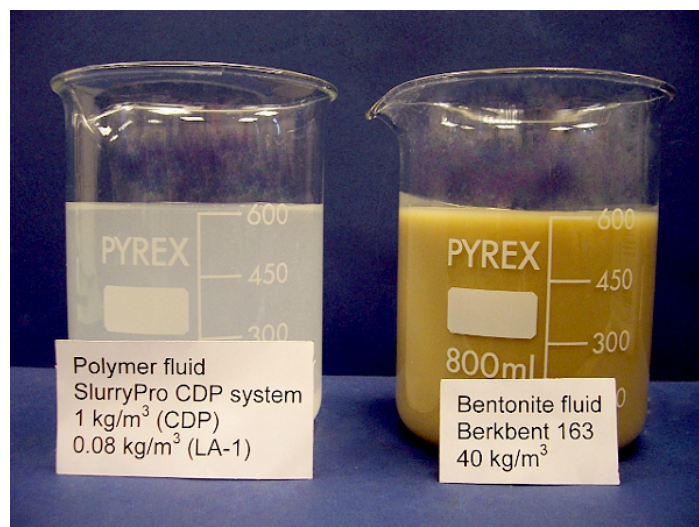


Figure 1.7: Photograph of synthetic polymer and bentonite support fluids.

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Figure 1.8: Photograph of a site in Glasgow where polymer fluids were used.



Figure 1.9: Sprinkling polymer granules into the mix water.



Figure 1.10: Adding emulsion polymer directly into a pile bore.

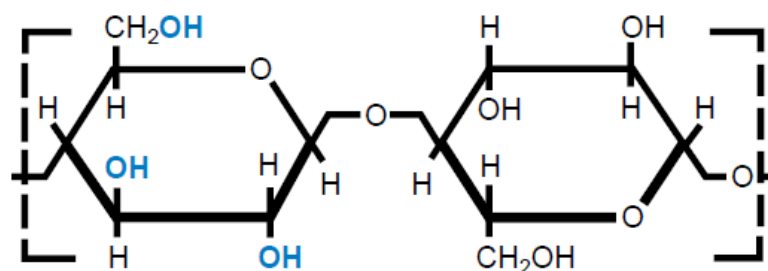


Figure 2.1: Structure of cellulose consisting of repeating anhydroglucose units.

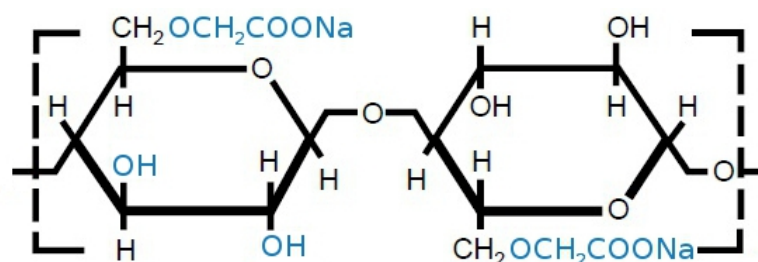


Figure 2.2: Structure of carboxymethyl cellulose (CMC), with a degree of substitution (*DS*) of 1.0.

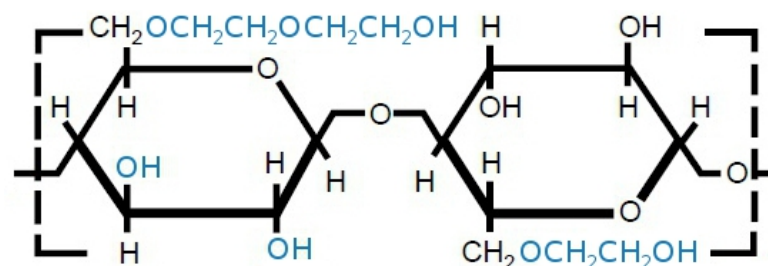


Figure 2.3: Structure of hydroxyethyl cellulose (HEC), with a degree of substitution (*DS*) of 1.0. The unit on the left shows a side chain resulting from a hydroxyethyl group reacting at a previously substituted hydroxyl.

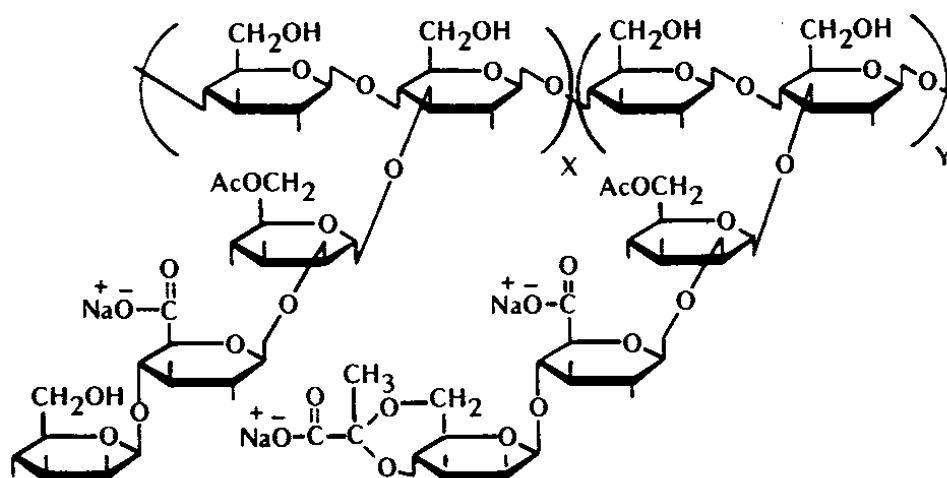


Figure 2.4: Structure of xanthan gum. (From Philips *et al.*, 1985.)

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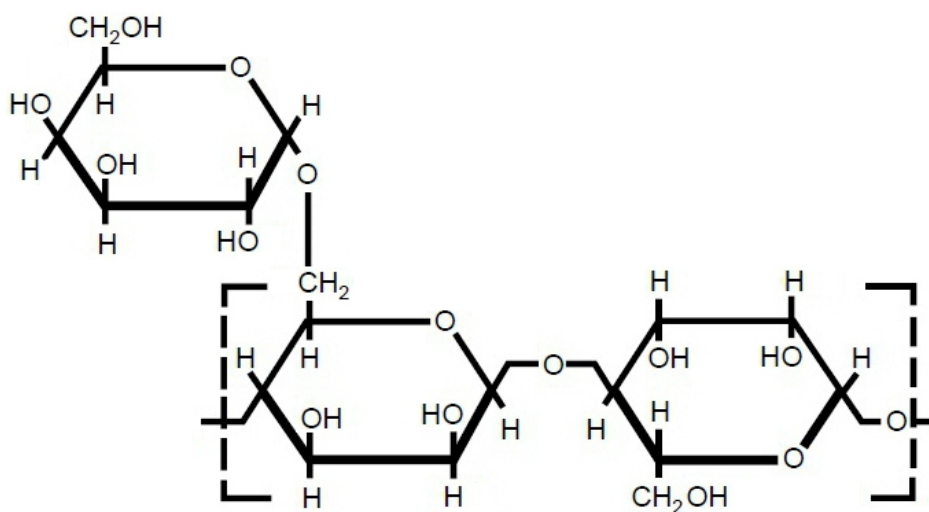


Figure 2.5: Structure of guar gum.

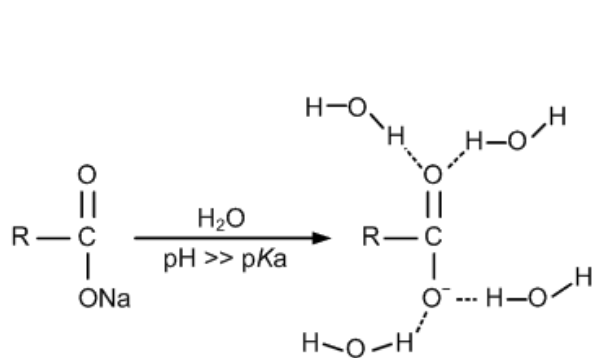


Figure 2.6: Carboxylate base: hydrogen bonding and ion-dipole attraction.

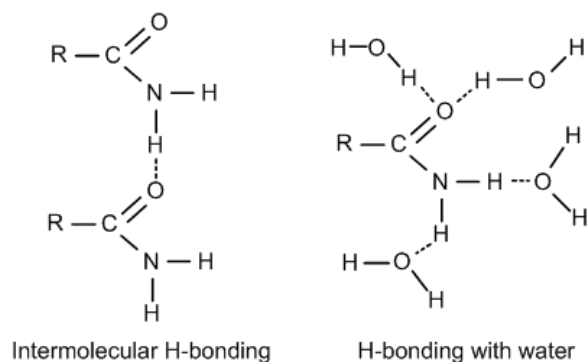


Figure 2.7: Amide group: hydrogen bonding with like molecules and with water.



Figure 2.8: Photograph of scanning electron microscope (SEM).

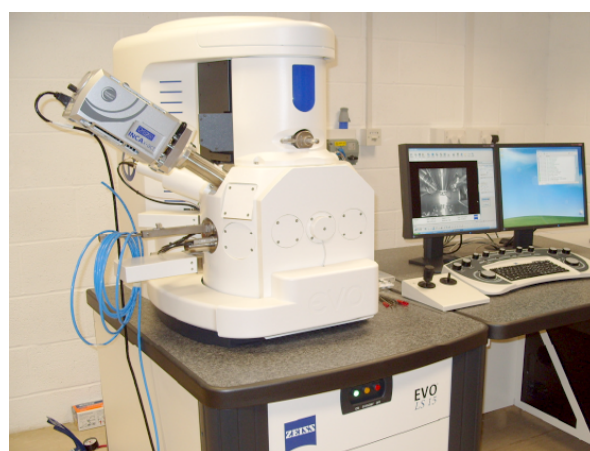


Figure 2.9: Photograph of environmental scanning electron microscope (ESEM).

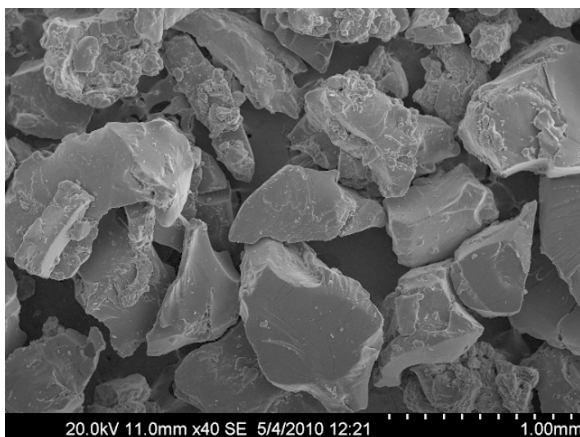


Figure 2.10: SEM micrograph of HPAM 'SlurryPro CDP'.

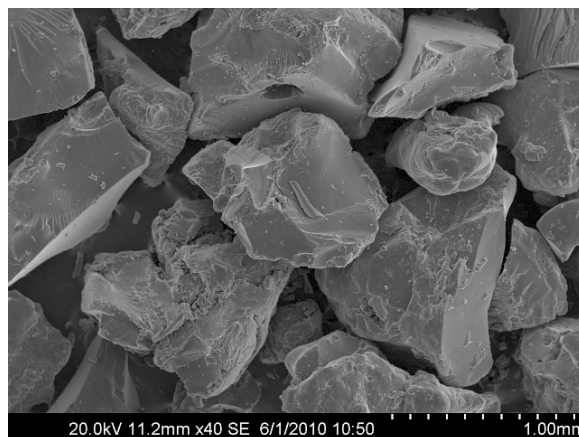


Figure 2.11: SEM micrograph of HPAM 'Shore Pac'.

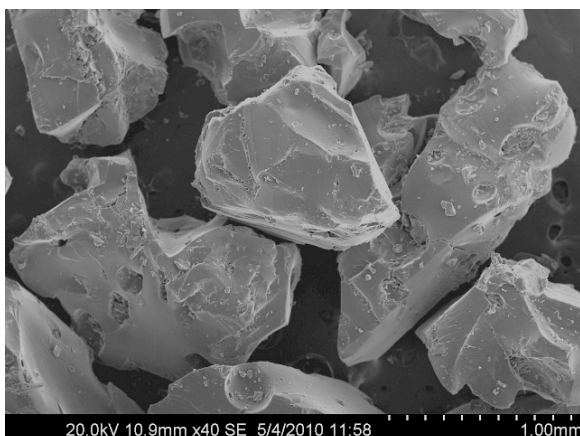


Figure 2.12: SEM micrograph of HPAM 'POLY-BORE'.

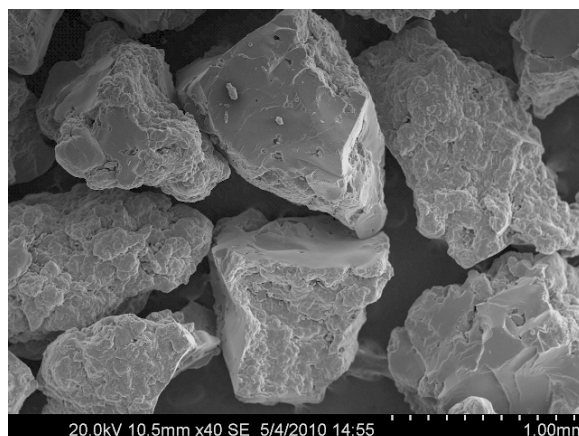


Figure 2.13: SEM micrograph of HPAM 'EZ MUD DP'.

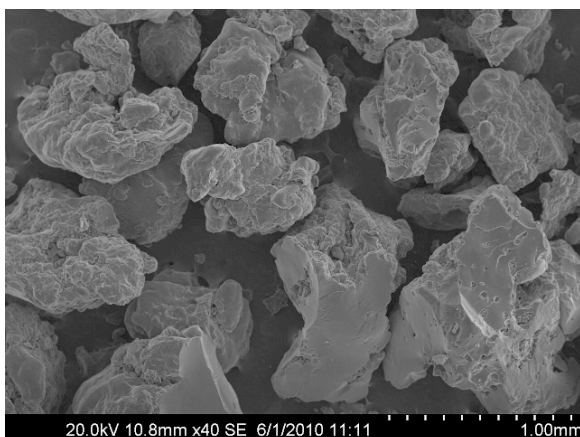


Figure 2.14: SEM micrograph of HPAM standard no. 1 (>10 million molecular weight; 40% degree of hydrolysis).

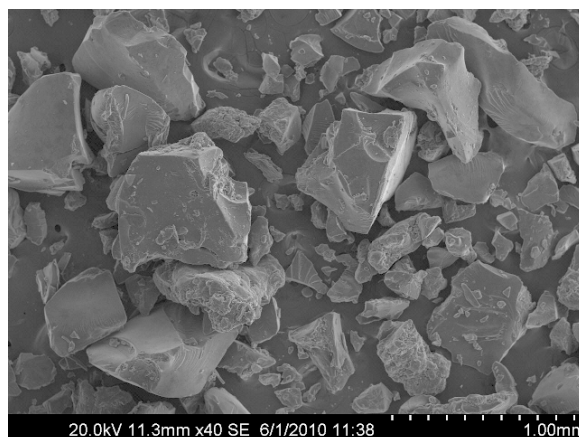


Figure 2.15: SEM micrograph of PAM standard no. 1 (18 million molecular weight).

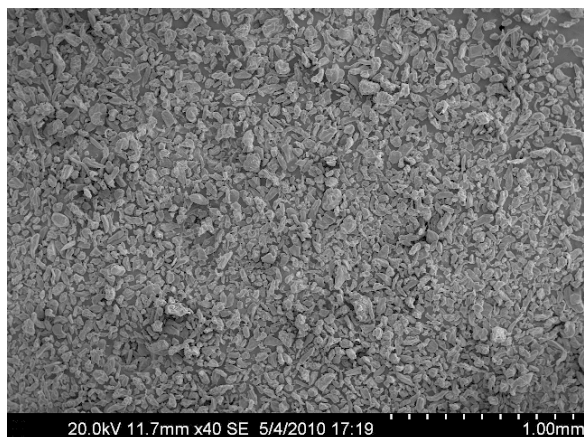


Figure 2.16: SEM micrograph of guar gum polymer 'SC GUM'.

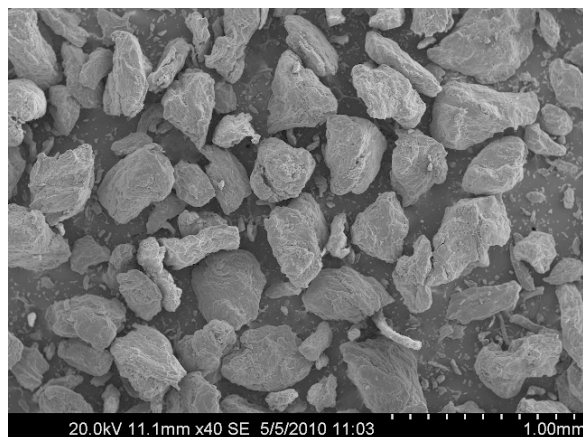


Figure 2.17: SEM micrograph of xanthan gum 'DUO-VIS PLUS NS'.

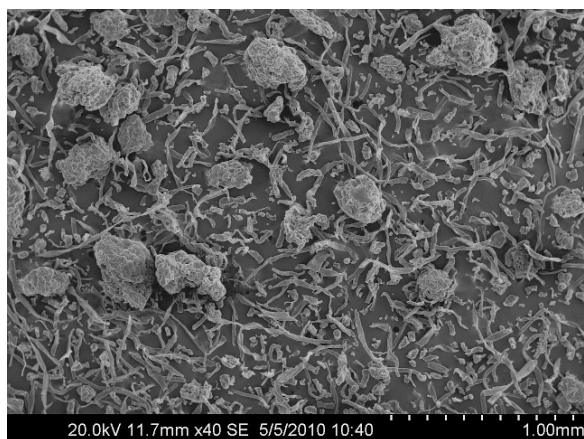


Figure 2.18: SEM micrograph of carboxymethyl cellulose 'Aqualon CMC 7H3SF'.

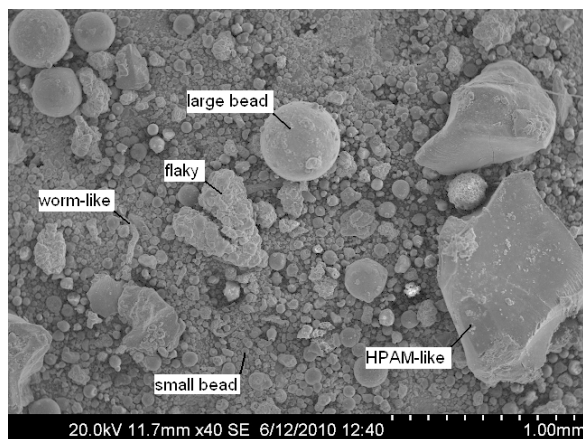


Figure 2.19: SEM micrograph of the blended polymer 'SlurrySMART'.



Figure 2.20: SEM micrograph of the blended additive 'EnhancIT 100'.

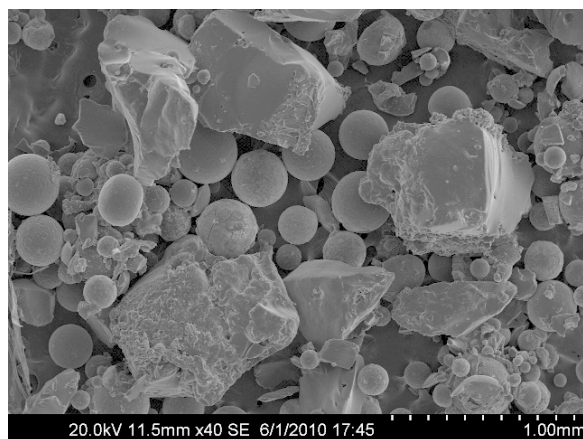


Figure 2.21: SEM micrograph of the blended additive 'EnhancIT 200'.

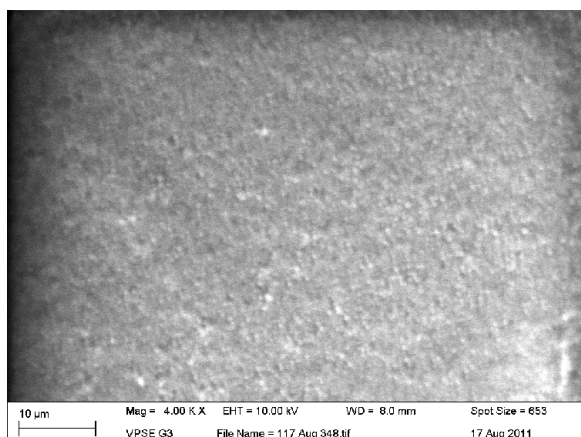


Figure 2.22: ESEM micrograph of an emulsion HPAM – possible surface feature.

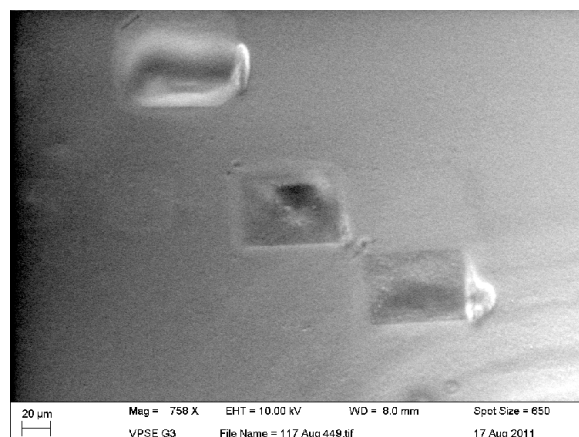


Figure 2.23: ESEM micrograph of an emulsion HPAM showing some ‘burn marks’ on the sample surface.

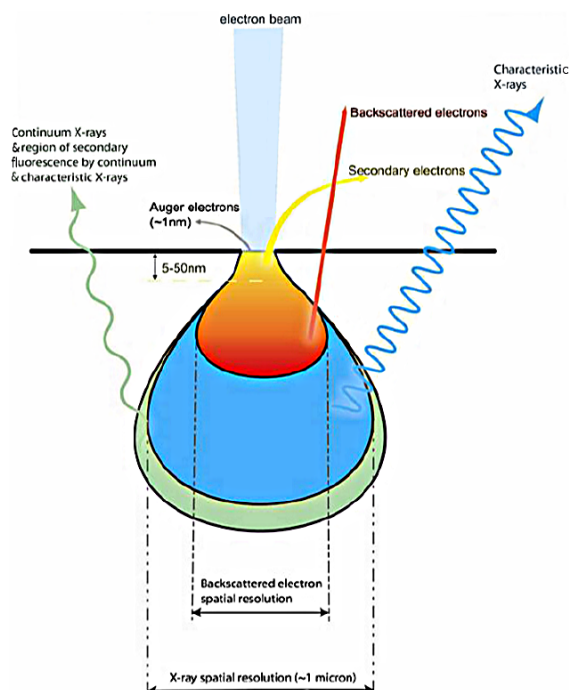


Figure 2.24: Electron-specimen interaction.

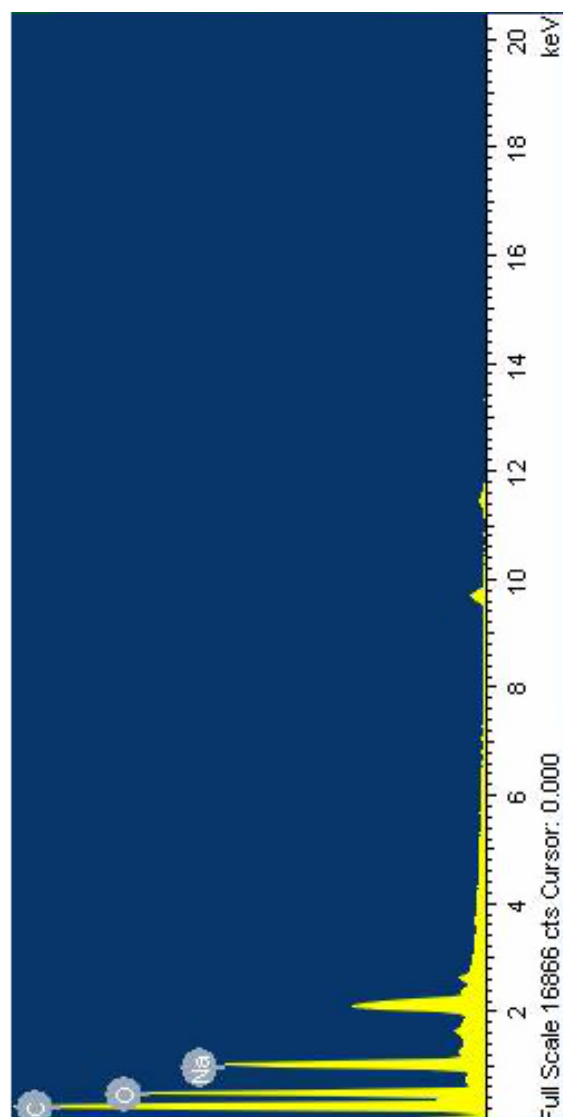
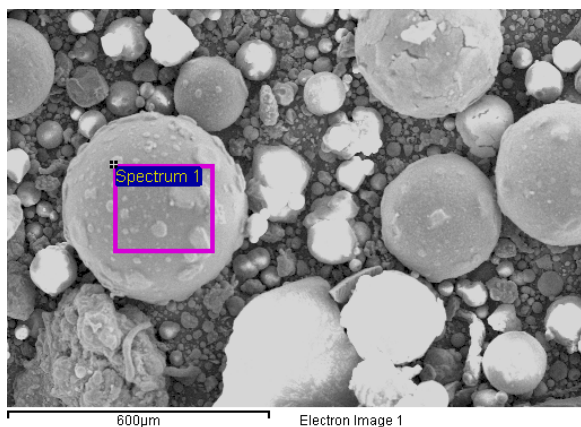
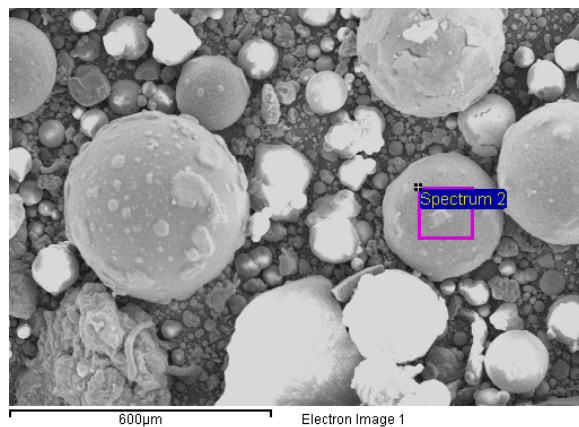


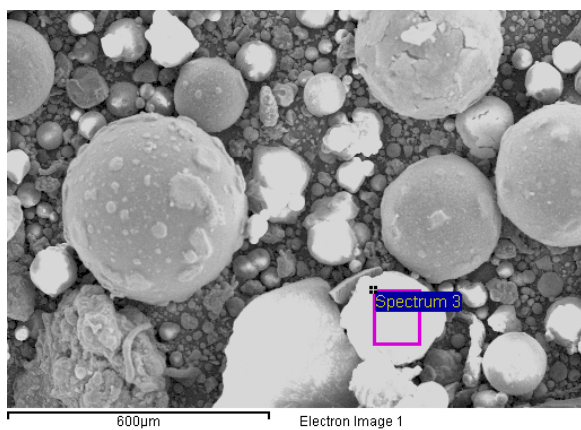
Figure 2.25: Typical X-ray spectrum of an HPAM by EDX.



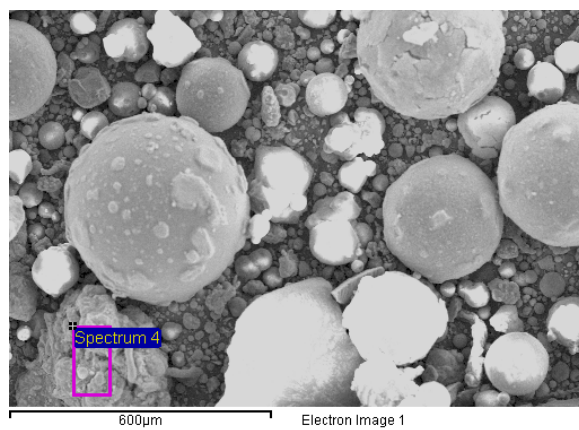
(a) Large bead



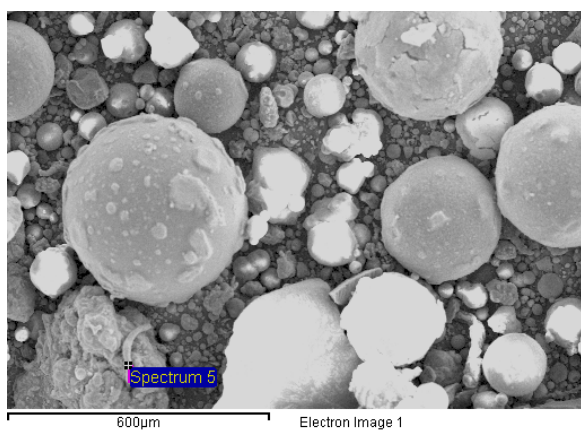
(b) Medium bead



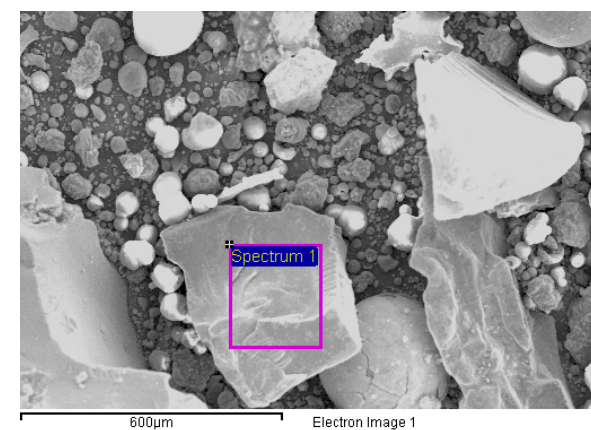
(c) Small bead



(d) Flaky particle

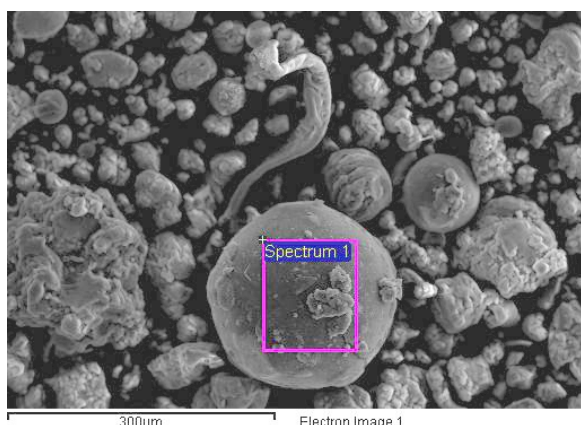


(e) Worm-like particle

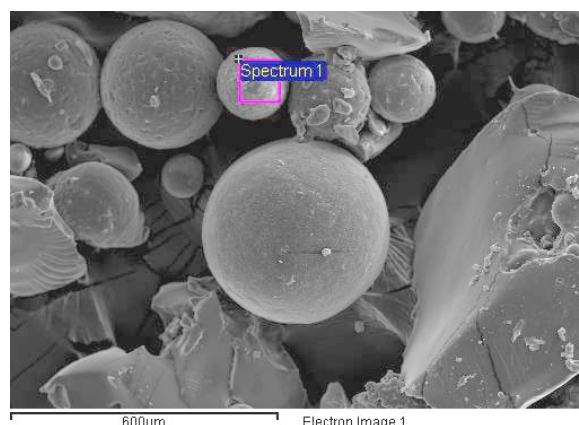


(f) HPAM-like granule

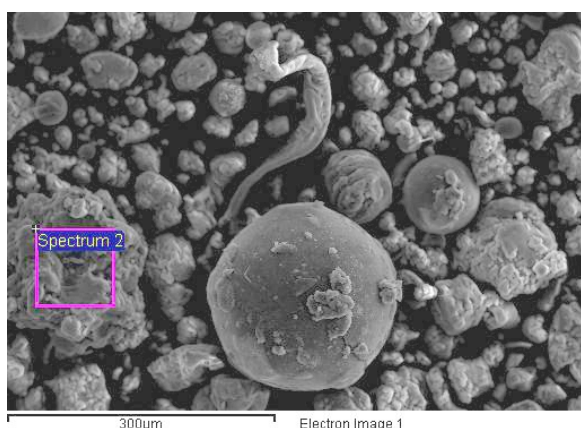
Figure 2.26: SEM micrographs of the blended polymer ‘SlurrySMART’ showing the areas selected for EDX analyses.



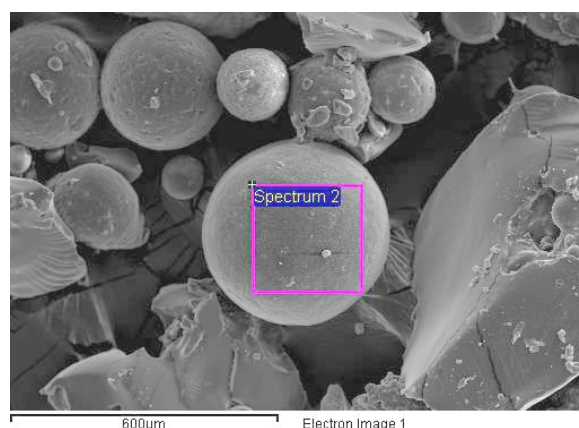
(a) Large bead



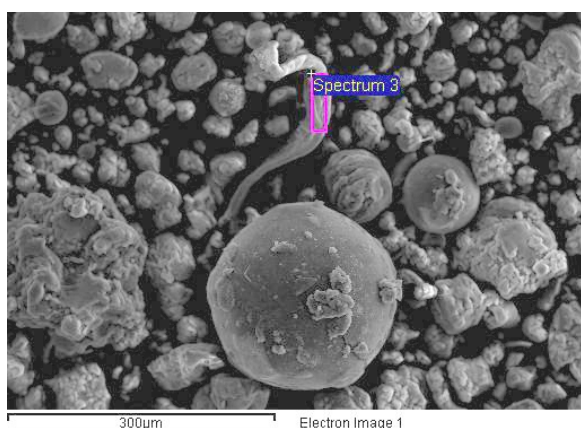
(a) Small bead



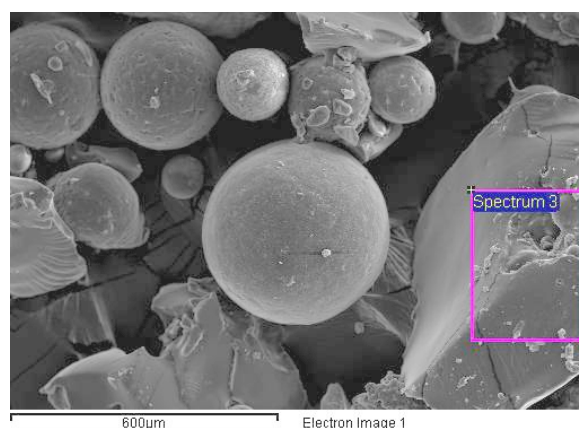
(b) Flaky particle



(b) Large bead



(c) worm-like particle



(c) HPAM-like granule

Figure 2.27: SEM micrographs of the blended additive 'EnhancIt 100' showing the areas selected for EDX analyses.

Figure 2.28: SEM micrographs of the blended additive 'EnhancIt 200' showing the areas selected for EDX analyses.

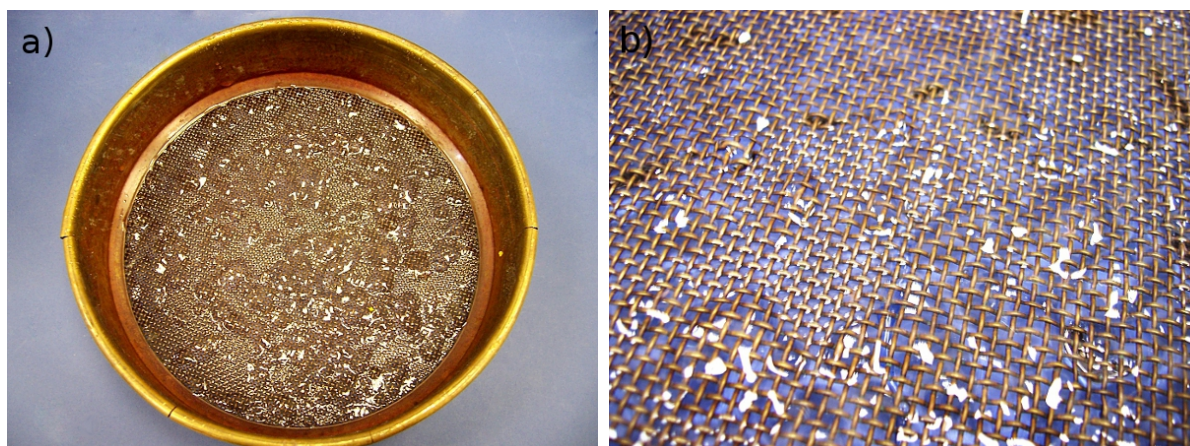


Figure 2.29: 'Fish eyes' left on a screen: *a)* medium shot; *b)* close-up.

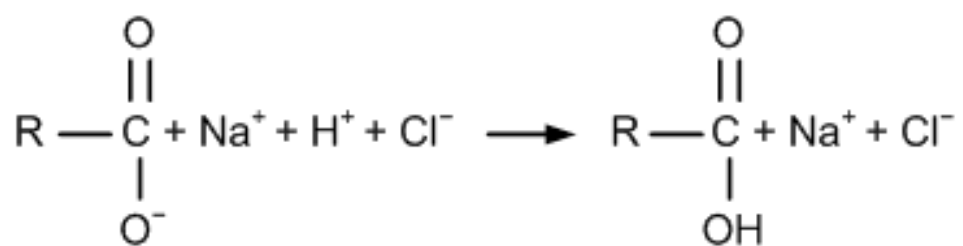


Figure 2.30: Reaction during titration of HPAM with HCl.



Figure 2.31: Colour change of the methyl-orange indigo-carmine indicator during a titration; from *left to right*: before, at, and after end point.

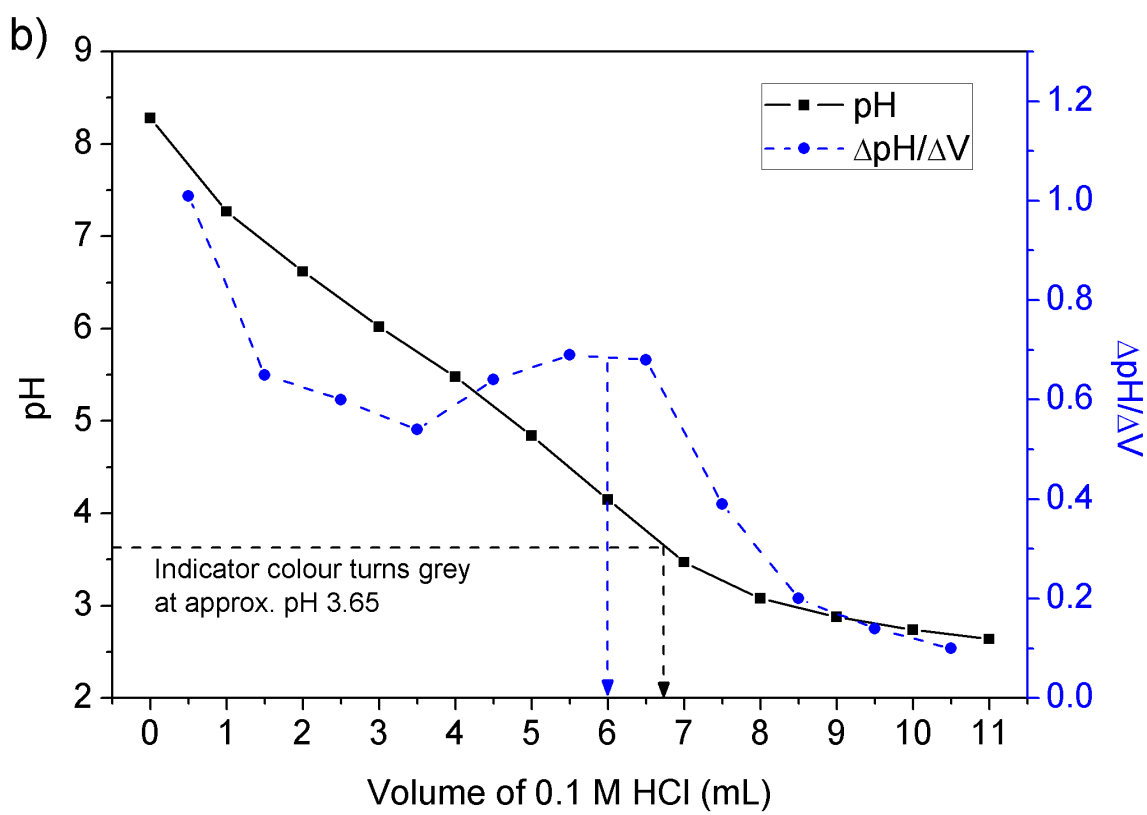
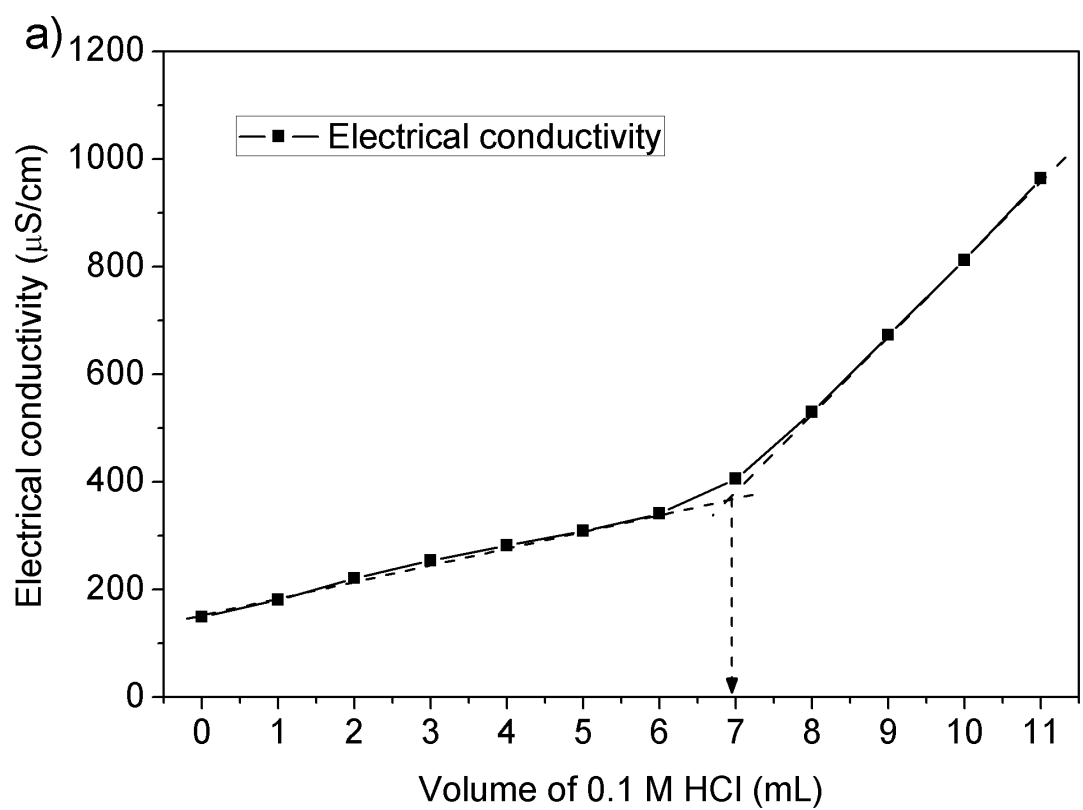


Figure 2.32: Titration curves of an HPAM: a) electrical conductivity; b) pH.

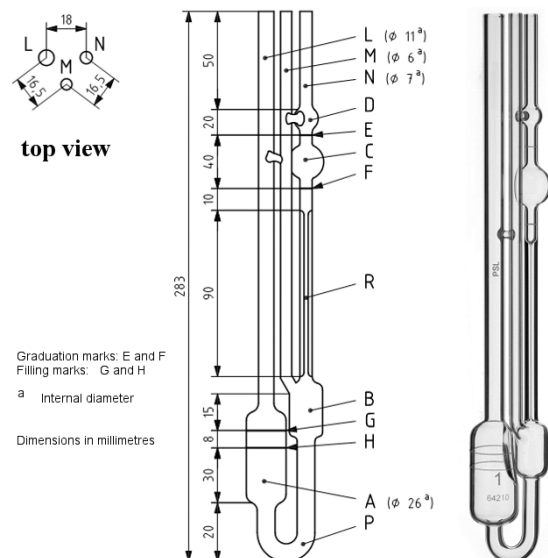


Figure 2.33: Ubbelohde viscometer. (From British Standards Institution, 2009b; Poulten Selfe & Lee Limited, 2010.)

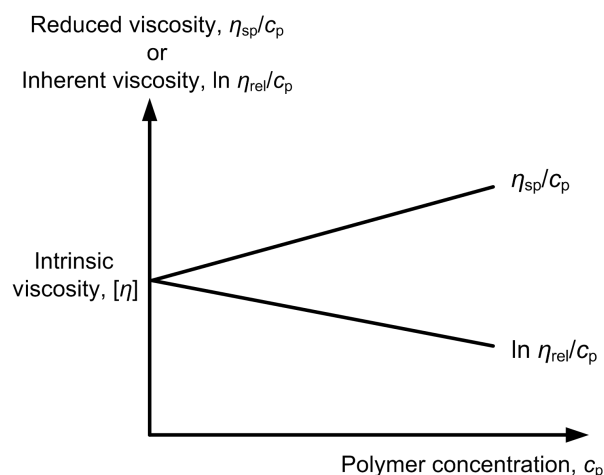


Figure 2.34: Determination of intrinsic viscosity from reduced and inherent viscosities.

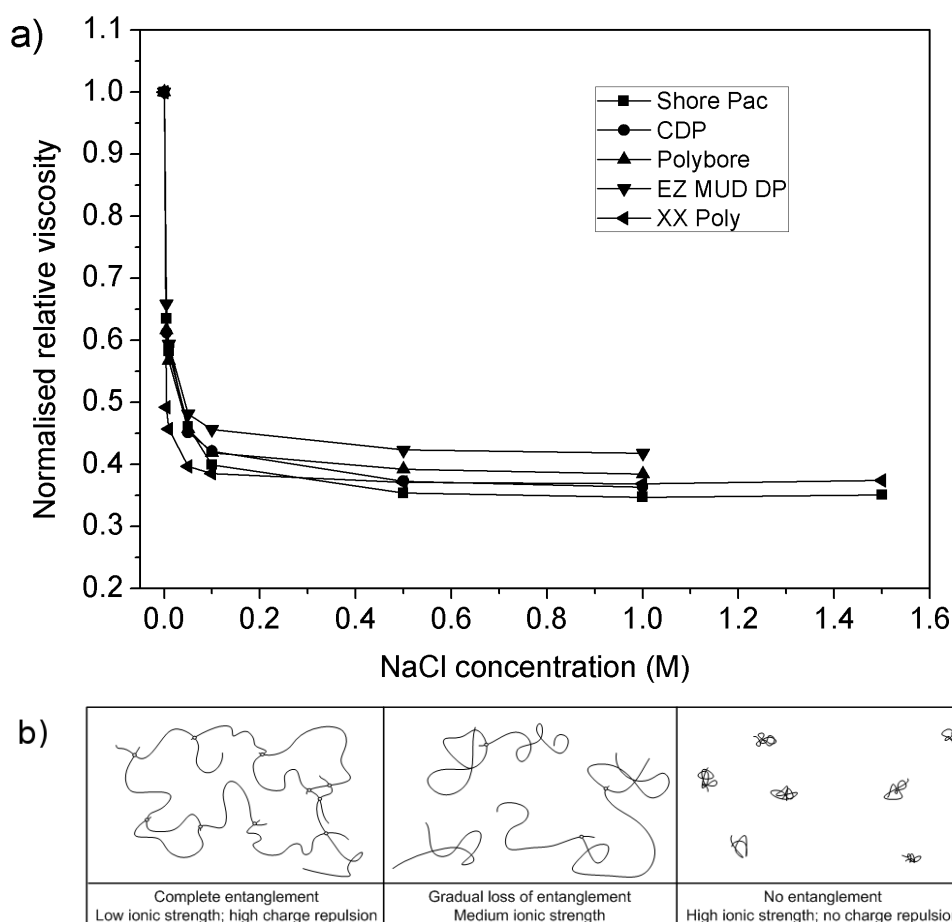


Figure 2.35: Effect of NaCl concentration on the viscosity of HPAM solutions: *a*) test results; *b*) schematic molecular structure.

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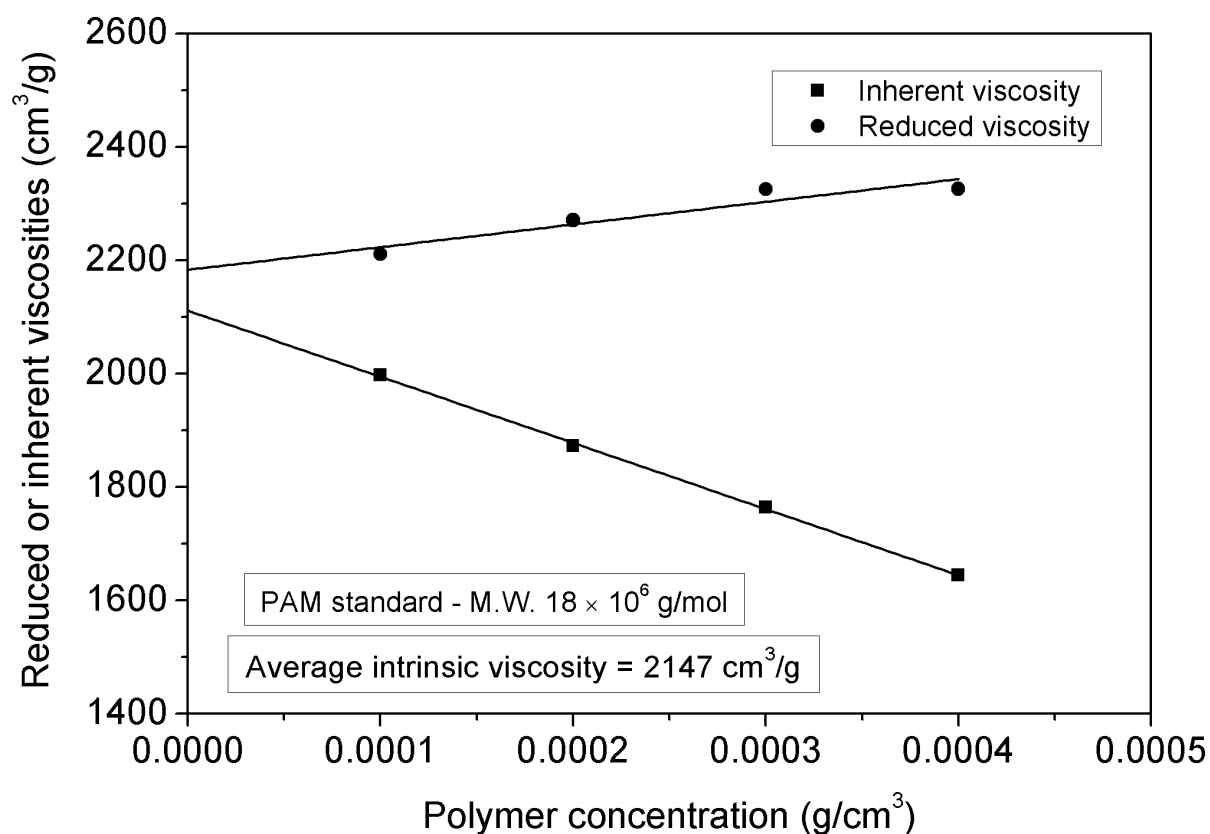


Figure 2.36: Determination of intrinsic viscosity of a HPAM standard with a known molecular weight of 18×10^6 g/mol.

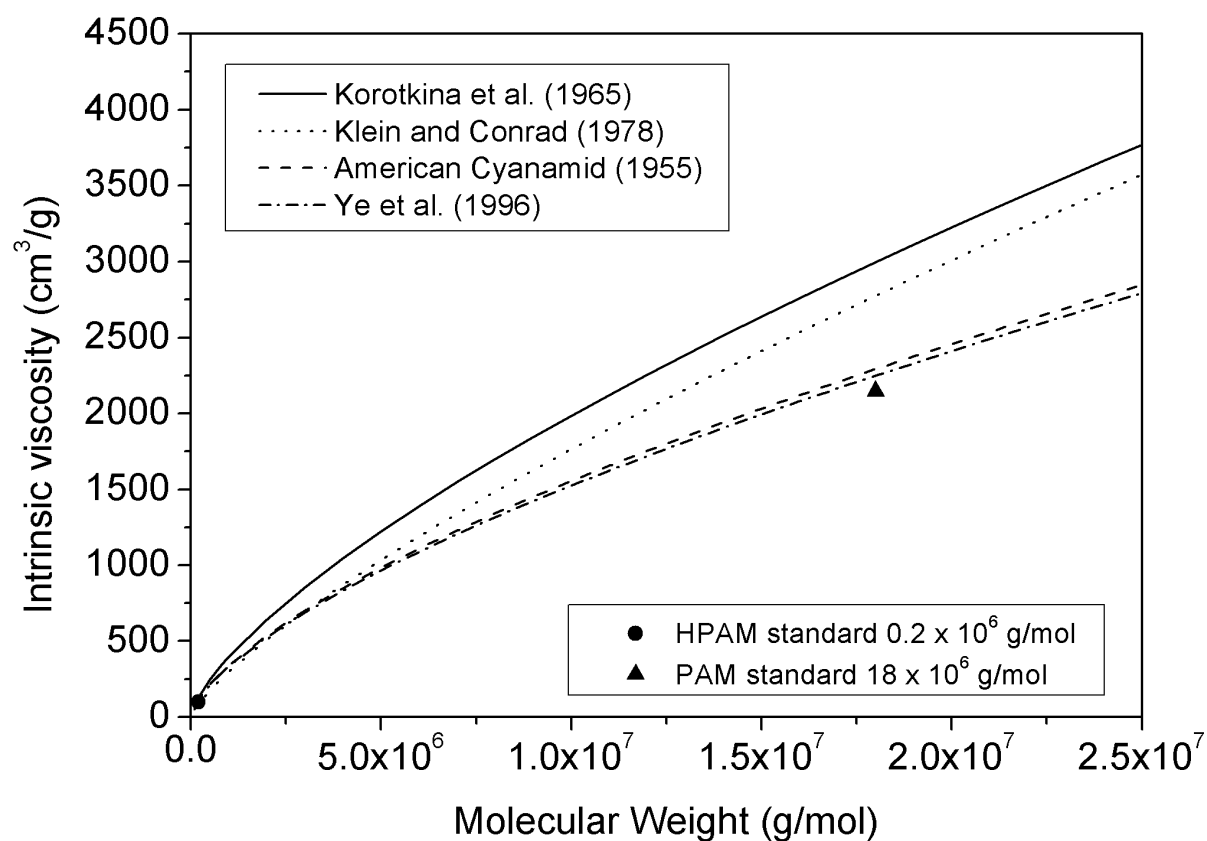


Figure 2.37: Mark-Houwink plot.

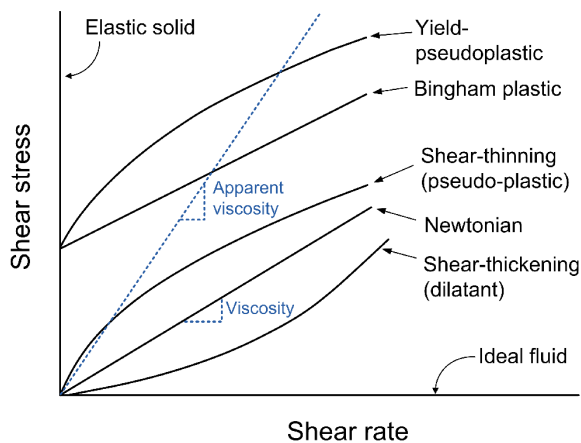


Figure 3.1: Different types of non-Newtonian flow behaviour.

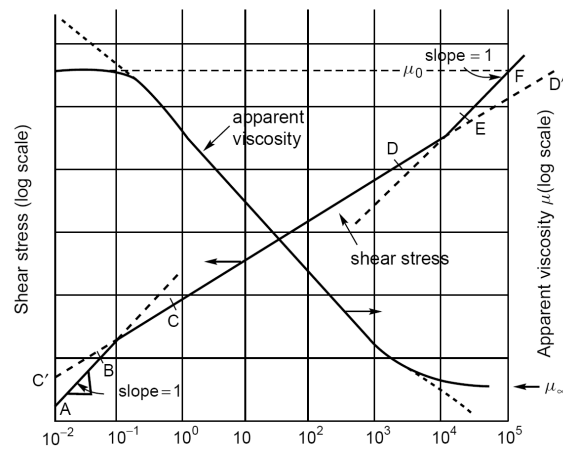


Figure 3.2: Shear-thinning behaviour. (From Chhabra & Richardson, 1999.)



Figure 3.3: Marsh funnel.



Figure 3.4: Fann viscometer.

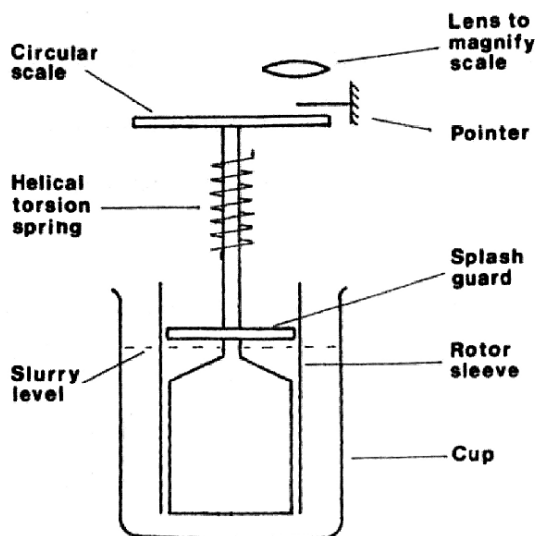


Figure 3.5: Schematic diagram of Fann viscometer. (From Jefferis, 1972.)

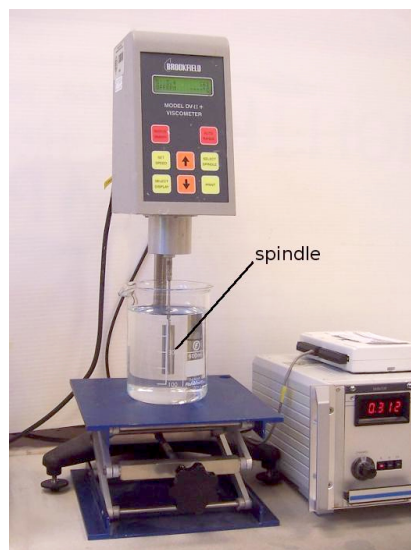


Figure 3.6: Brookfield viscometer with standard geometry.

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Figure 3.7: Brookfield viscometer with UL adaptor.



Figure 3.8: Bohlin Gemini rheometer.



Figure 3.9: Measuring element of the rheometer in contact with a polymer fluid sample.

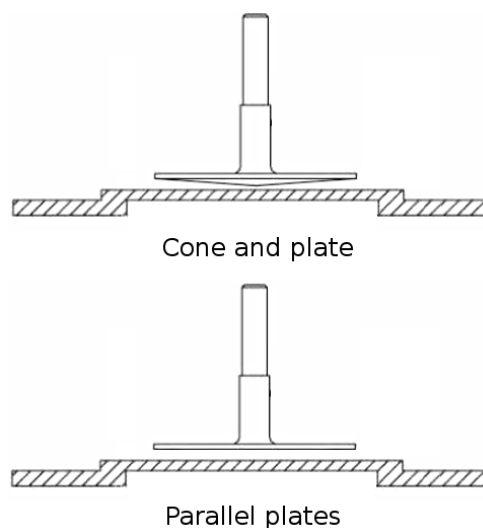


Figure 3.10: Rheometer measurement geometries

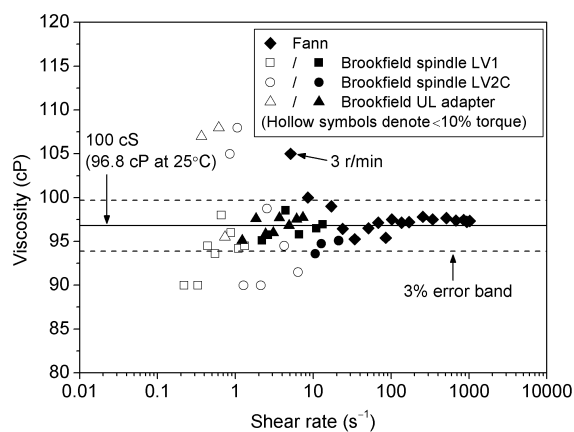


Figure 3.11: Fann and Brookfield viscometer checks using 100-cS silicone fluid.

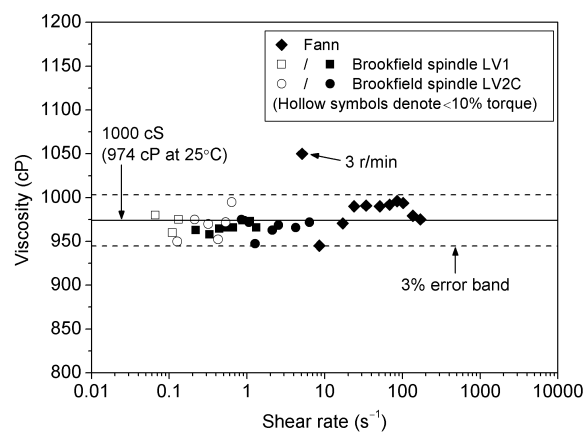


Figure 3.12: Fann and Brookfield viscometer checks using 1000-cS silicone fluid.



Figure 3.13: Lightnin mixer.

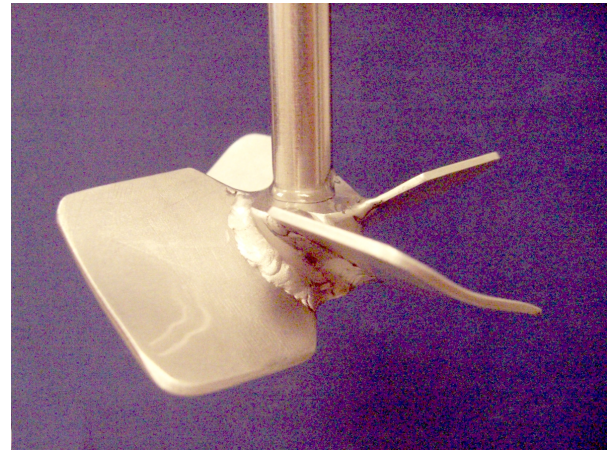


Figure 3.14: Low-shear impeller.

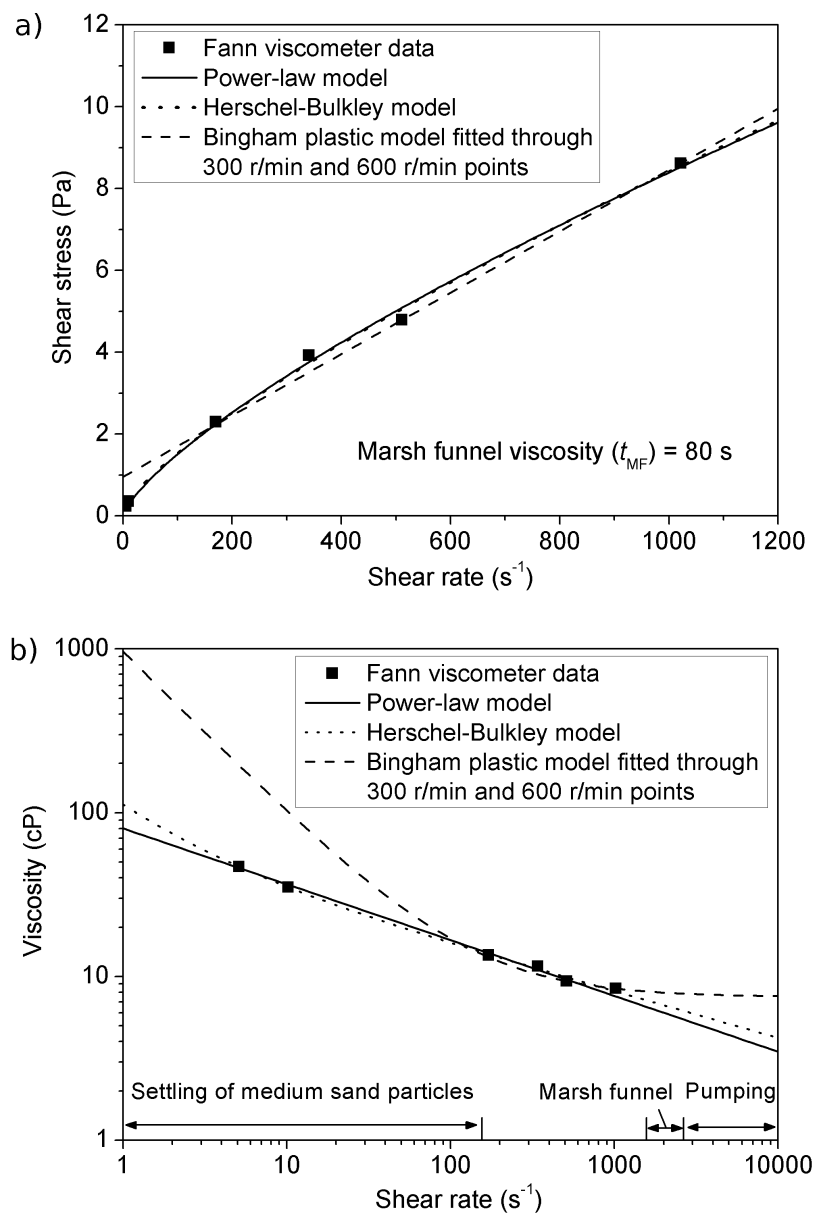


Figure 3.15: Test data of a clean HPAM fluid measured by Fann viscometer: *a*) shear stress versus shear rate on linear scale; *b*) viscosity versus shear rate on logarithmic scale.

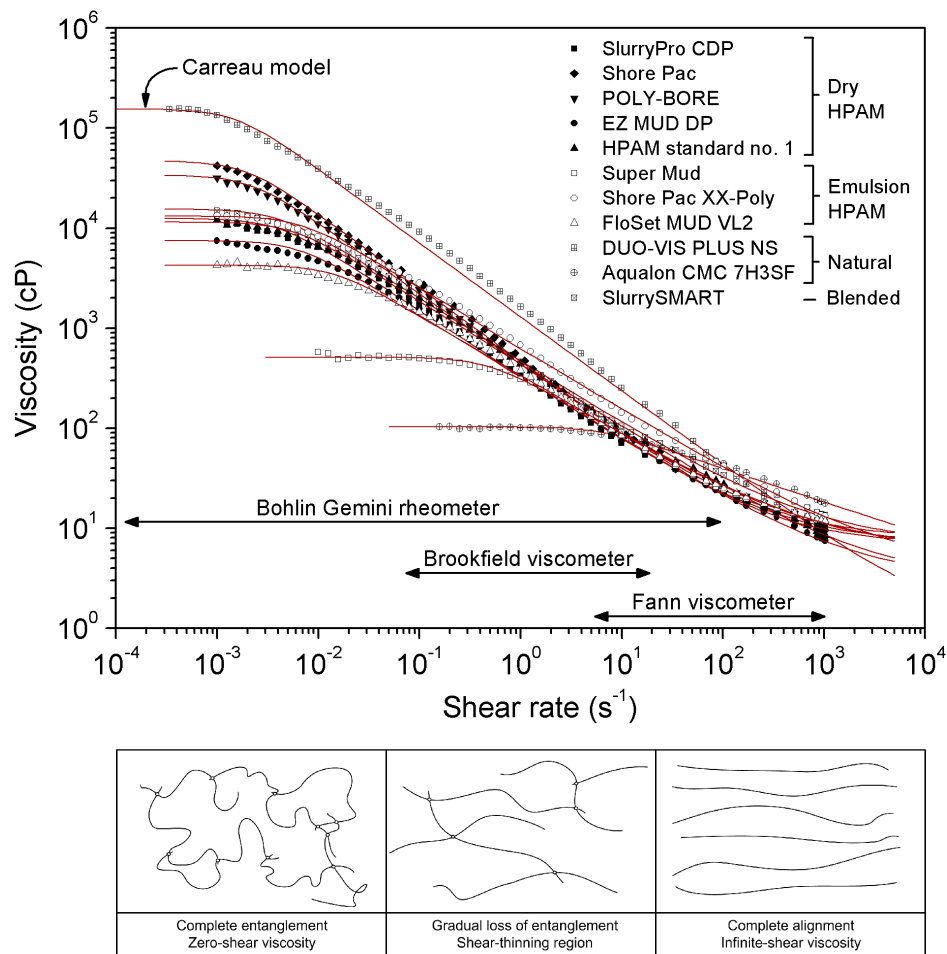


Figure 3.16: Flow curves of different types of polymer products and schematic illustrations of molecular arrangement – the flow curve data was obtained using the Bohlin rheometer and the Fann viscometer.

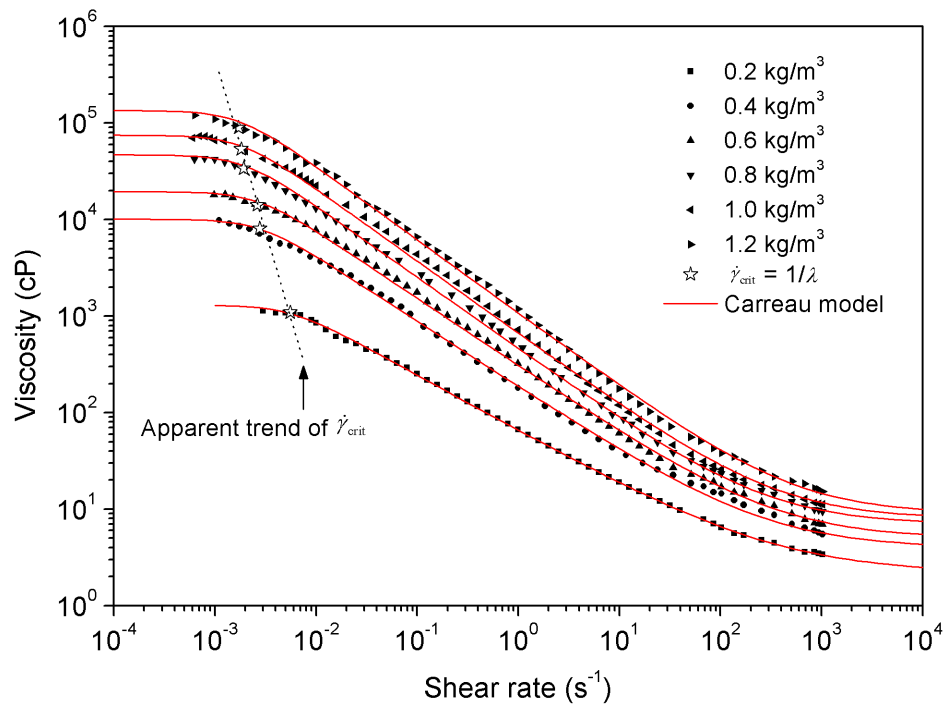


Figure 3.17: Flow curves of a commercial HPAM fluid.

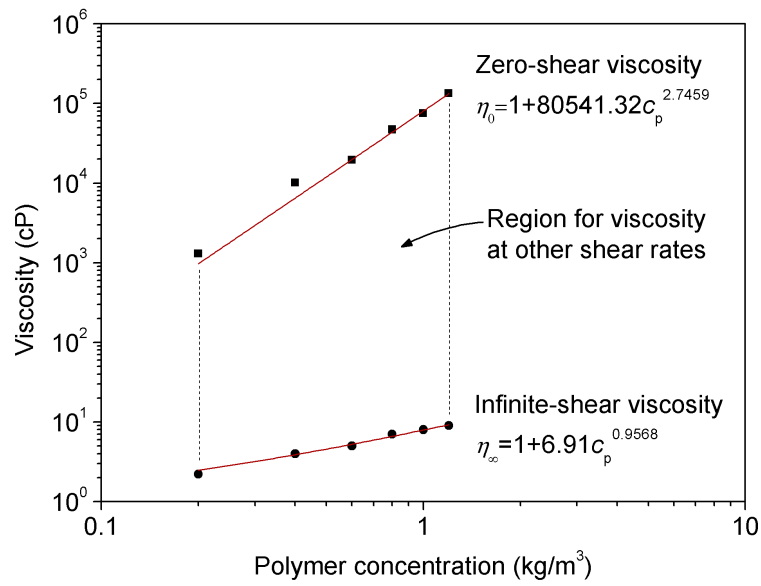


Figure 3.18: Concentration dependence of infinite and zero-shear viscosities.

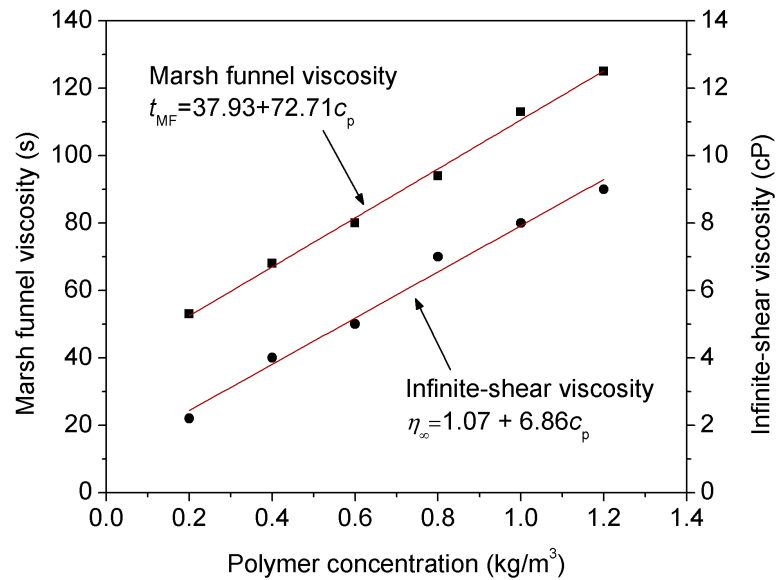


Figure 3.19: Concentration dependence of Marsh funnel viscosity.



Figure 3.20: Three double-diaphragm pumps used in parallel. (Photograph courtesy of G. Goodhue.)



Figure 3.21: Centrifugal pump.

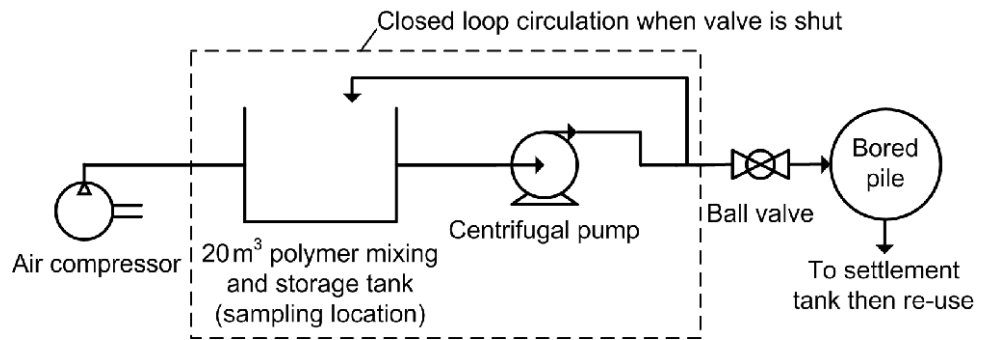


Figure 3.22: Schematic plant setup for a site in Glasgow.



Figure 3.23: Freshly mixed polymer fluids by aeration.



Figure 3.24: Fluid sampler.

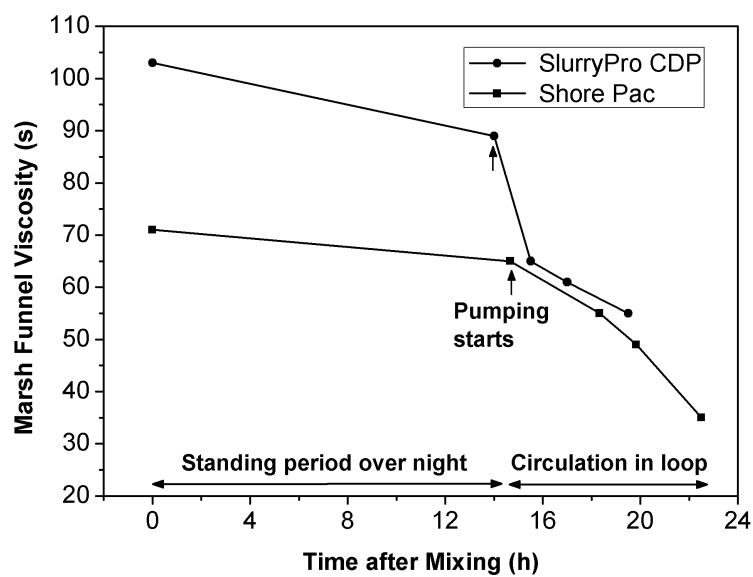


Figure 3.25: Change in Marsh funnel viscosity of two clean HPAM fluids in a mixing tank.

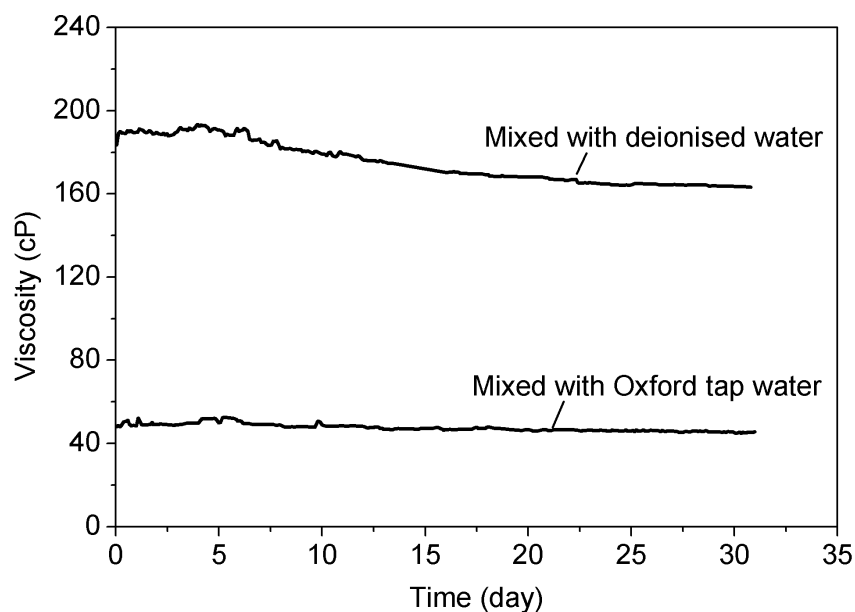


Figure 3.26: Ageing test results of a commercial HPAM.

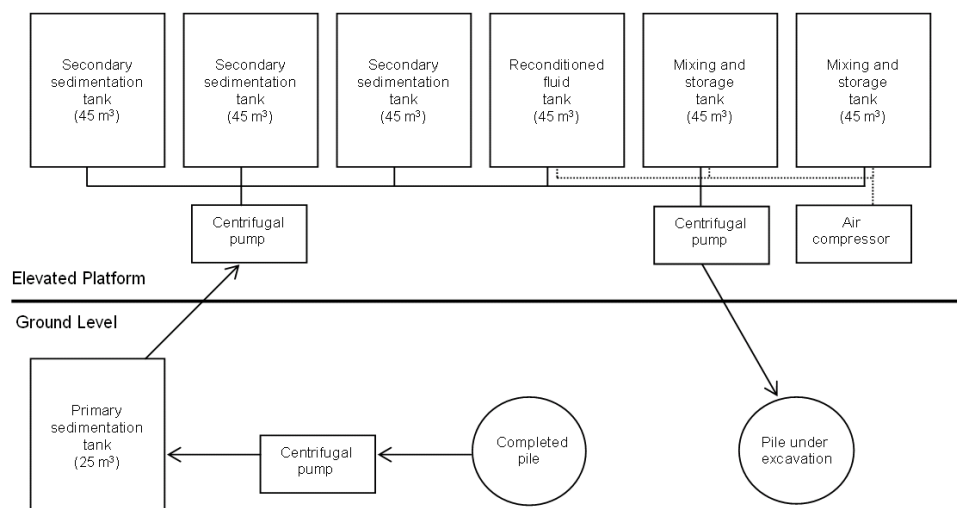


Figure 3.27: Schematic tank layout at the Castle House site.



Figure 3.28: Photograph of tank layout at the Castle House site.



Figure 3.29: SlurryPro CDP polymer fluids after repeated use at the Castle House site.

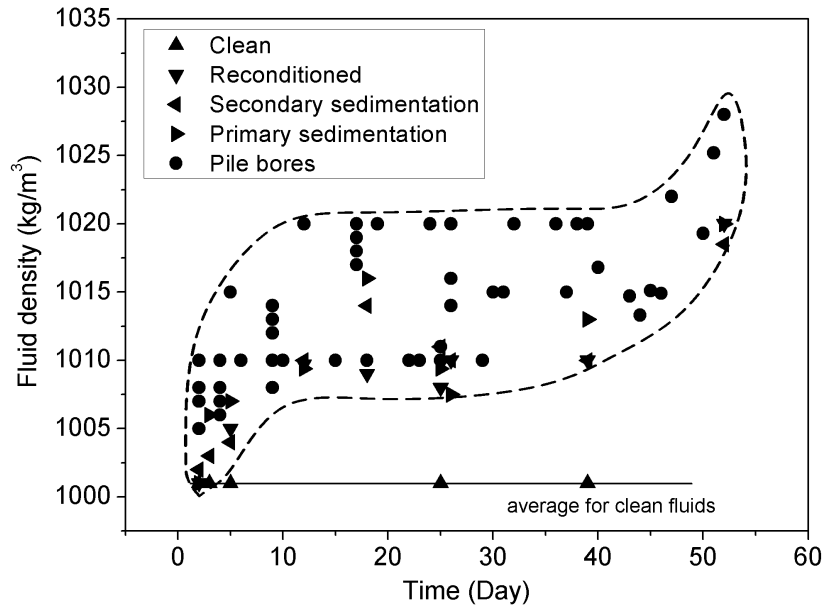


Figure 3.30: Density of polymer fluids during bored pile construction at the Castle House site.

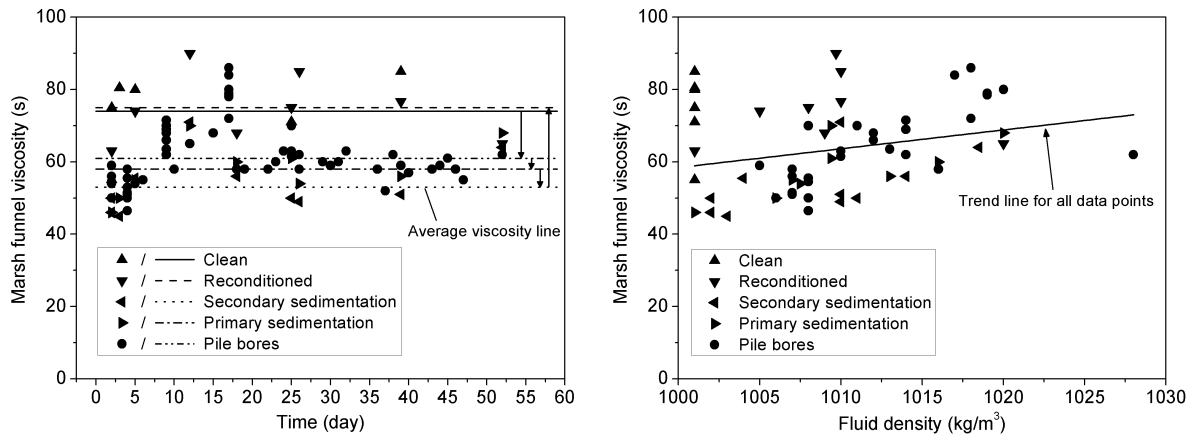


Figure 3.31: Marsh funnel viscosity of polymer fluids at the Castle House site: *a*) plotted against time; *b*) plotted against density.

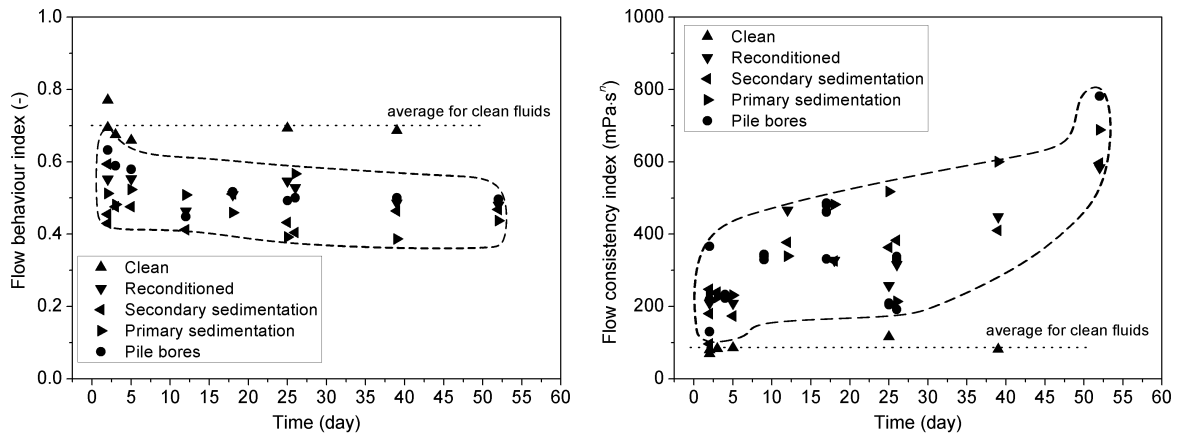


Figure 3.32: Power-law parameters of polymer fluids at the Castle House site, plotted against time: *a*) flow behaviour index; *b*) flow consistency index.

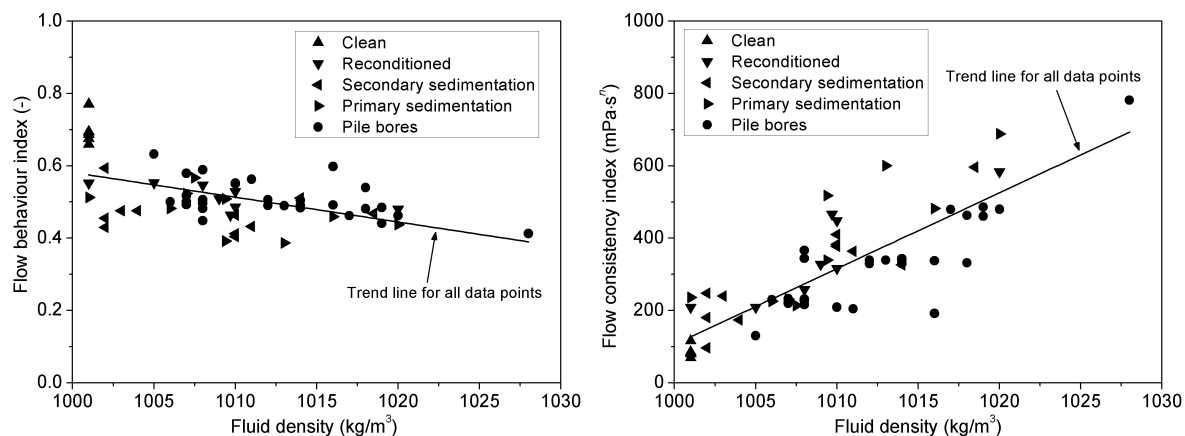


Figure 3.33: Power-law parameters of polymer fluids at the Castle House site, plotted against density: a) flow behaviour index; b) flow consistency index.

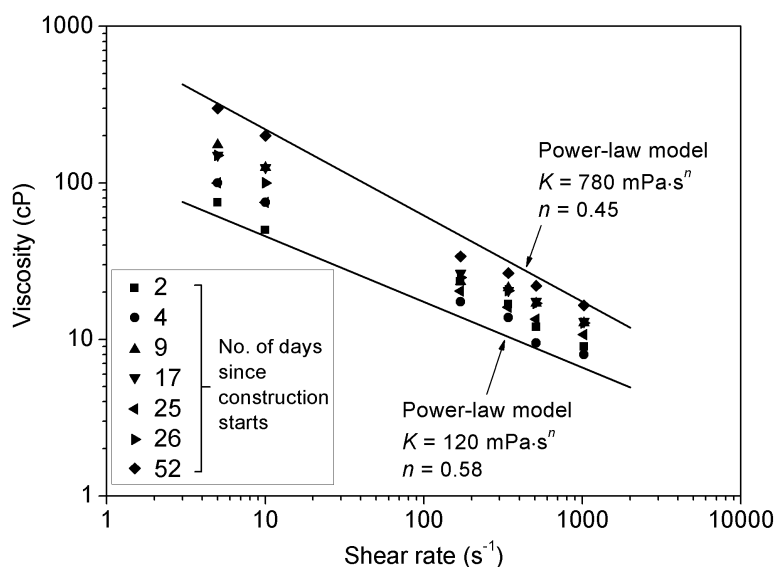


Figure 3.34: Selected flow curves of polymer fluids taken from the pile bores at the Castle House site.

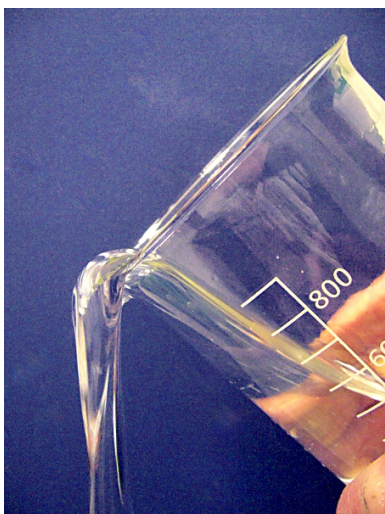


Figure 3.35: Self-siphoning effect shown by a high-molecular-weight HPAM fluid.

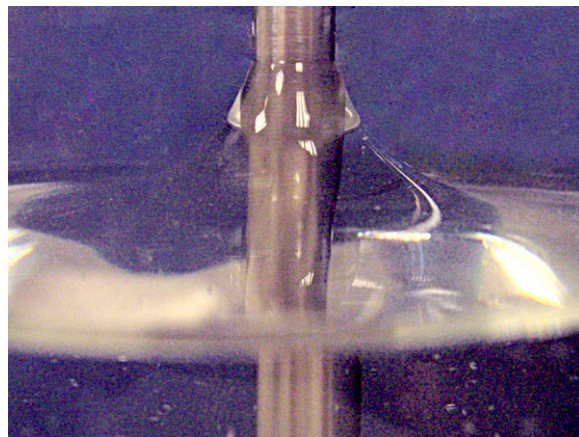


Figure 3.36: Weissenberg effect shown by a high-molecular-weight HPAM fluid.

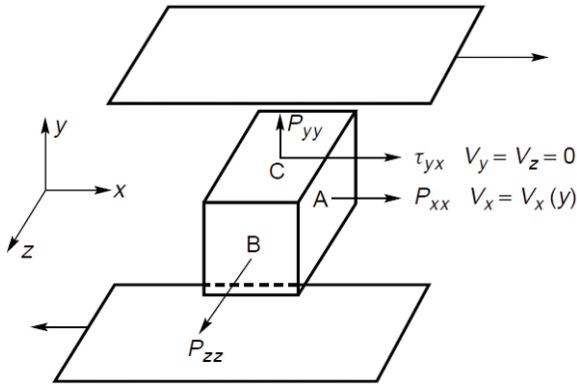


Figure 3.37: Non-zero stress components in one dimensional steady shearing motion of a viscoelastic fluid. (From Chhabra & Richardson, 1999.)

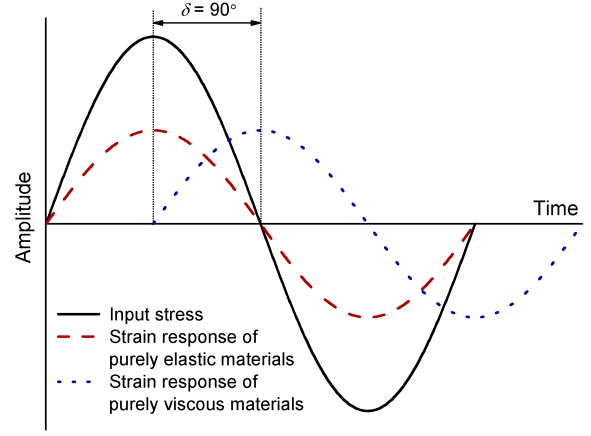


Figure 3.38: Strain response of purely elastic and purely viscous materials when subjected to oscillatory shear.

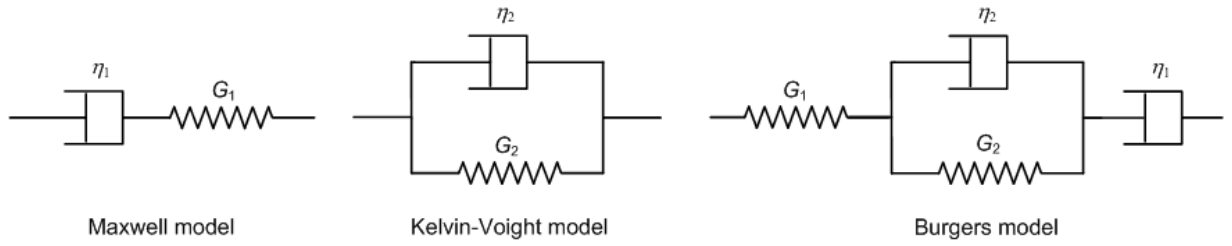


Figure 3.39: Maxwell, Kelvin-Voigt, and the four-element Burgers models.

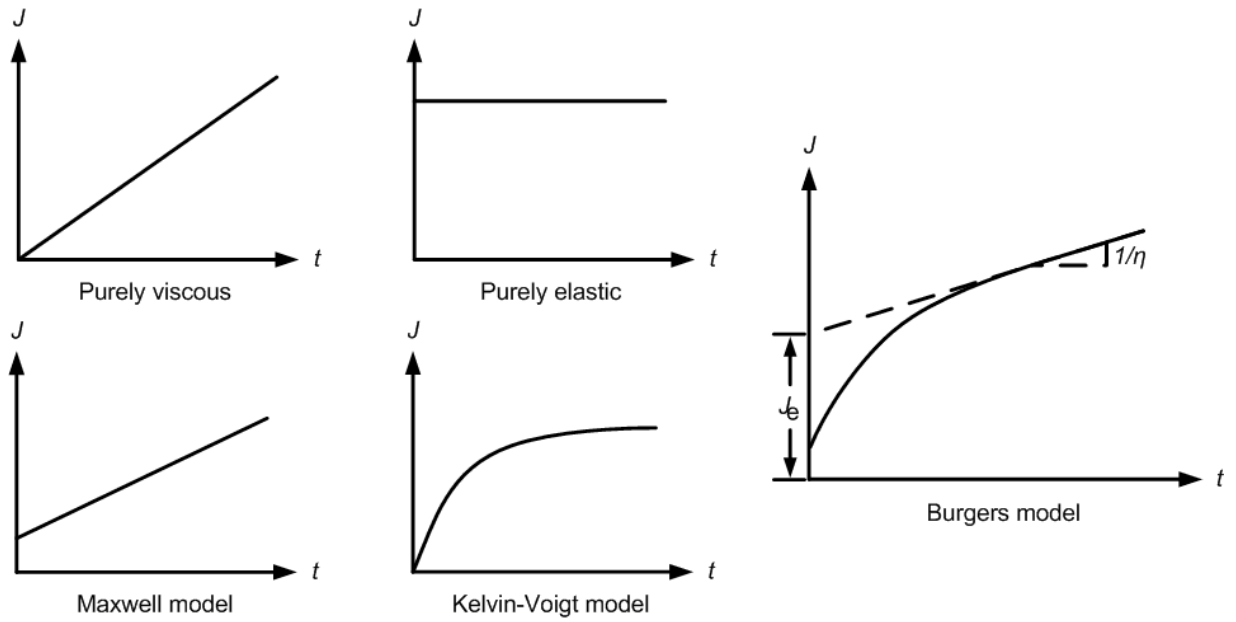


Figure 3.40: Schematic creep compliance curves of different material models.

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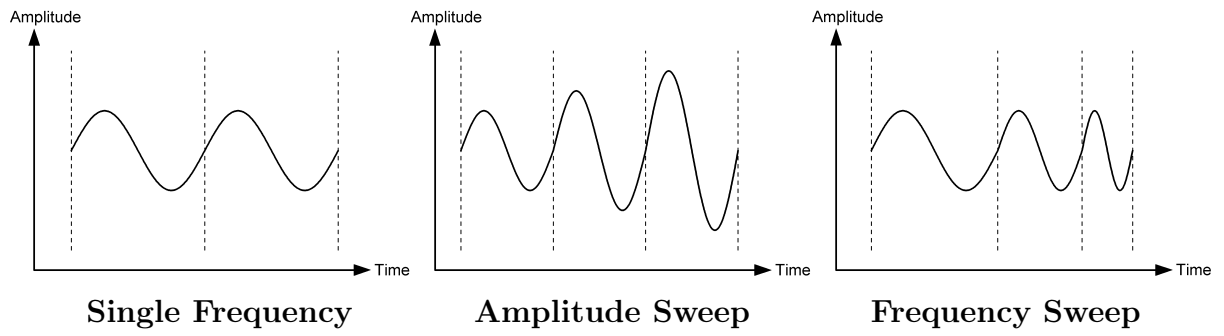


Figure 3.41: Oscillatory test types.

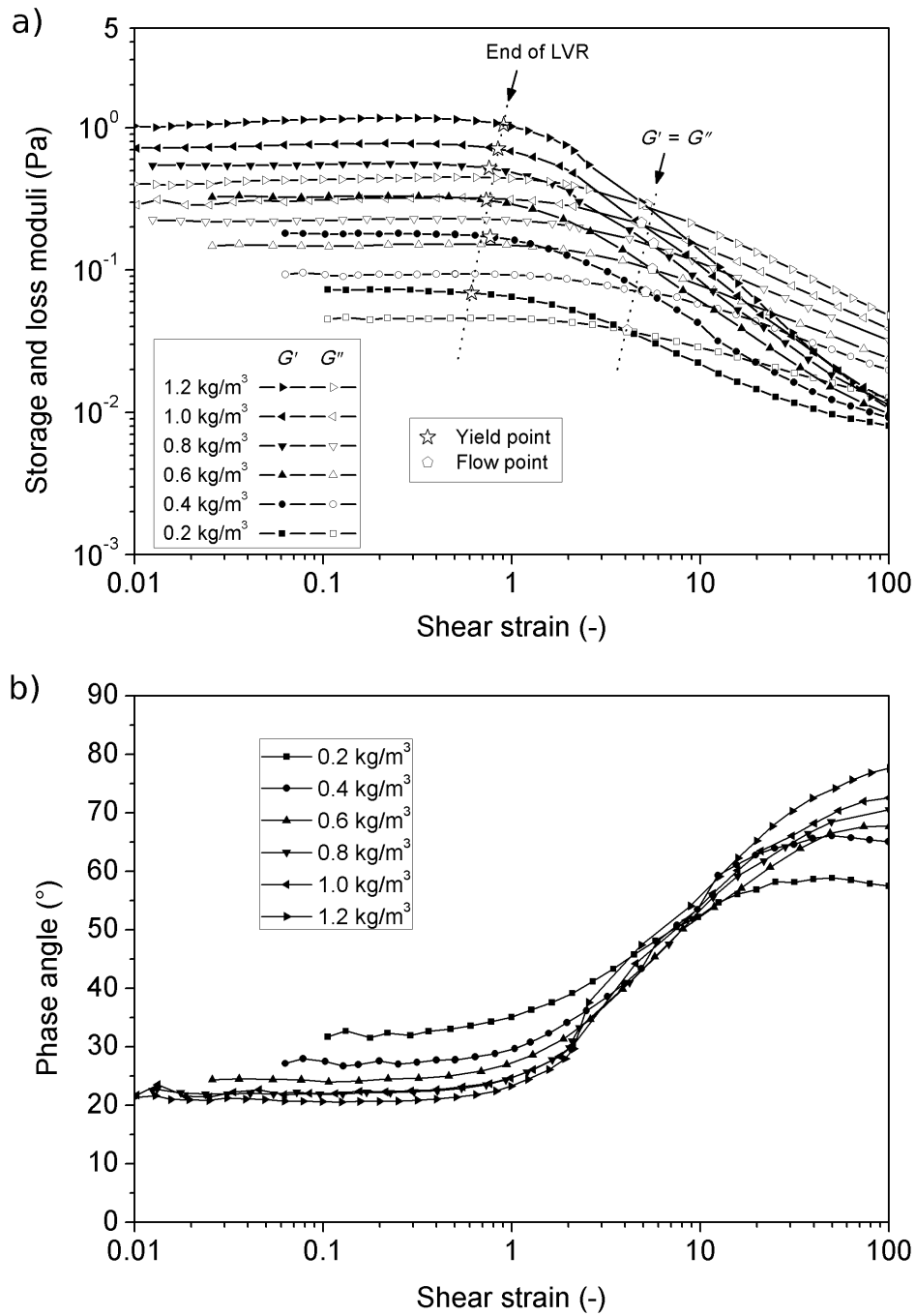


Figure 3.42: Amplitude sweep test results of a commercial HPAM fluid; *a*) storage and loss moduli; *b*) phase angle.

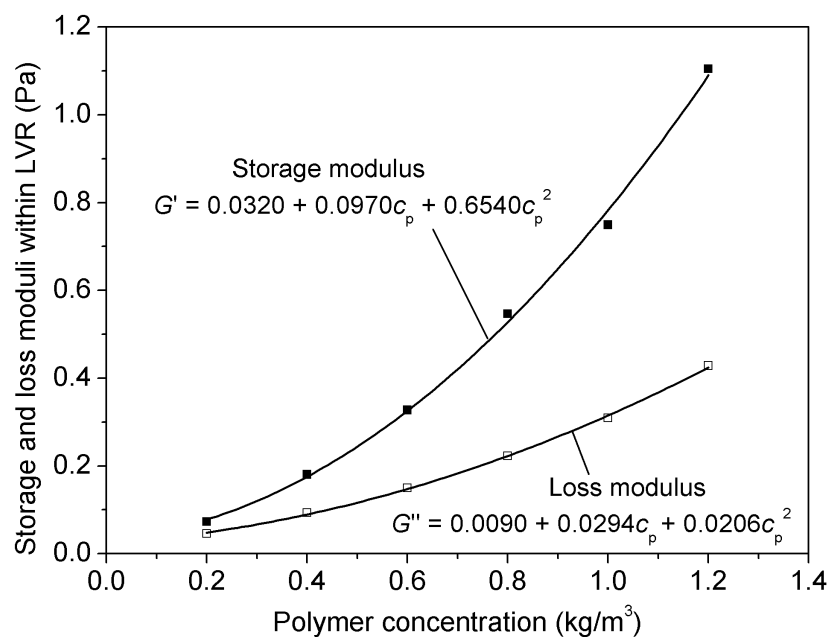


Figure 3.43: Storage and loss moduli within LVR against polymer concentration.

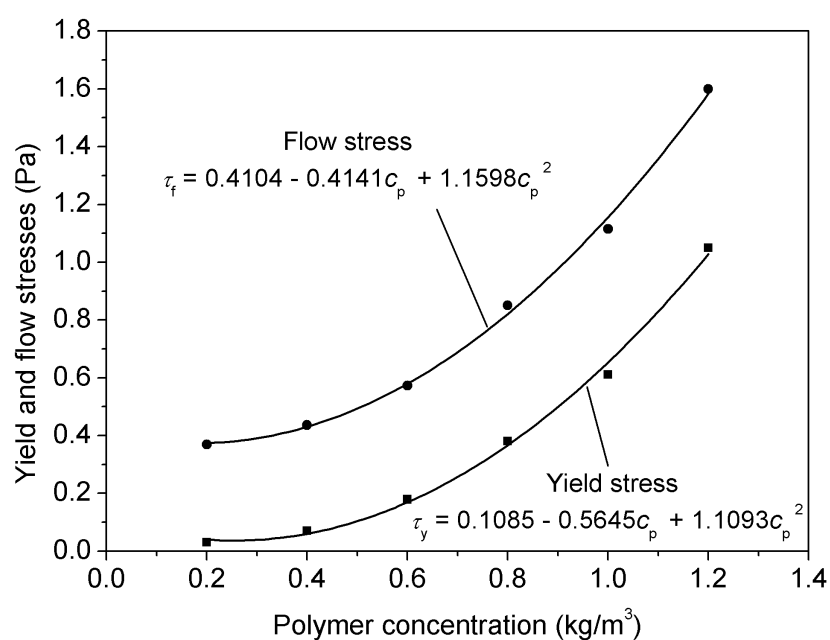


Figure 3.44: Yield and flow stresses against polymer concentration.

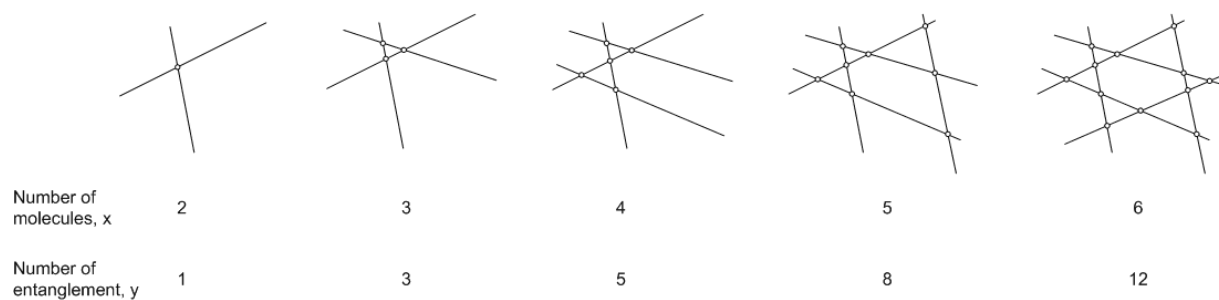


Figure 3.45: Schematic diagram showing the non-linear relationship between polymer concentration and chain entanglement.

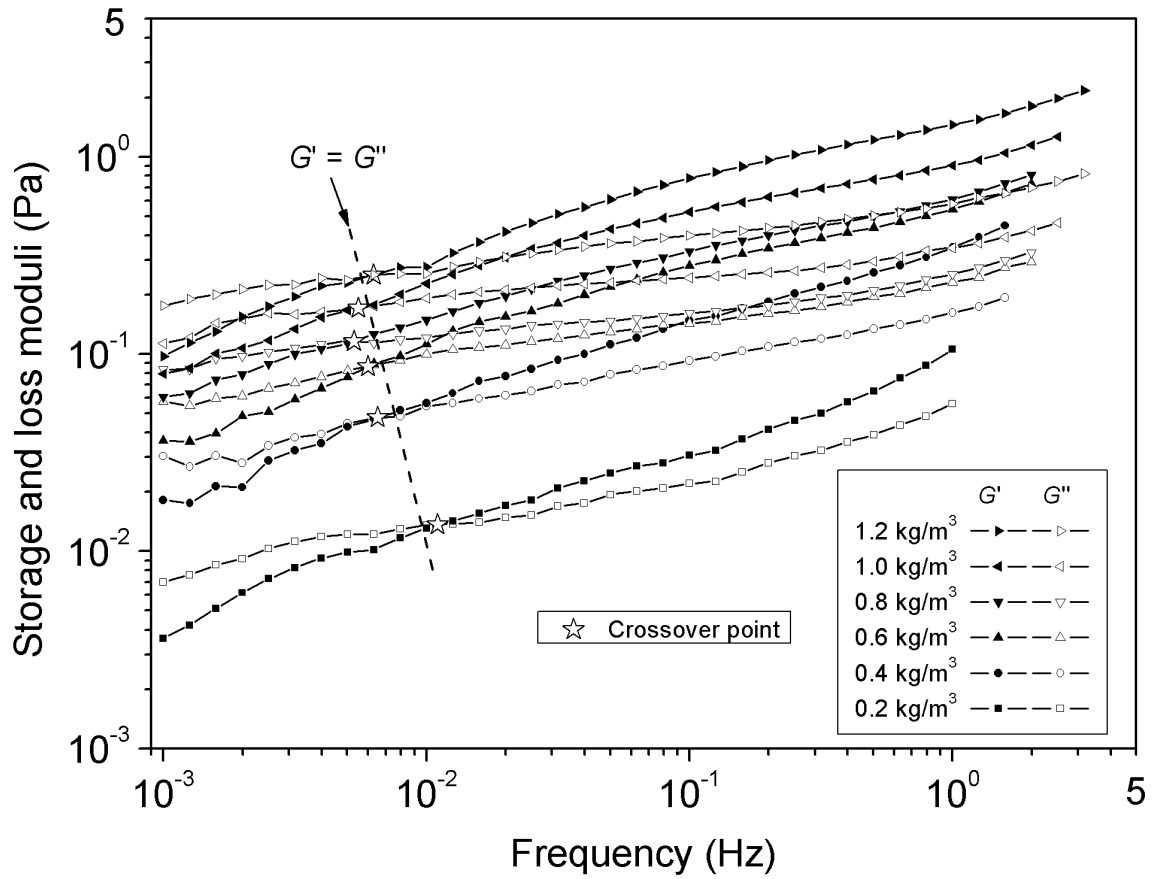


Figure 3.46: Frequency sweep test results of the commercial HPAM fluid.

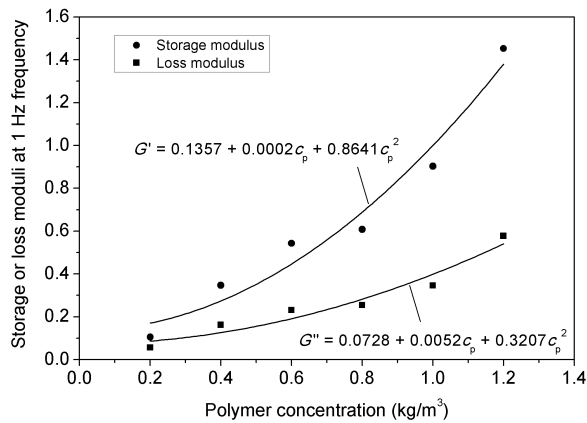


Figure 3.47: Storage and loss moduli at 1 Hz frequency

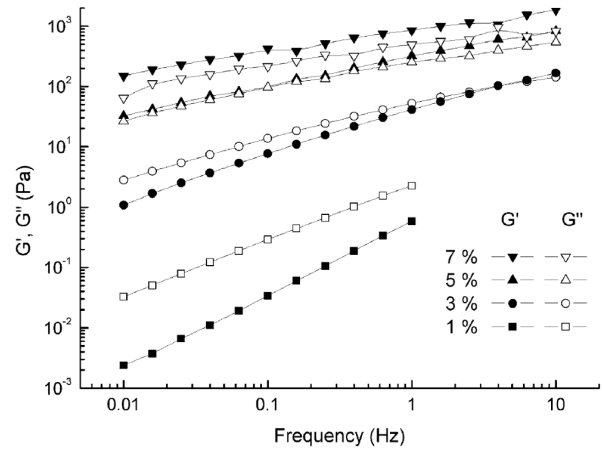


Figure 3.48: Frequency sweep test results of a carboxymethyl cellulose (CMC) fluid (From Benchabane & Bekkour, 2008)

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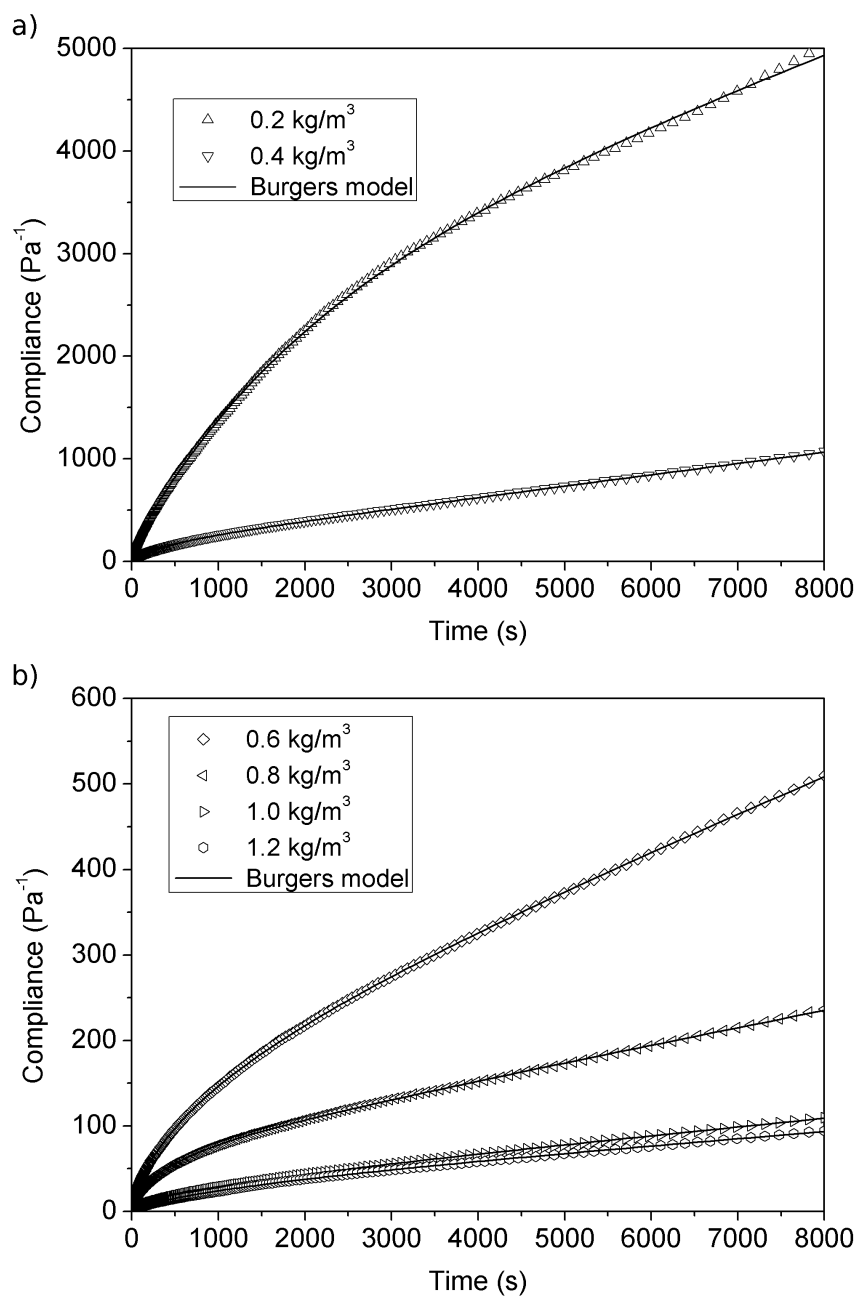


Figure 3.49: Creep test results of a high-molecular-weight HPAM fluid: *a)* 0.2 and 0.4 kg/m³; *b)* 0.6 to 1.2 kg/m³

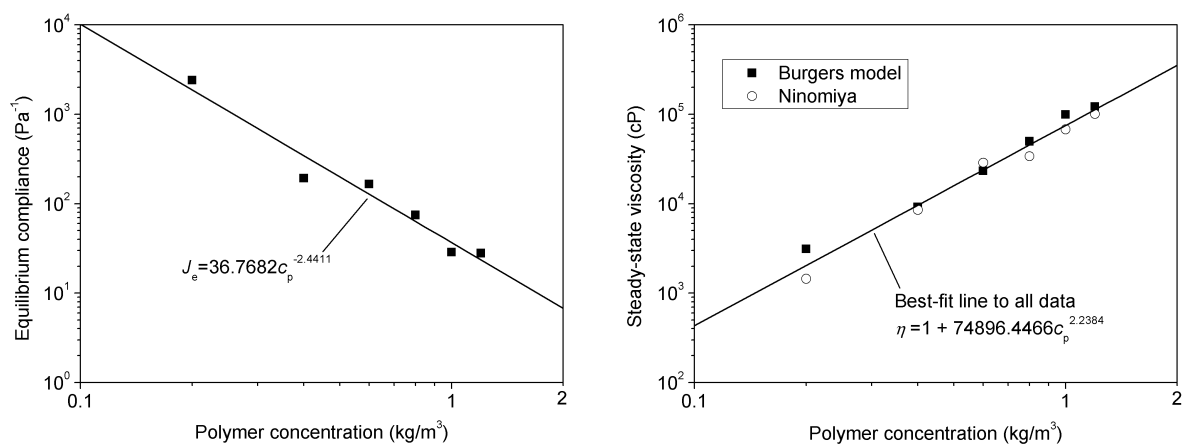


Figure 3.50: Equilibrium compliance from creep tests.

Figure 3.51: Steady-state viscosity from creep tests.

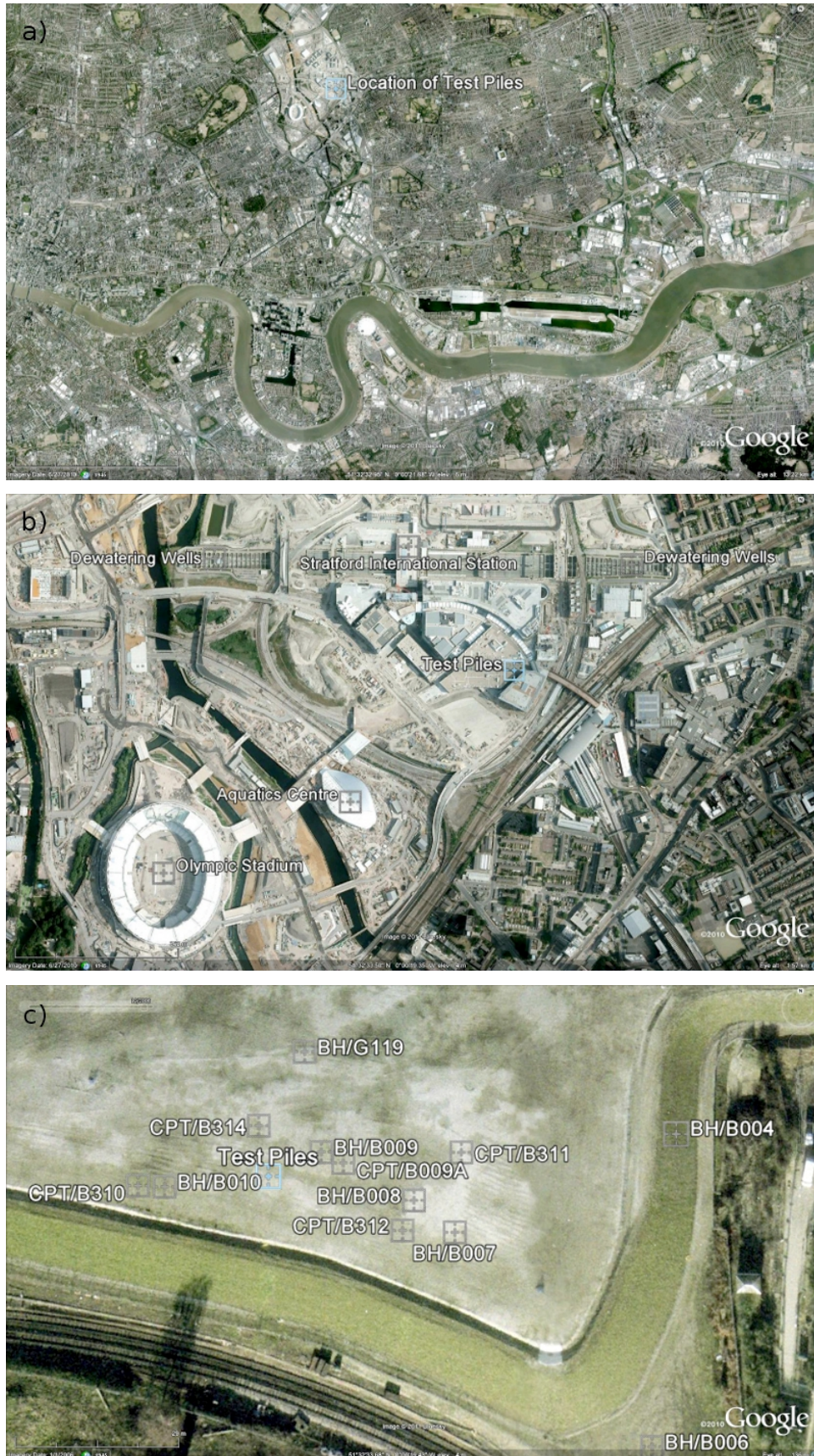


Figure 4.1: Location of test piles: *a*) overview; *b*) in relation to other key structures in the area; *c*) in relation to the site investigation boreholes. (Images from Google Earth. Note that an aerial photo taken in year 2006 was used as the background of the bottom image to show the locations of the exploratory holes more clearly.)

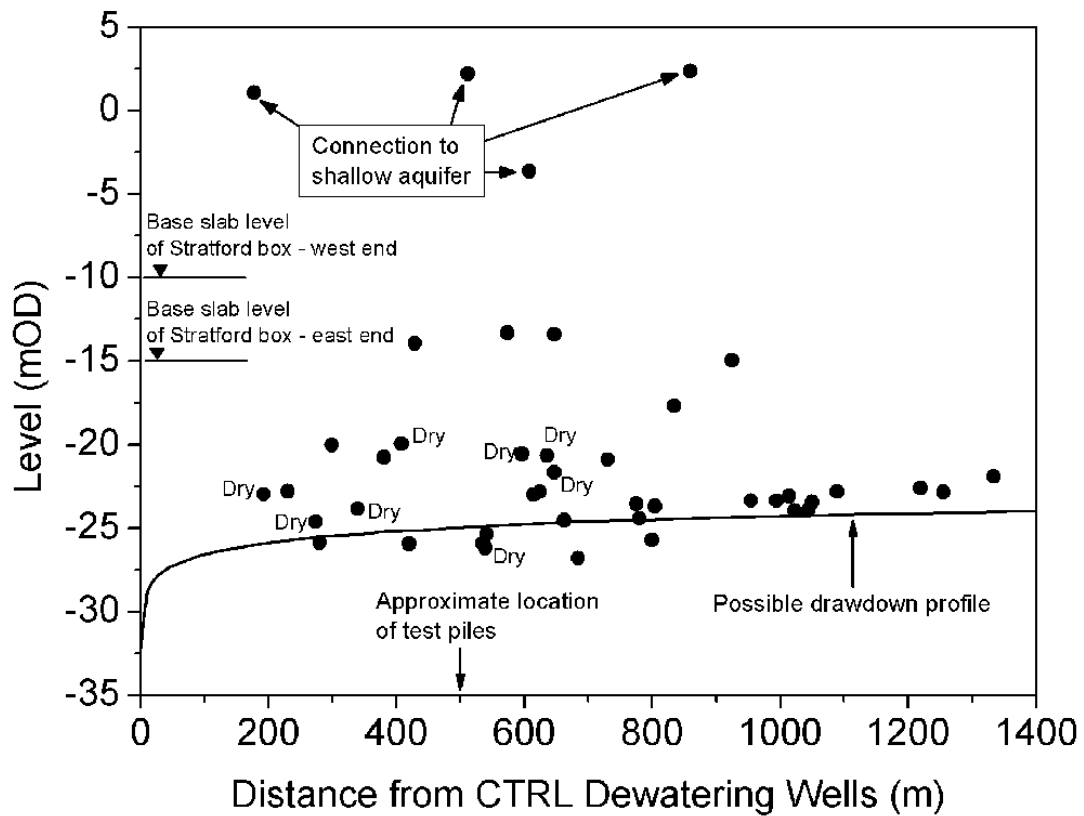


Figure 4.2: Groundwater level in deep aquifer in the vicinity of Stratford box. (Modified after Ove Arup & Partners Ltd, 2008.)

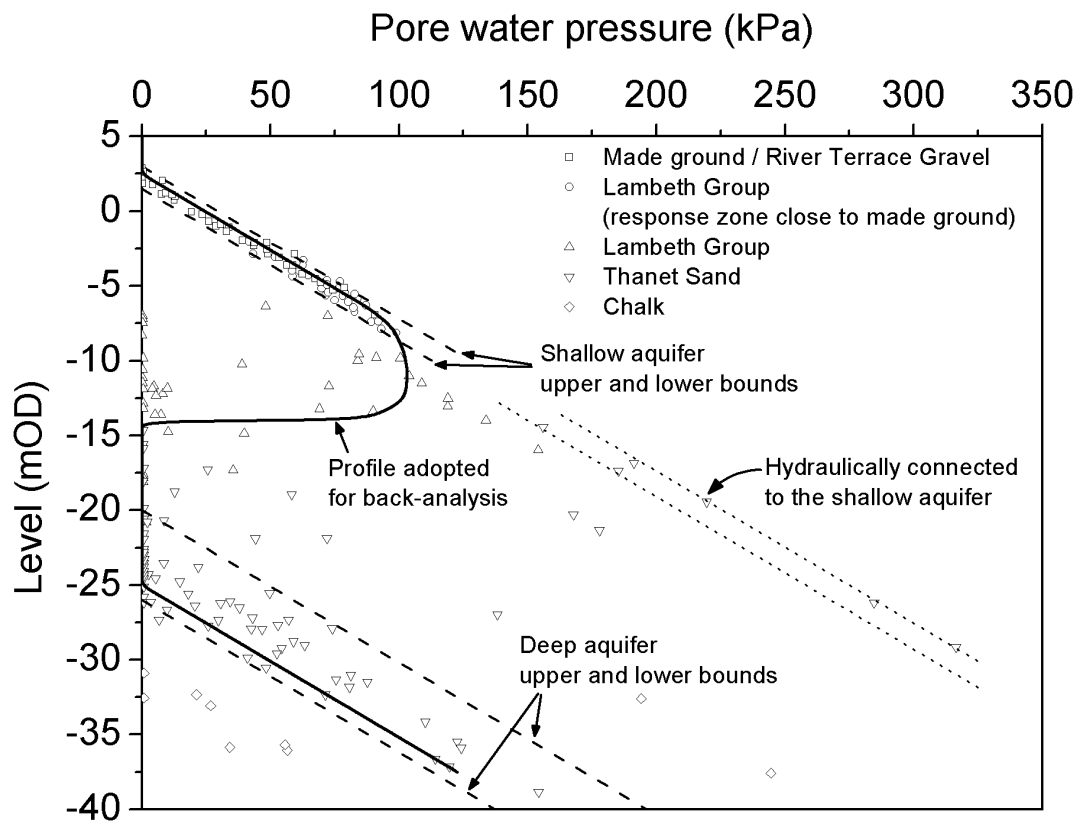


Figure 4.3: Pore water pressure in the vicinity of Stratford box. (Modified after Ove Arup & Partners Ltd, 2008.)

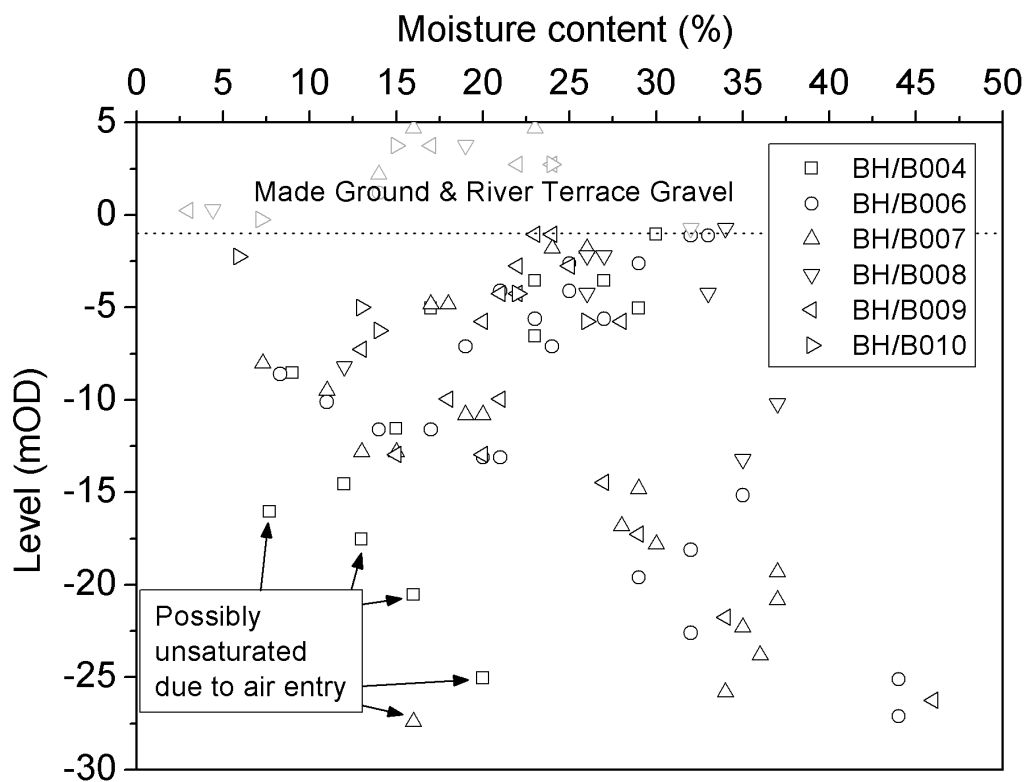


Figure 4.4: Moisture content test results. (Data after Norwest Holst Ltd, 2008.)

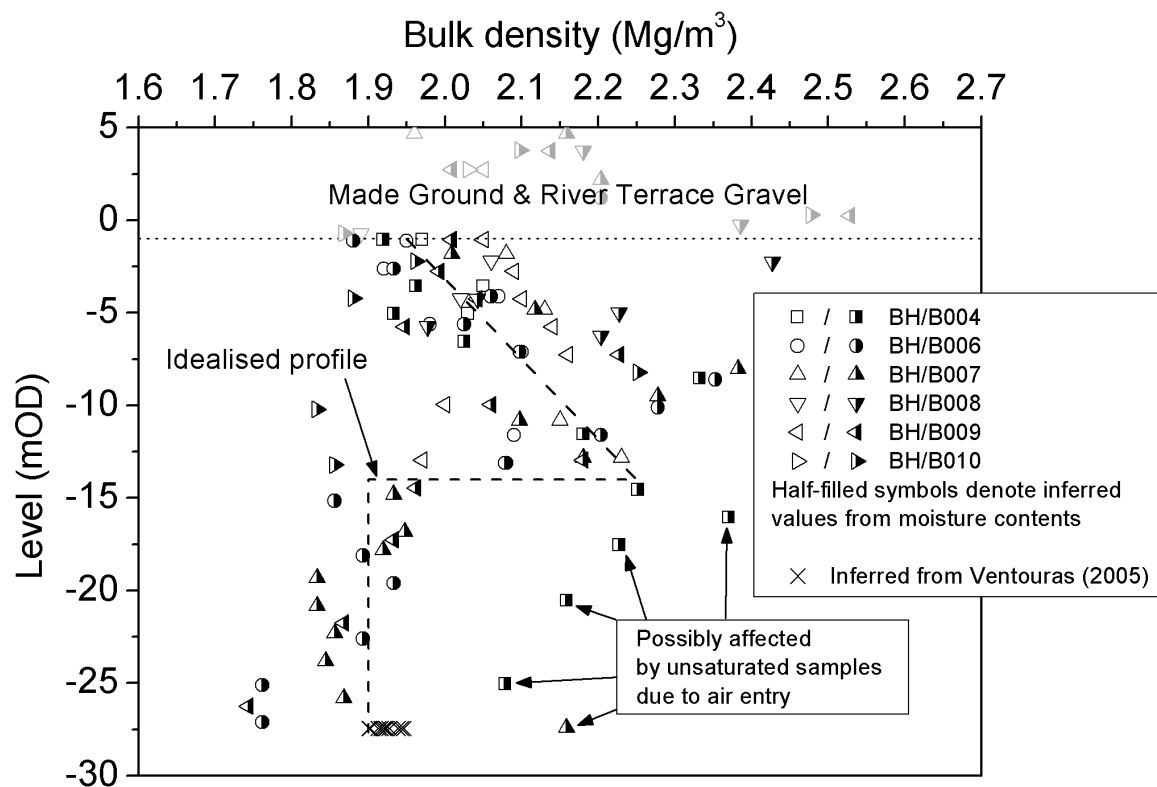


Figure 4.5: Bulk density test results. (Data after Norwest Holst Ltd, 2008 and Ventouras, 2005.)

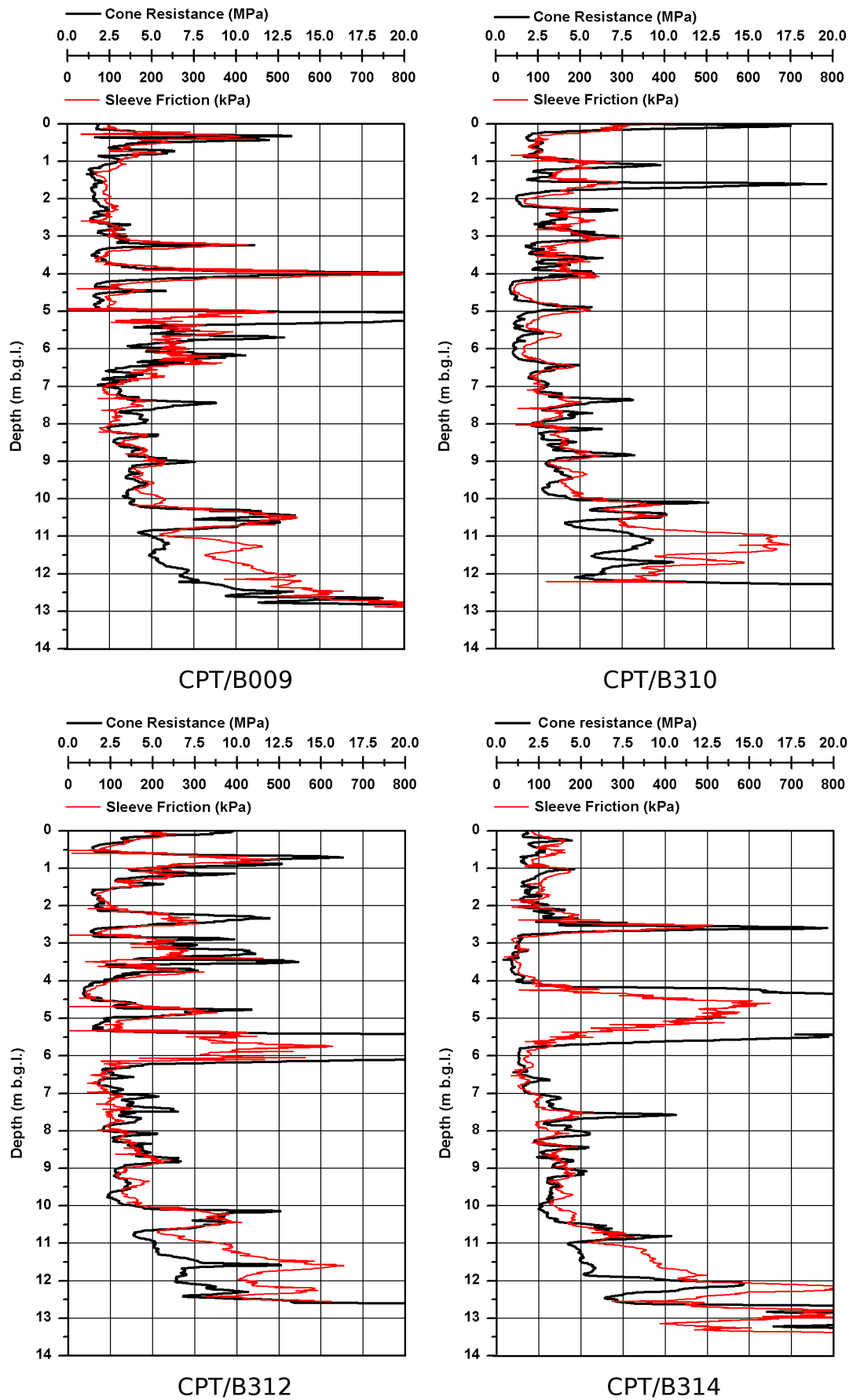


Figure 4.6: Cone penetration test results in the vicinity of the test piles – zero depth corresponds to 5.5 mOD which is the ground level at the test pile location. (After Norwest Holst Ltd, 2008.)

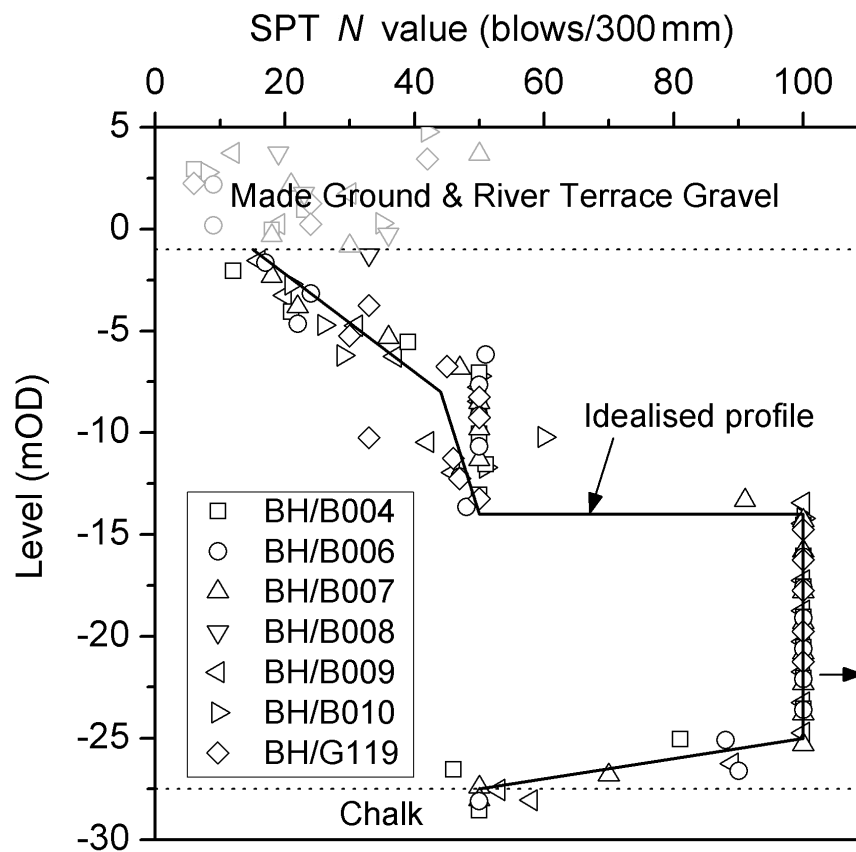


Figure 4.7: Standard penetration test results in the vicinity of the test piles. (Data after Norwest Holst Ltd, 2008.)

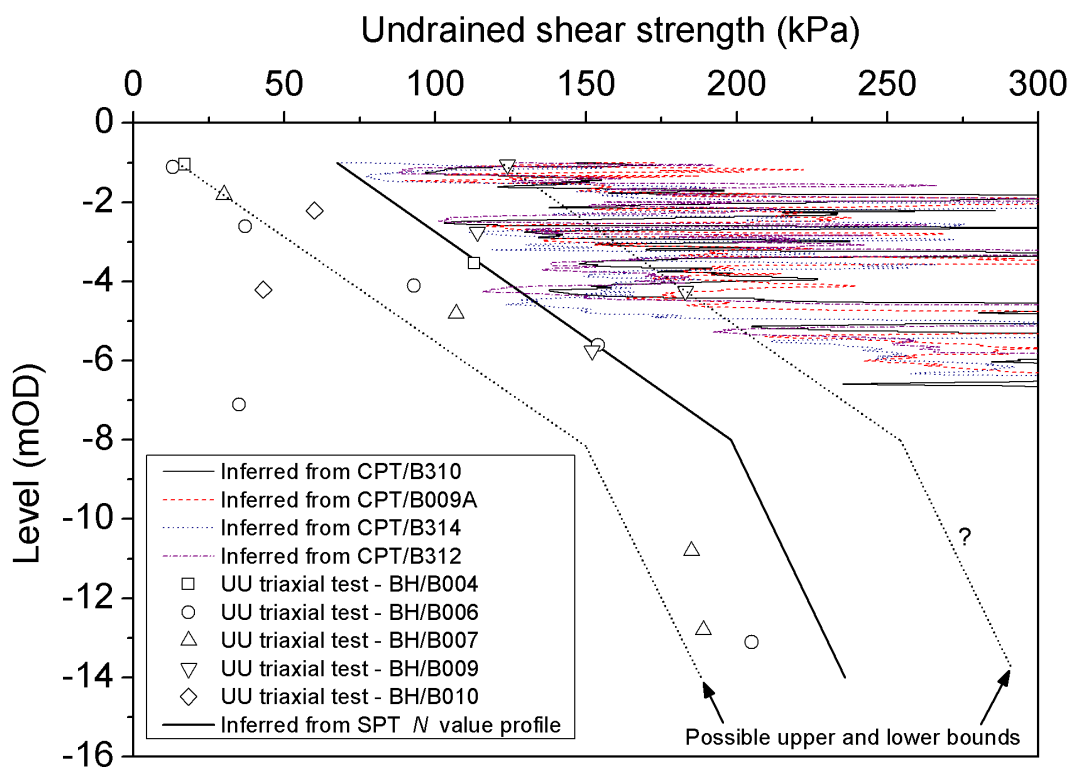


Figure 4.8: Undrained shear strength of Lambeth Group. (Data after Norwest Holst Ltd, 2008.)

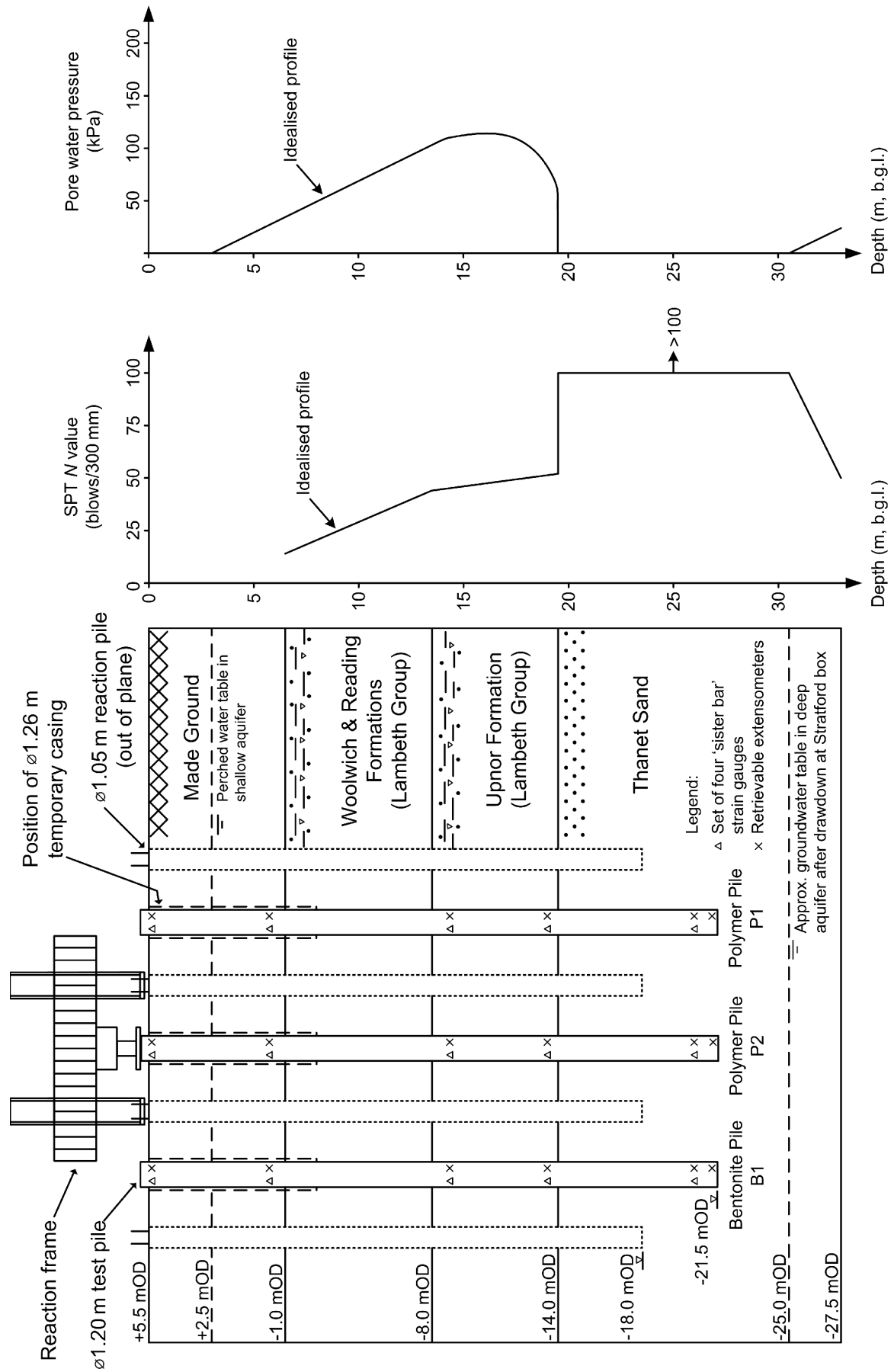


Figure 4.9: Test pile details and ground conditions.

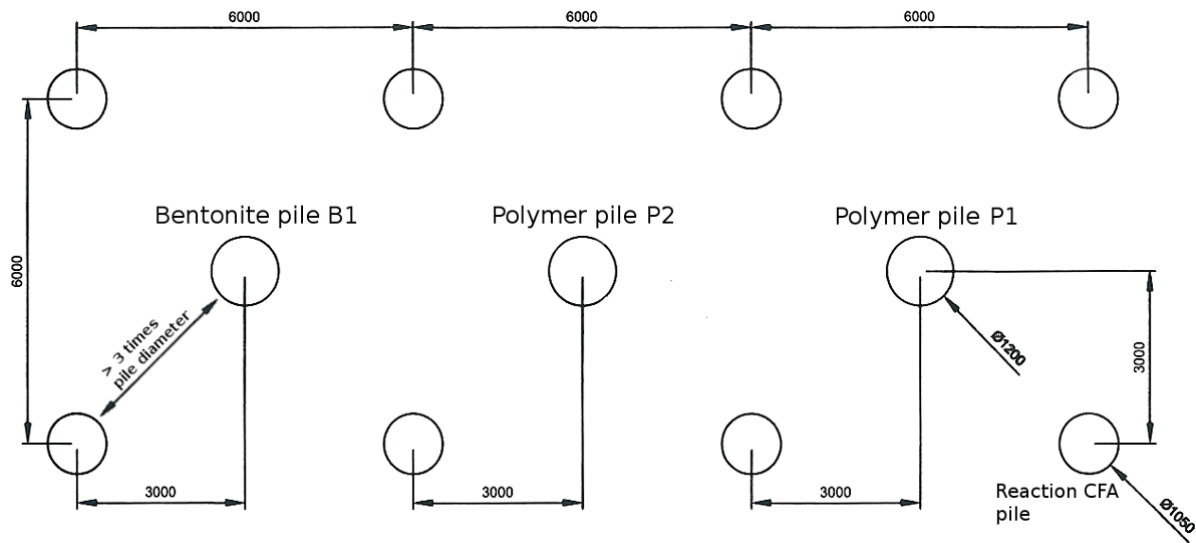


Figure 4.10: Setting out of test and reaction piles.

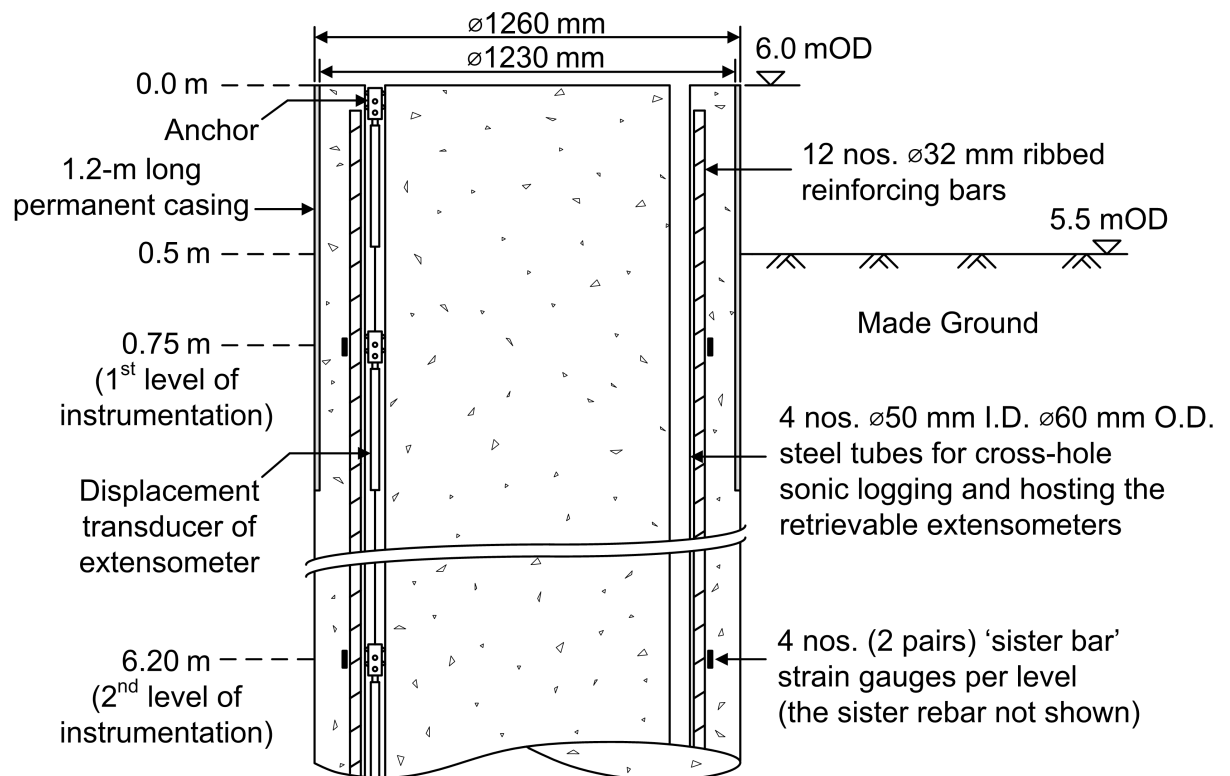


Figure 4.11: Pile head details.



Figure 4.12: ‘Sister bar’ strain gauges and access tubes for cross-hole sonic logging and extensometer installation: *left*) instrumented steel cage; *right*) close-up.



Figure 4.13: SlurryPro CDP and Pro-Tek 100.



Figure 4.14: Dropping an MPA ‘bomb’ into a pile bore. (Photograph taken at the Castle House site in London.)



Figure 4.15: Effect of additive SlurryPro MPA on ‘dirty’ polymer fluids: *left*) before addition; *right*) after addition and stirring. (Photographs taken at a site in Glasgow, Scotland.)



Figure 4.16: Bentonite mixing and storage equipment.



Figure 4.17: Adding bentonite powder into a mixer.



Figure 4.18: Temporary casing.



Figure 4.19: Vented digging bucket – note the teeth at the base.



Figure 4.20: Digging bucket vent.



Figure 4.21: Flat-sided cleaning bucket – note the teeth at the base are replaced with a scraper plate.

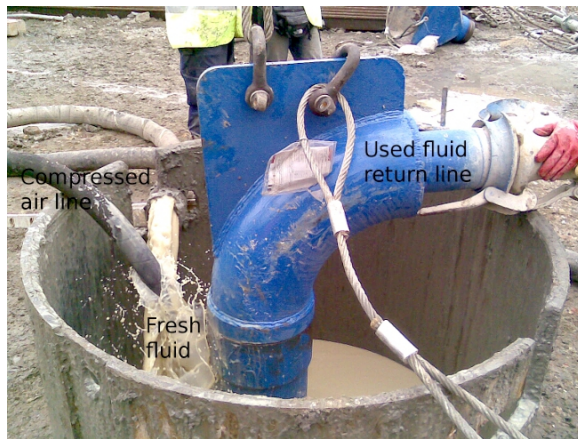


Figure 4.22: Air lifting operation to exchange bentonite fluids.



Figure 4.23: Hand-held density meter.

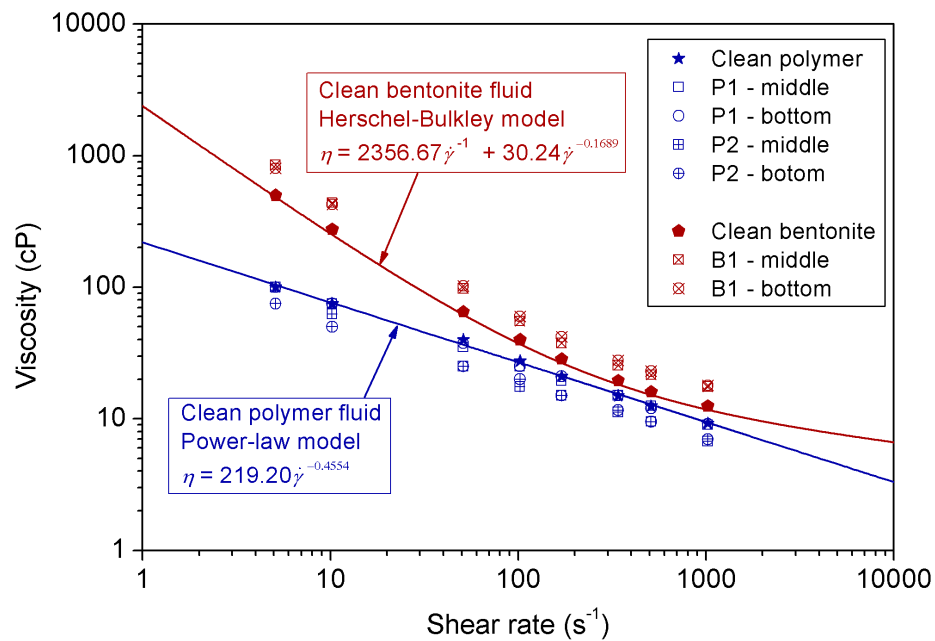


Figure 4.24: Flow curves of polymer and bentonite fluids used for the Stratford field trial.



Figure 4.25: Tremie concreting.



Figure 4.26: Taking samples from the concrete-support fluid interface.

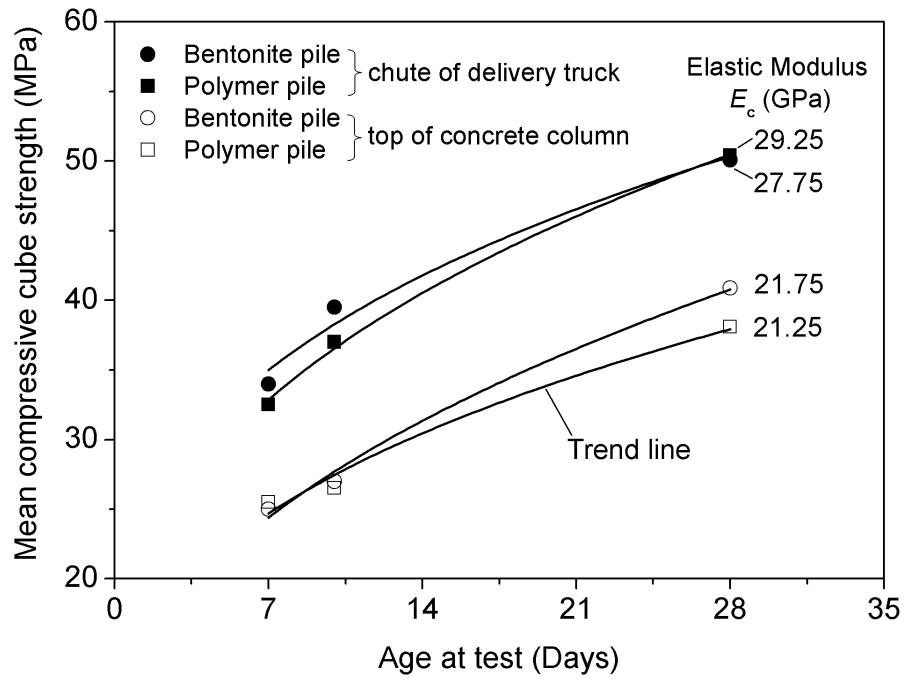


Figure 4.27: Effect of support fluid type on the strength and stiffness of hardened concrete.

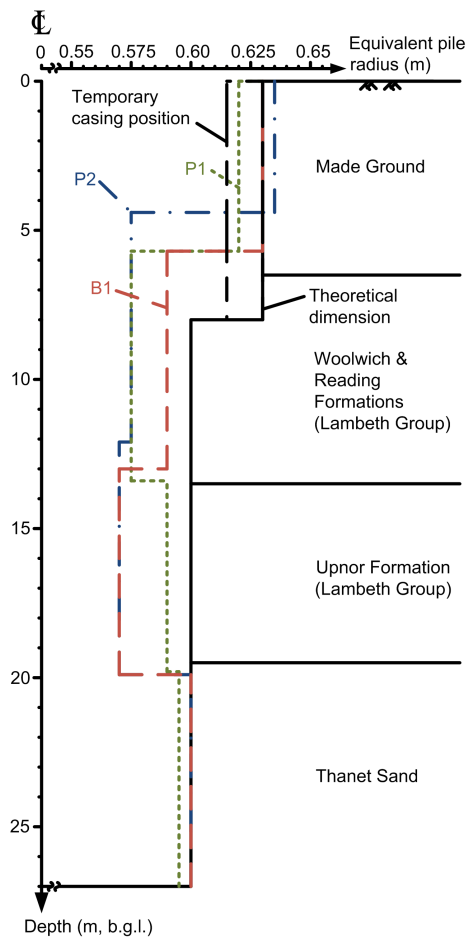


Figure 4.28: Equivalent pile radii from concreting records.

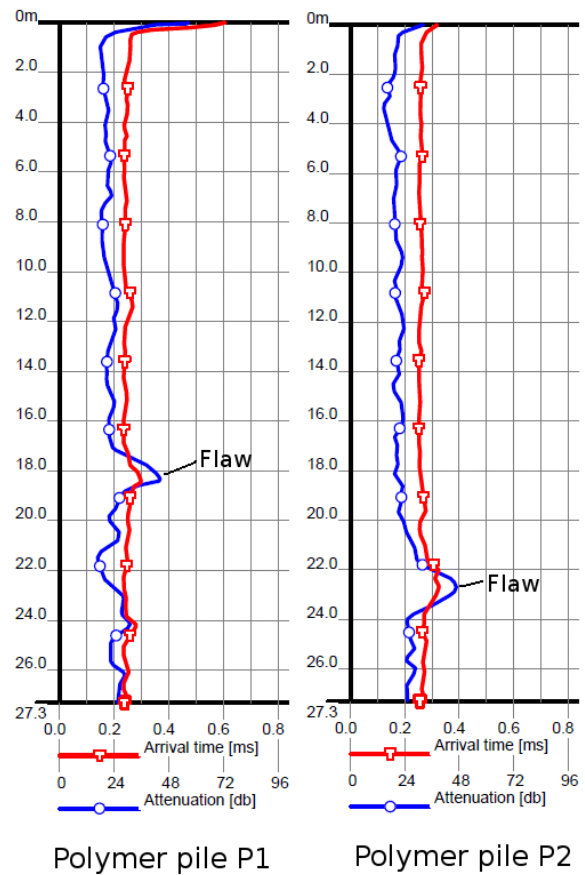


Figure 4.29: Cross-hole sonic logging results for the two polymer piles. (Note the depth is measured from the pile head.)

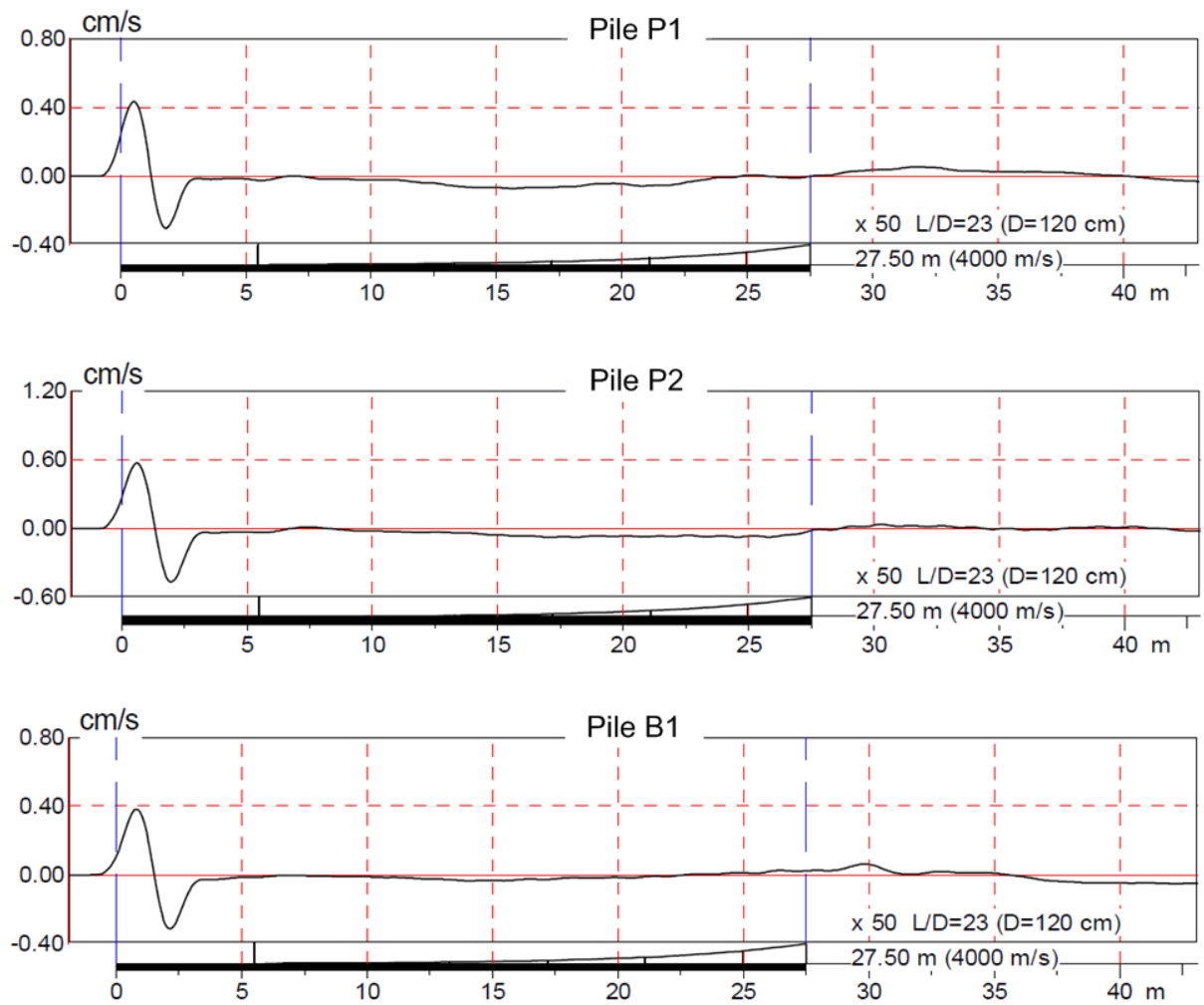


Figure 4.30: Pile integrity test results.



Figure 4.31: Preparing pile head for the load test and some instrumentation equipment on the ground surface.



Figure 4.32: Reaction frame being constructed for the load test.

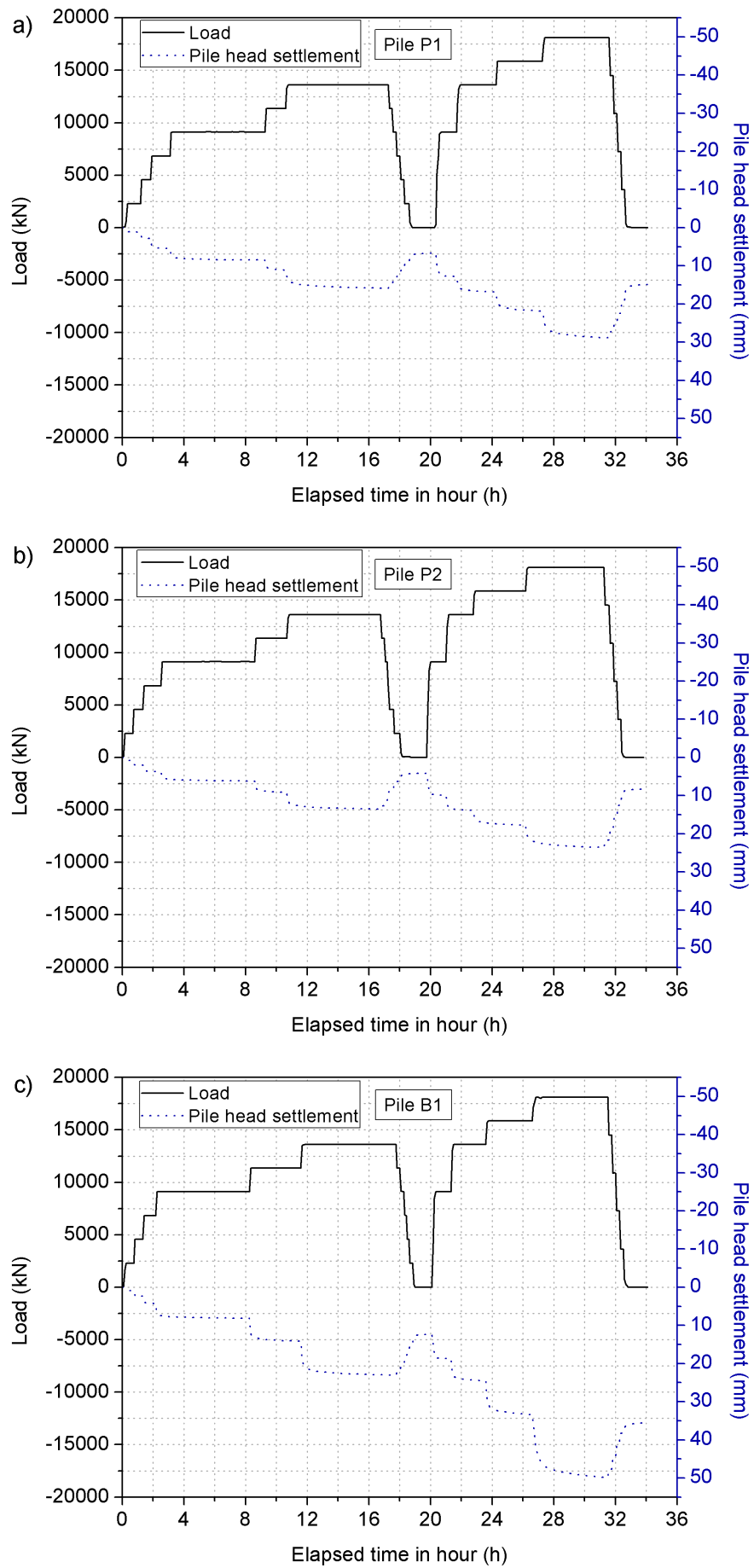


Figure 4.33: Load and settlement changes with time: a) polymer pile P1; b) polymer pile P2; c) bentonite pile B1.

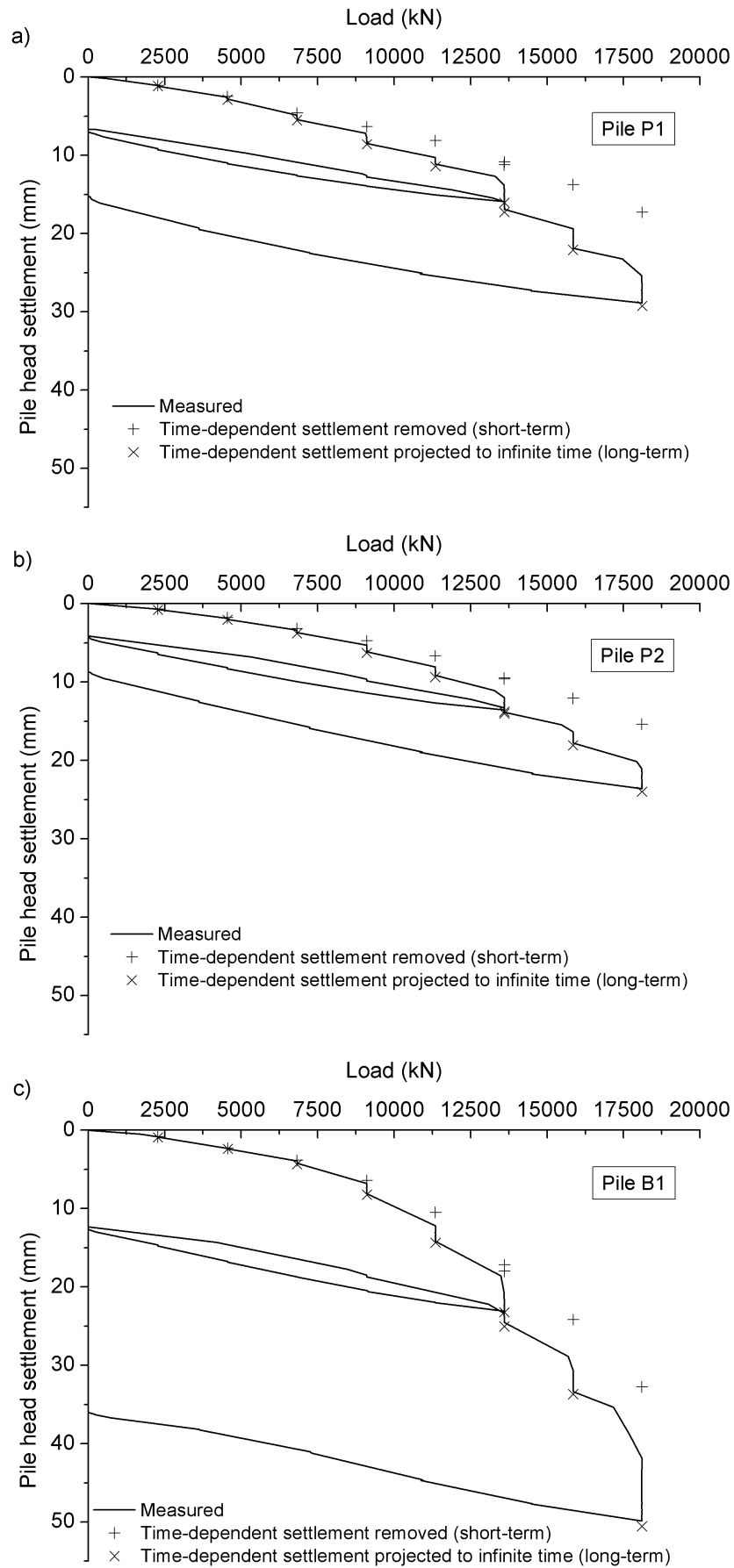


Figure 4.34: Load versus settlement plots: a) polymer pile P1; b) polymer pile P2; c) bentonite pile B1.

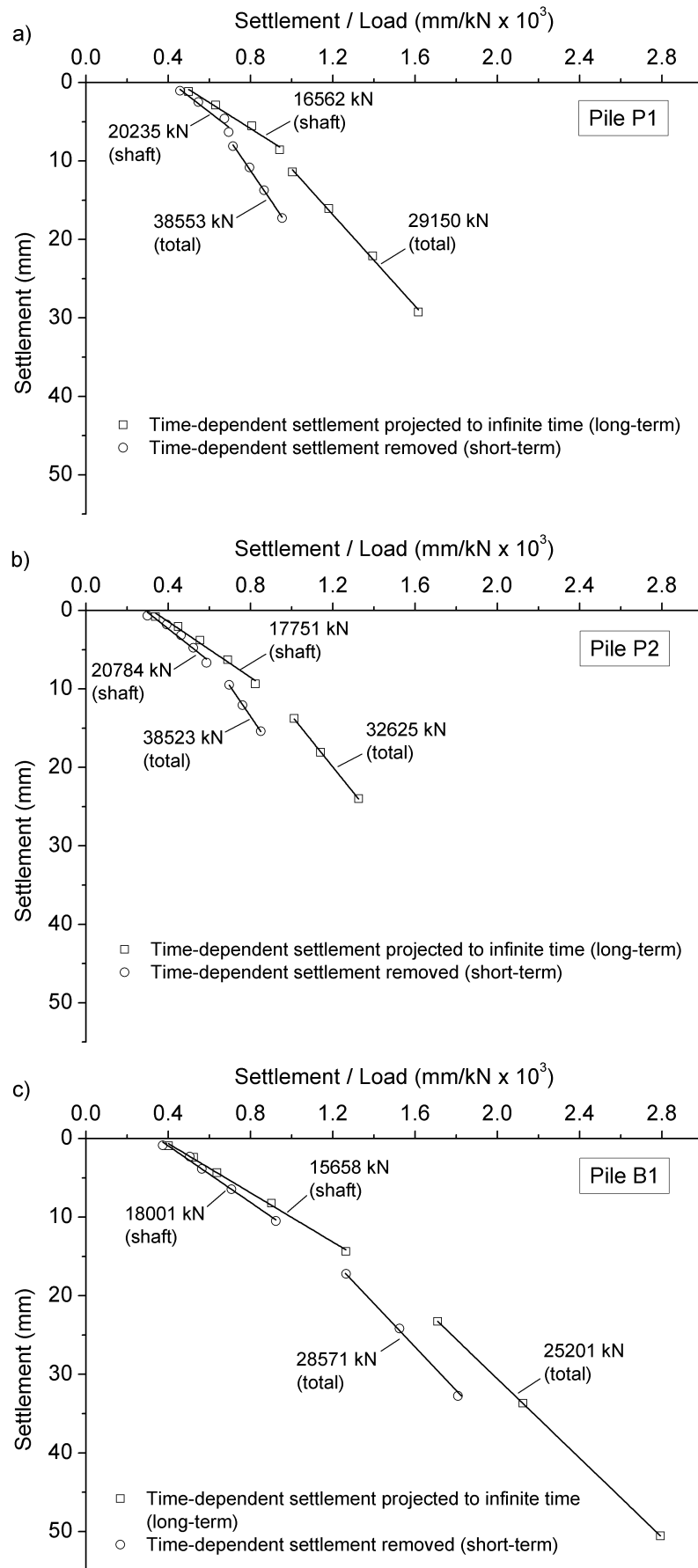


Figure 4.35: Ultimate shaft and total resistances using the method proposed by Chin (1970): a) polymer pile P1; b) polymer pile P2; c) bentonite pile B1.

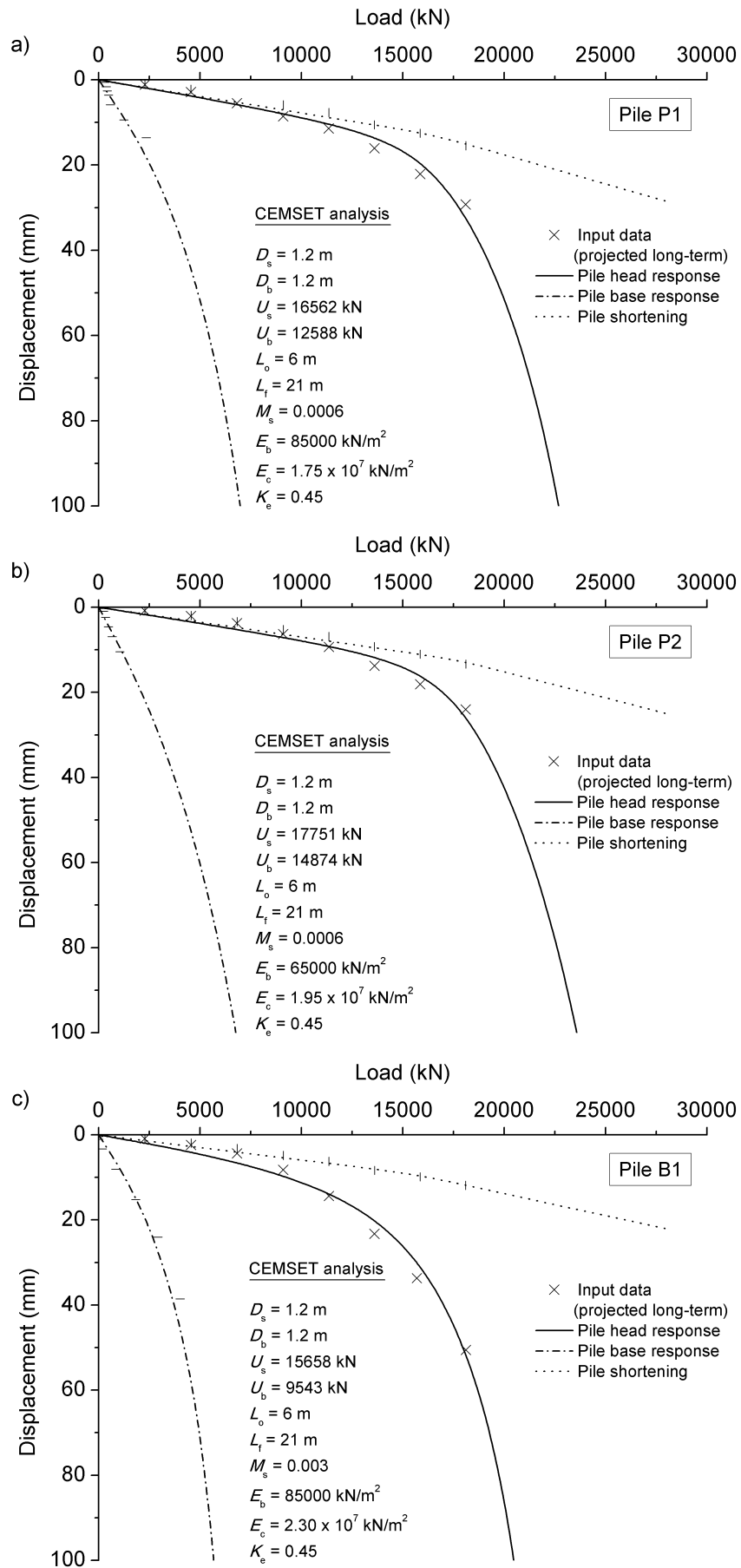


Figure 4.36: Measured and simulated response of test piles using the method proposed by Fleming (1992): a) polymer pile P1; b) polymer pile P2; c) bentonite pile B1.

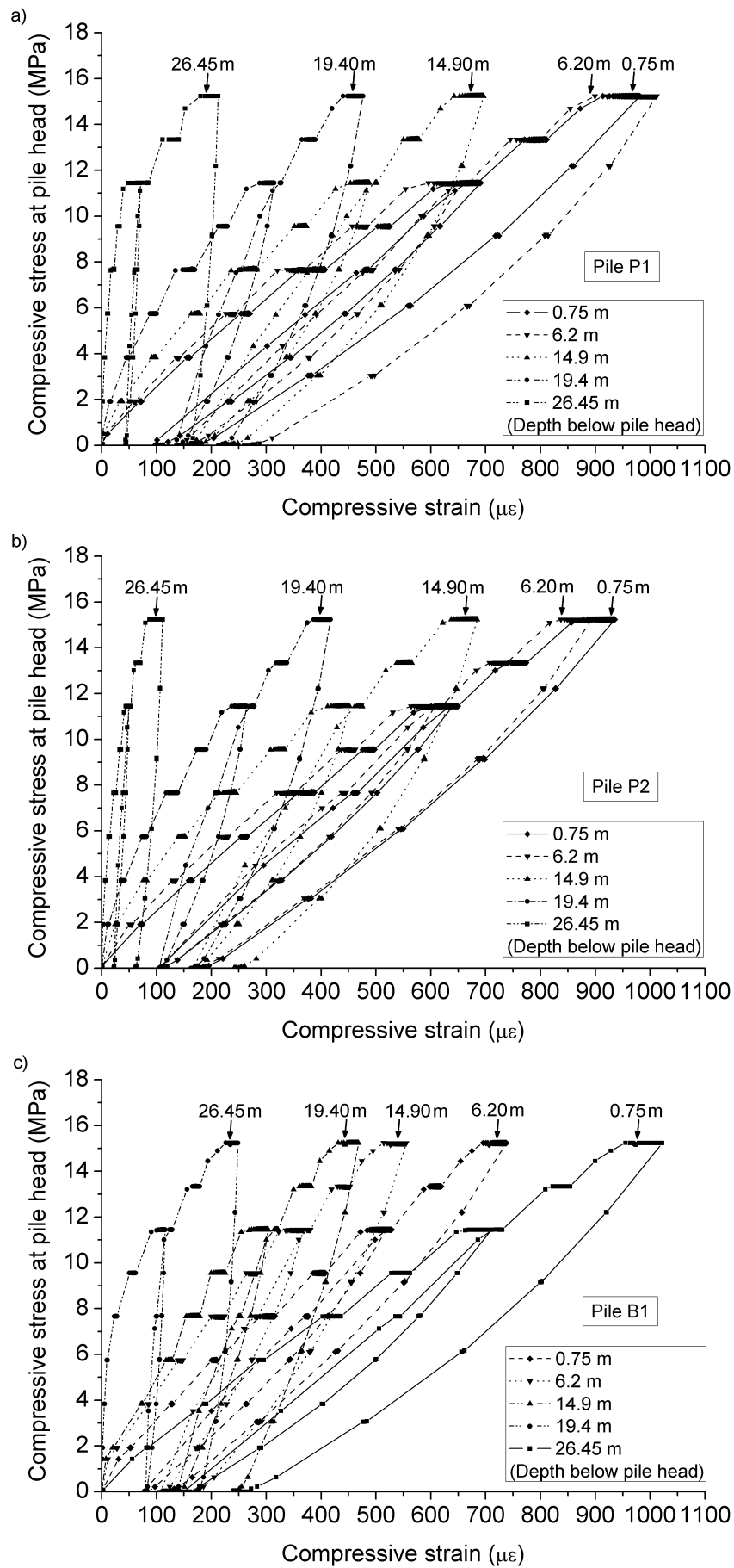


Figure 4.37: Stress–strain curves of pile material at different levels measured by ‘sister bar’ strain gauges: *a*) polymer pile P1; *b*) polymer pile P2; *c*) bentonite pile B1.

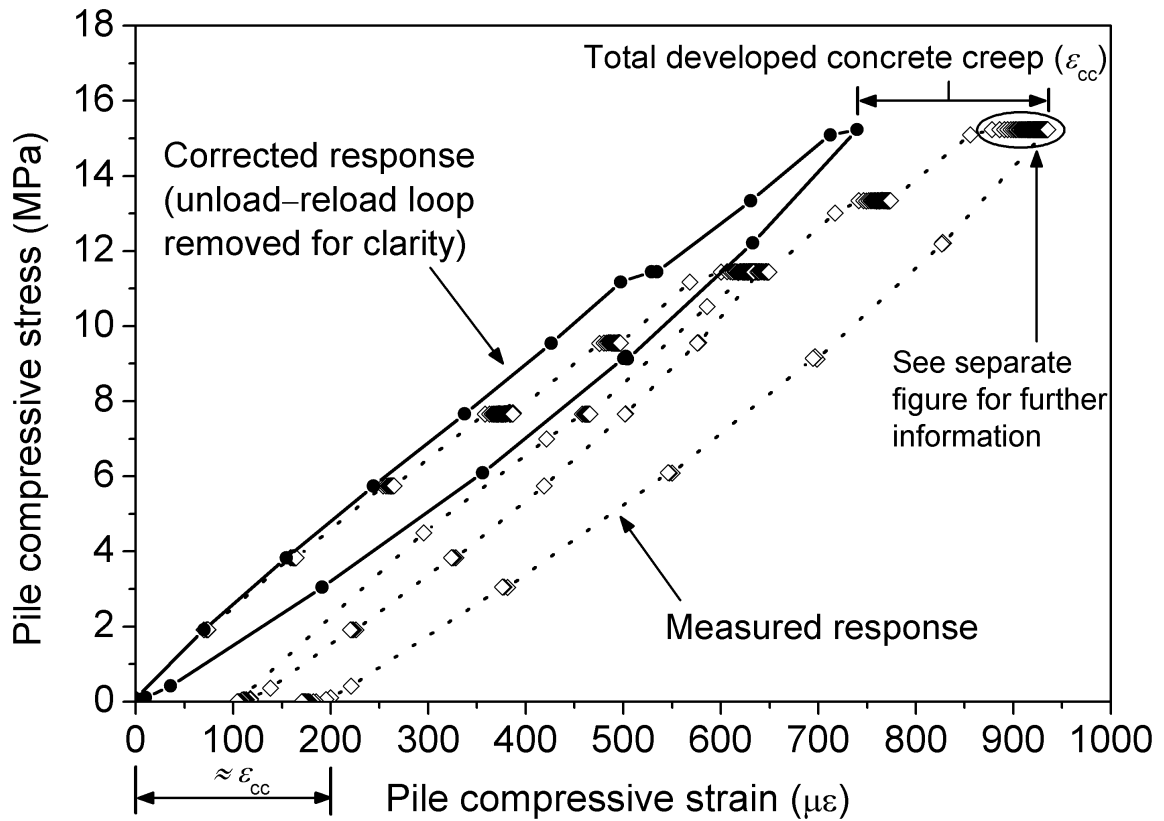


Figure 4.38: Measured and corrected stress-strain curves of pile material at the uppermost instrumentation level as recorded by the 'sister bar' strain gauges at pile P2.

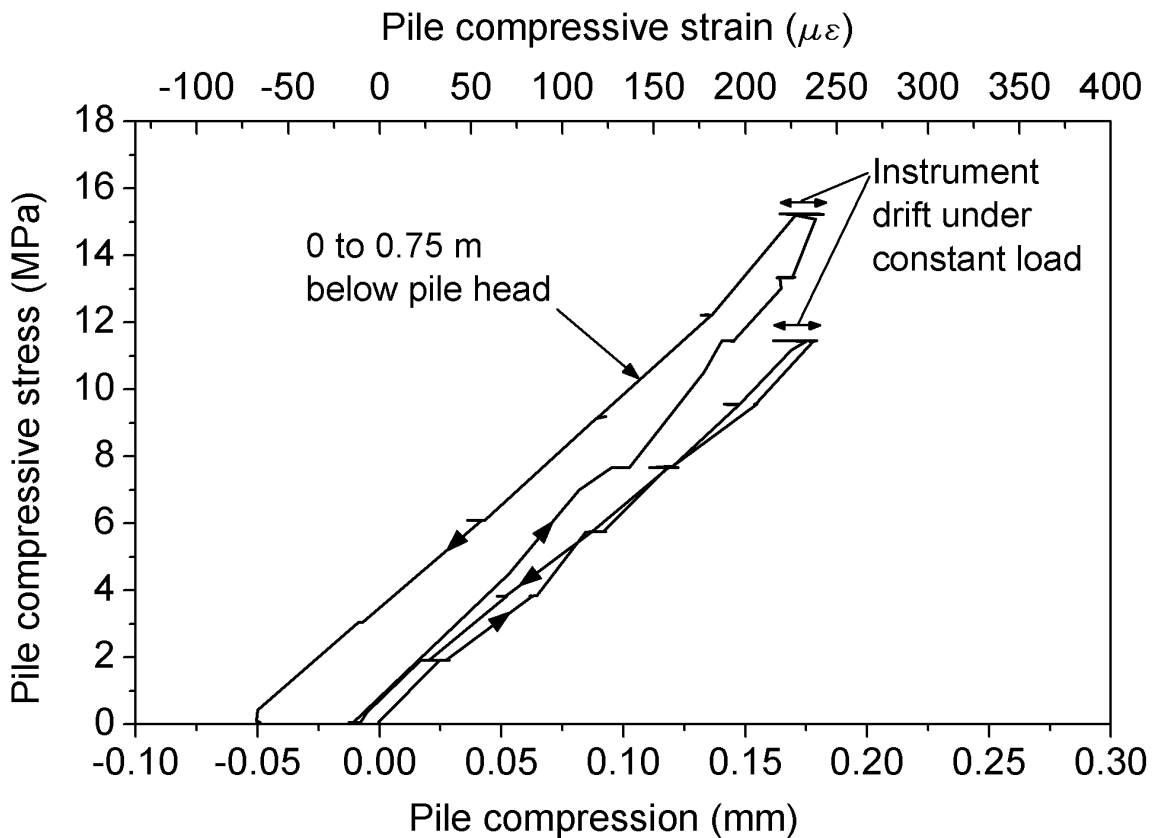


Figure 4.39: Stress-strain curve of pile material at the uppermost instrumentation level as recorded by the retrievable extensometer at pile P2.

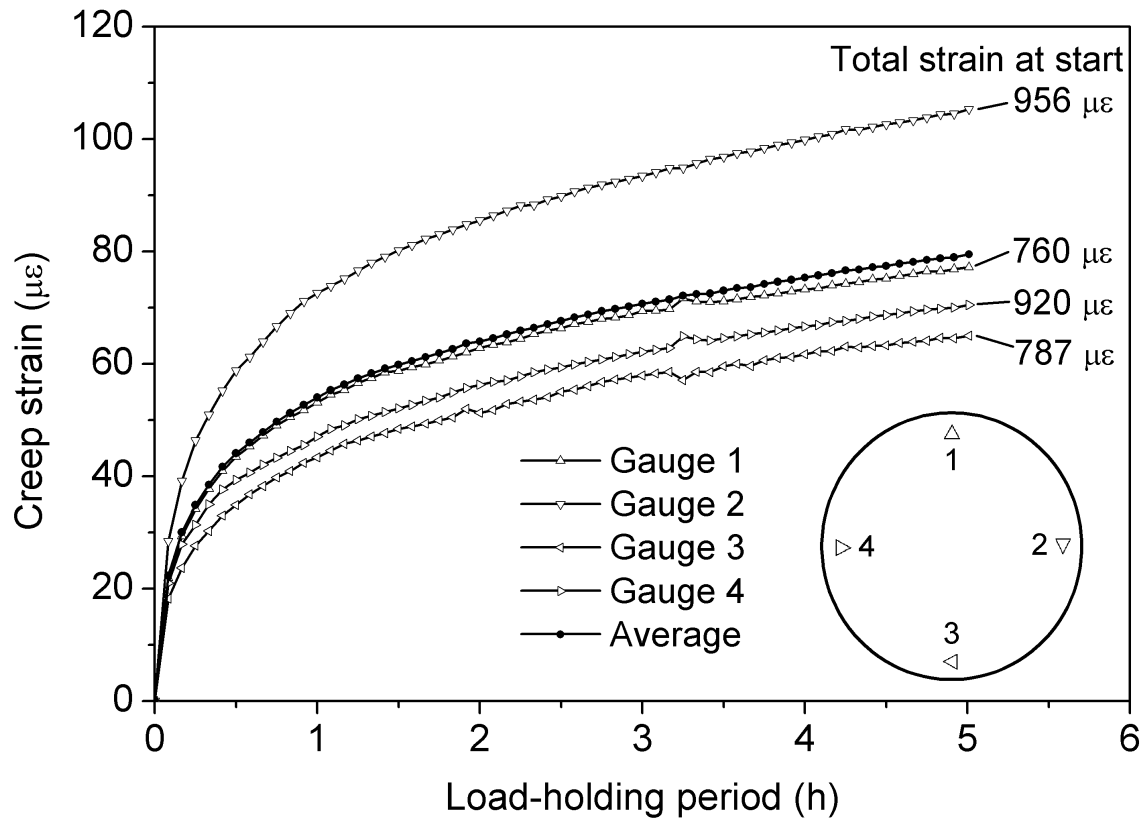


Figure 4.40: Development creep strain in pile material at the uppermost instrumentation level of pile P2 under proof load.

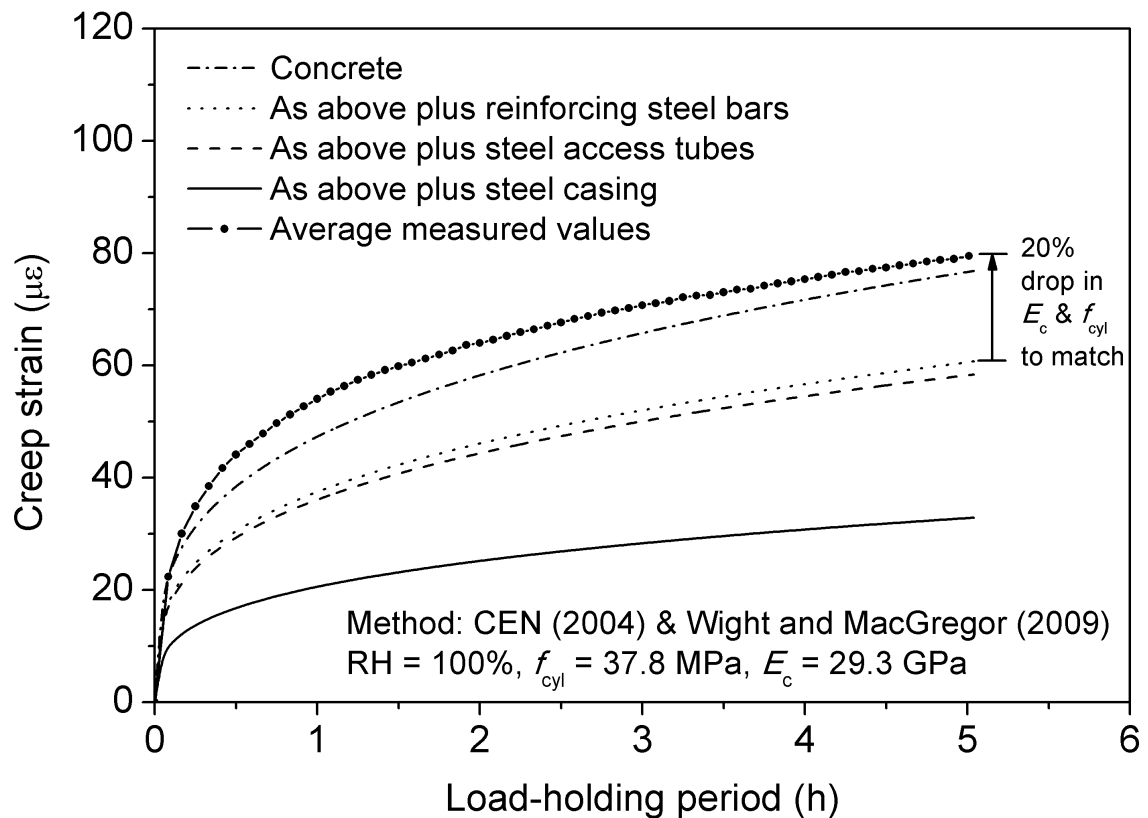


Figure 4.41: Predicted and measured creep strain in pile material at the uppermost instrumentation level of pile P2 under proof load.

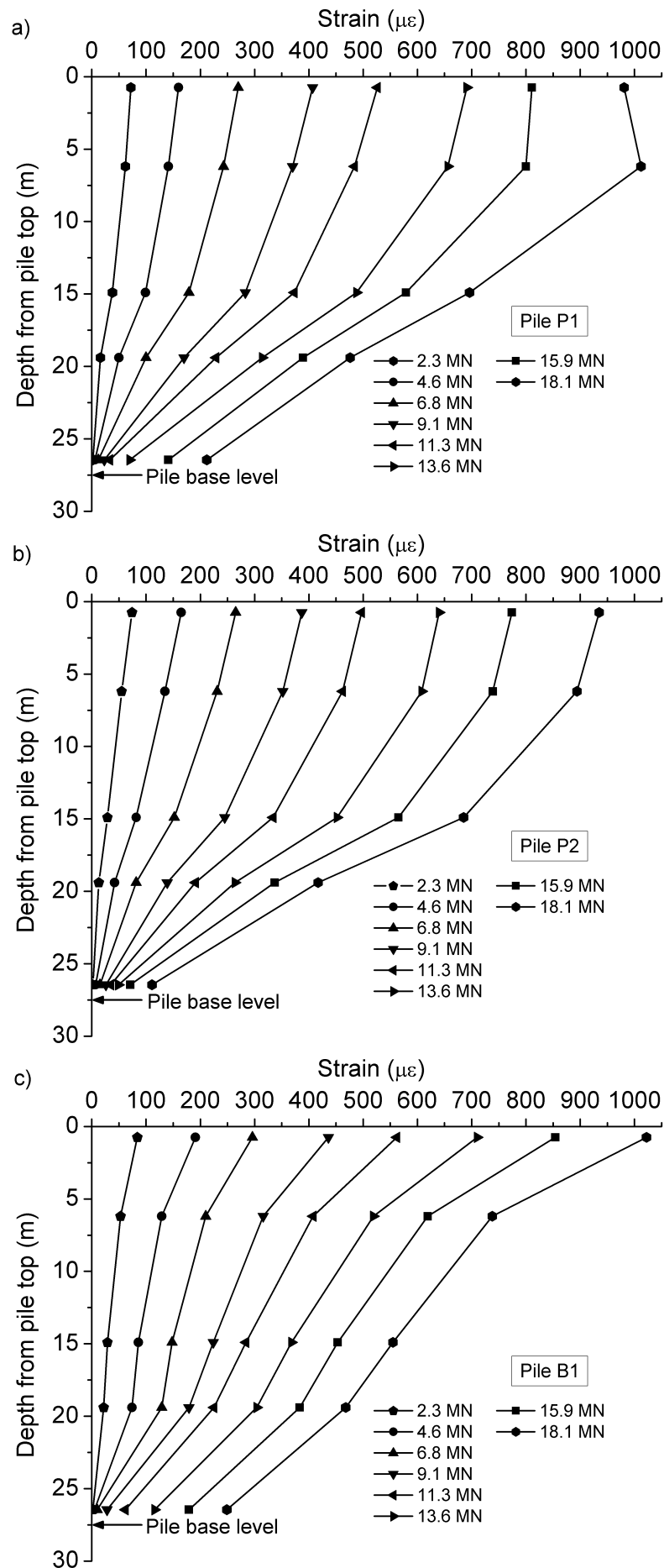


Figure 4.42: Measured strain profiles at the end of each load-holding period: a) polymer pile P1; b) polymer pile P2; c) bentonite pile B1.

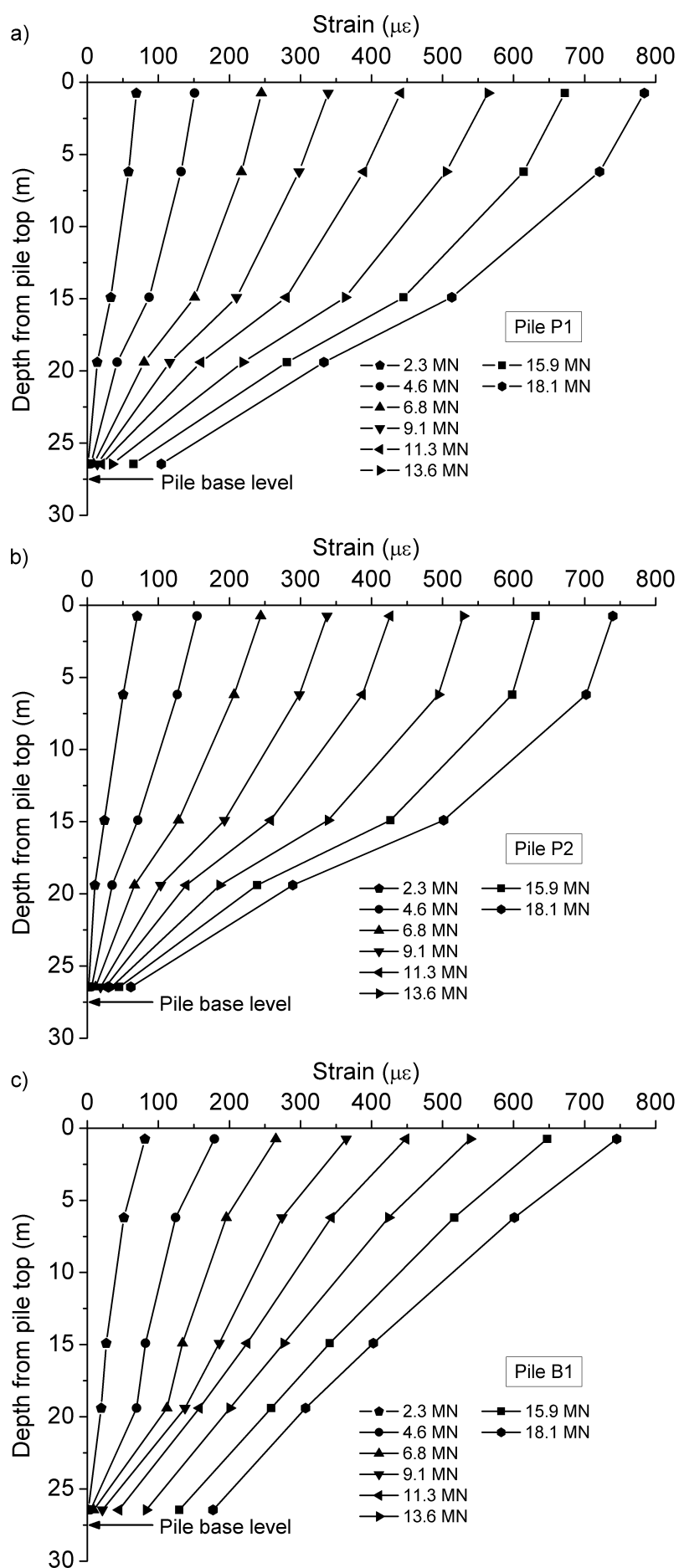


Figure 4.43: Strain profiles corrected for the effect of concrete creep: a) polymer pile P1; b) polymer pile P2; c) bentonite pile B1.

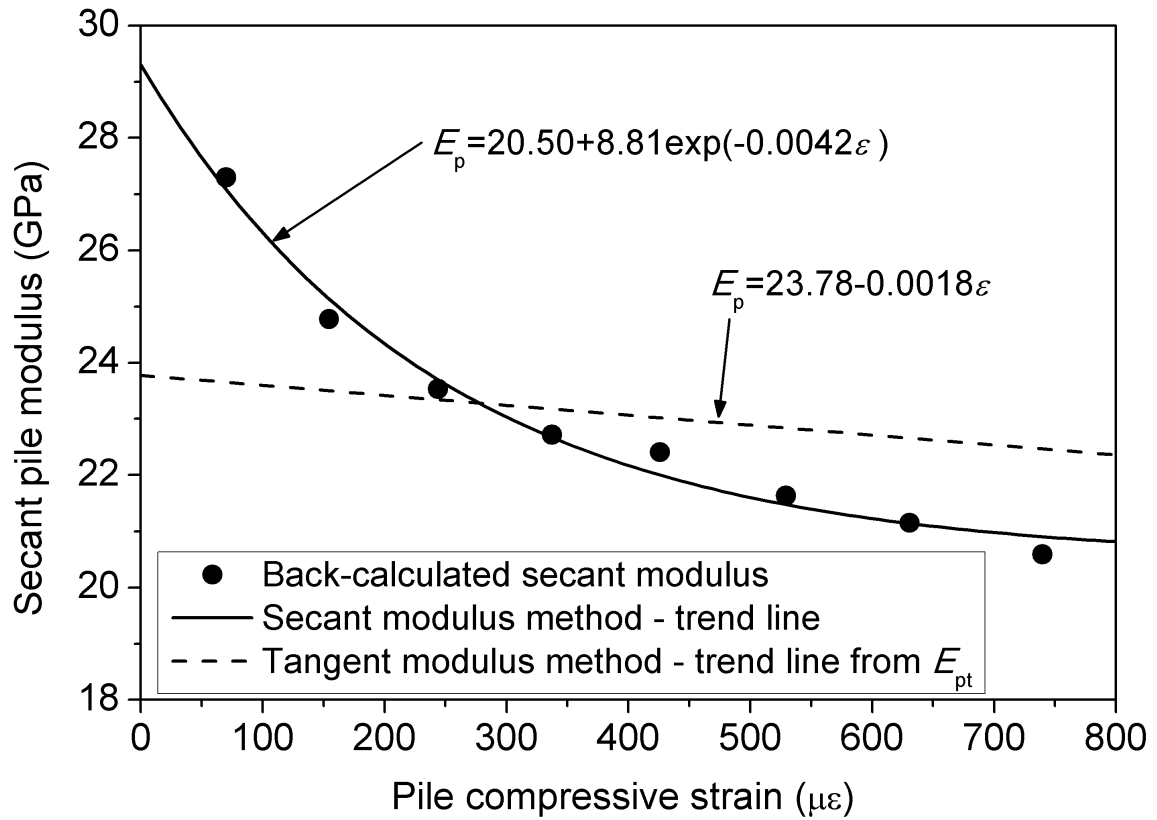


Figure 4.44: Secant modulus of the material of pile P2.

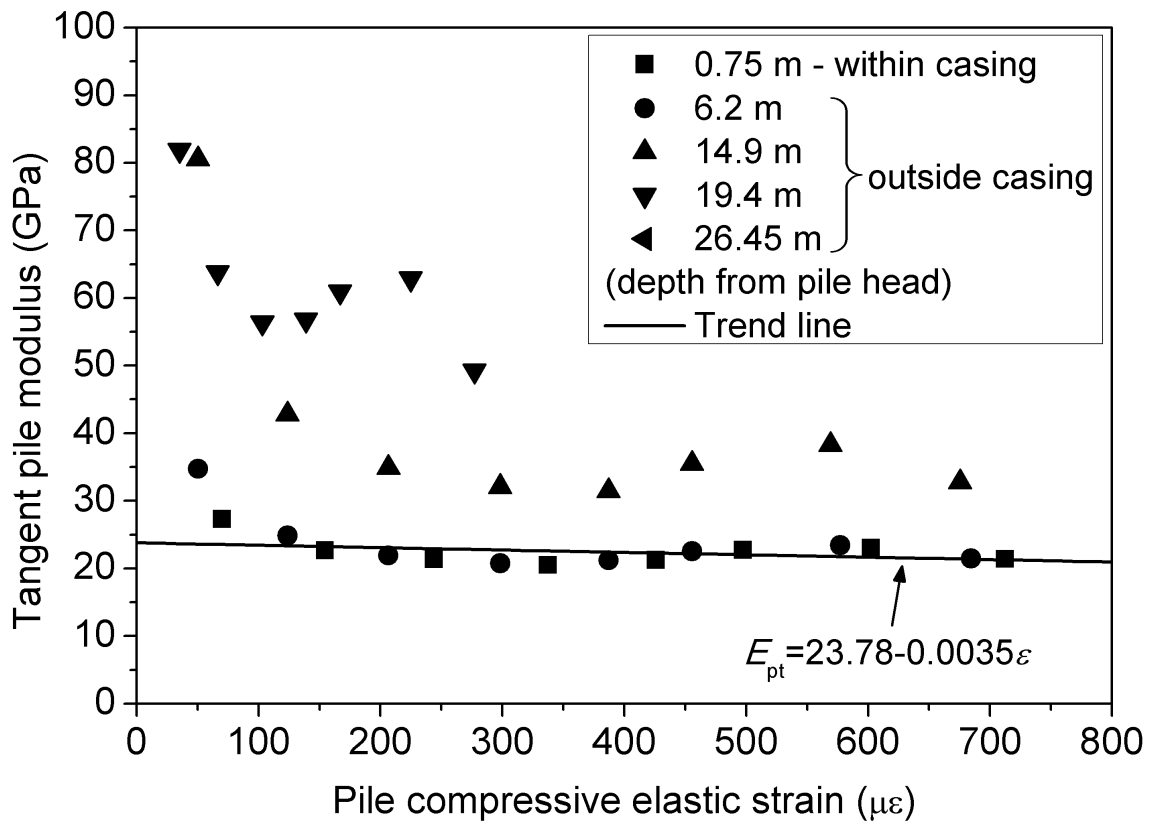


Figure 4.45: Tangent modulus of the material of pile P2.

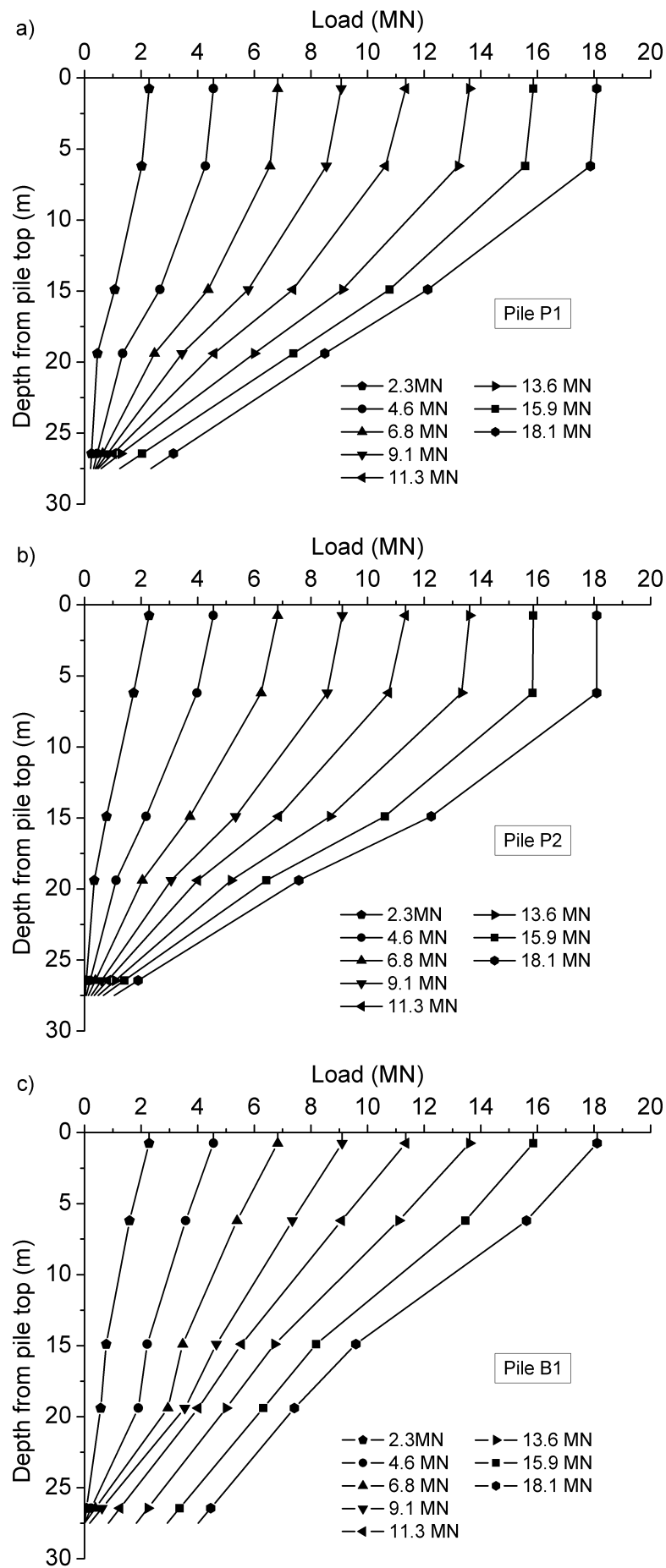


Figure 4.46: Interpreted load distribution: a) polymer pile P1; b) polymer pile P2; c) bentonite pile B1.

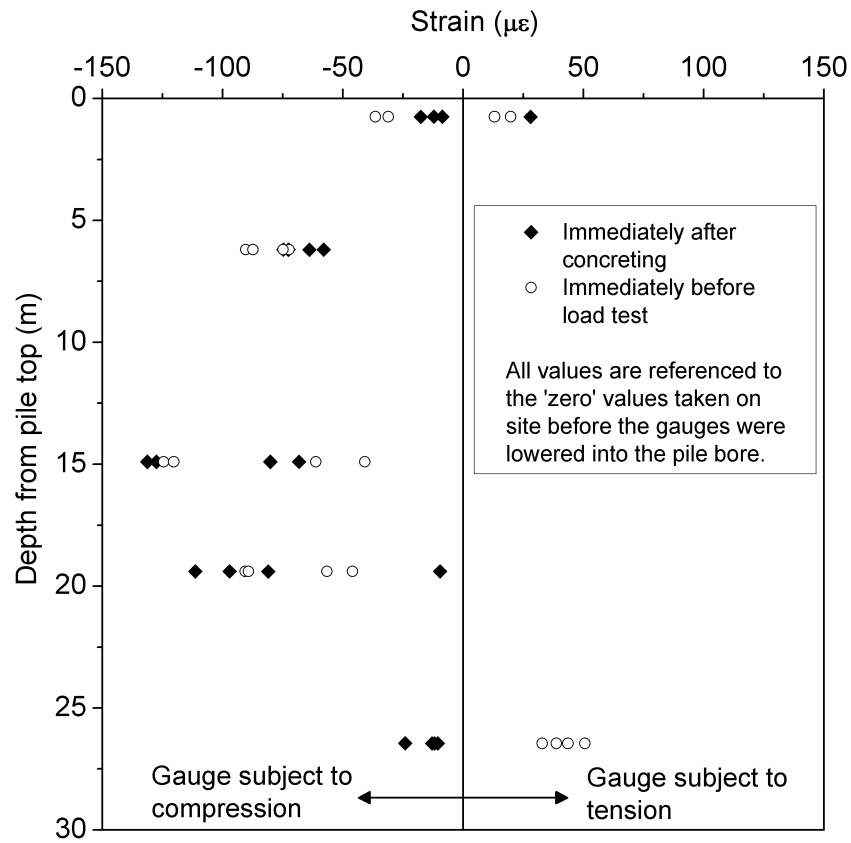


Figure 4.47: Effect of installation on strain gauge readings.

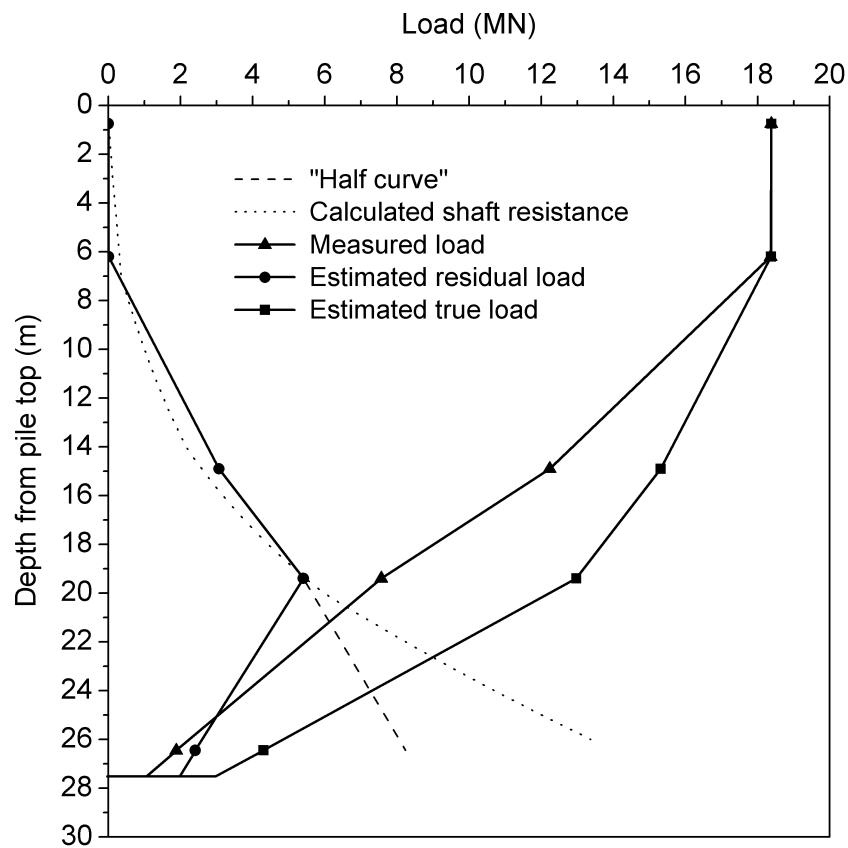


Figure 4.48: Estimation of residual loads using the method proposed by Fellenius (2002b).

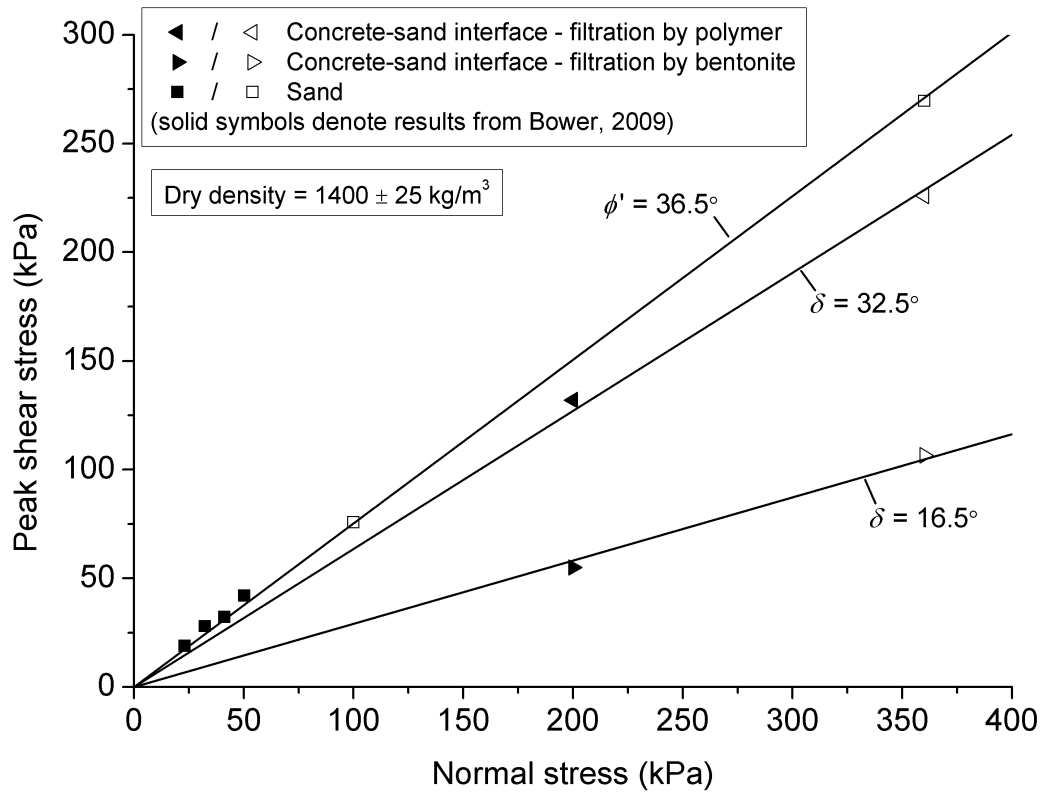


Figure 4.49: Failure envelopes of Thanet Sand and its interface with concrete after filtration by polymer and bentonite support fluids.

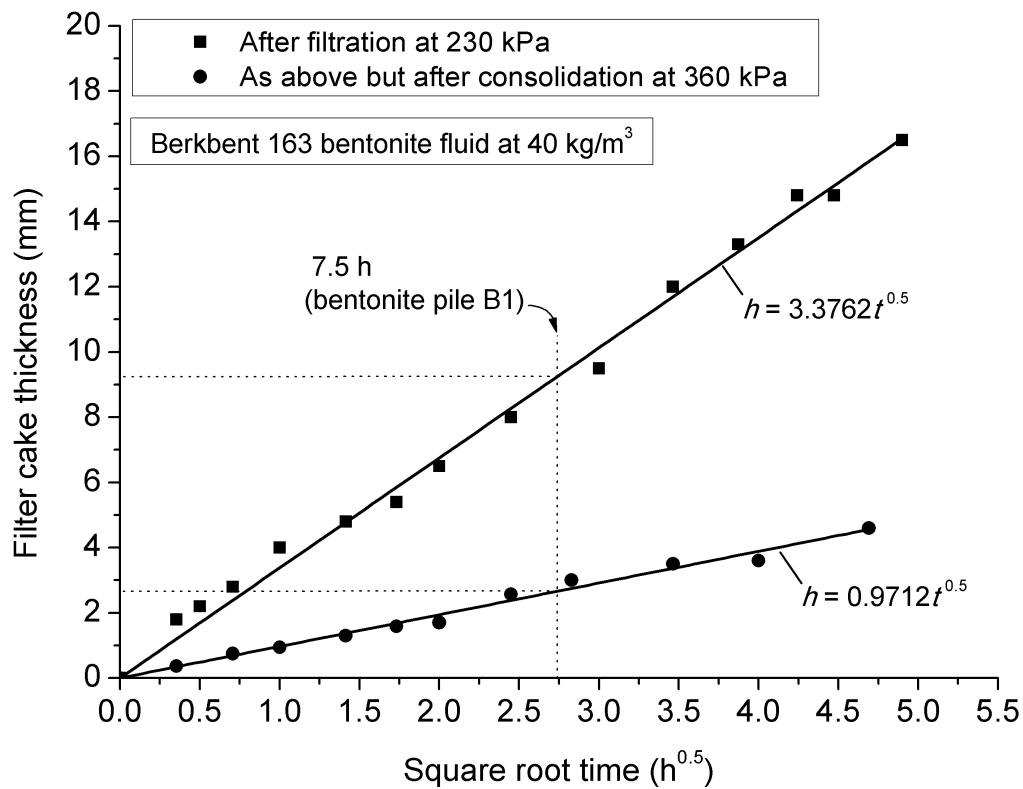


Figure 4.50: Thickness of bentonite filter cake.

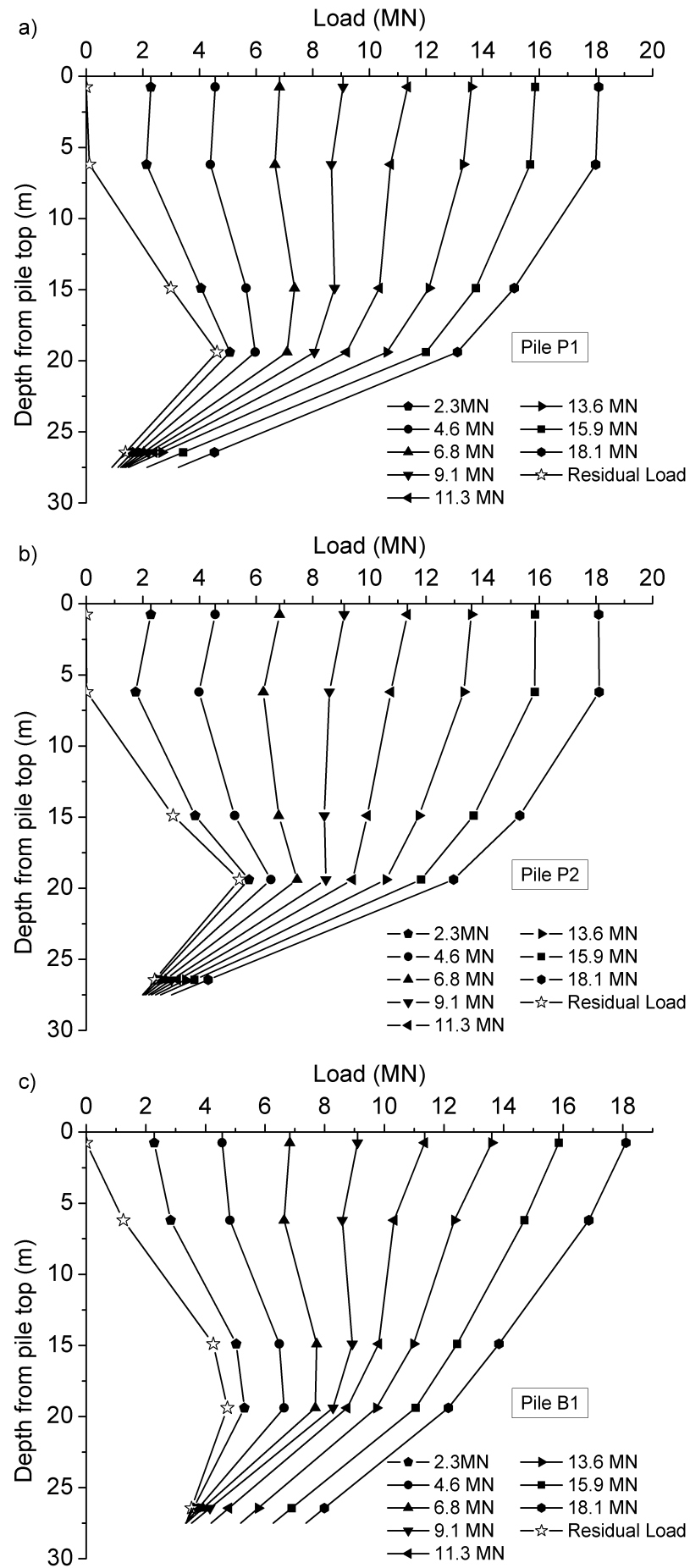


Figure 4.51: Interpreted load distribution including residual load: *a)* polymer pile P1; *b)* polymer pile P2; *c)* bentonite pile B1.

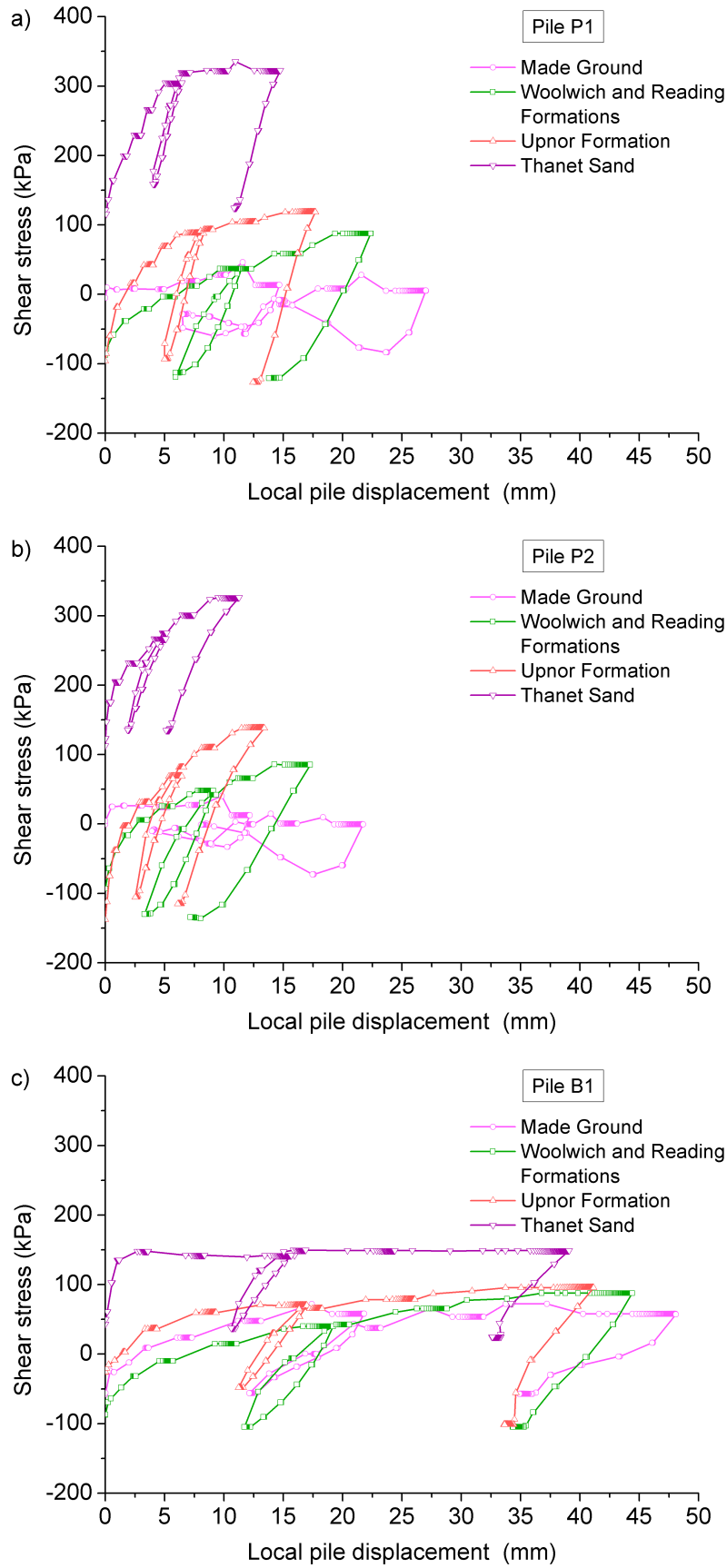


Figure 4.52: Mobilised shear stress of each soil layer against measured pile settlement: *a*) polymer pile P1; *b*) polymer pile P2; *c*) bentonite pile B1.

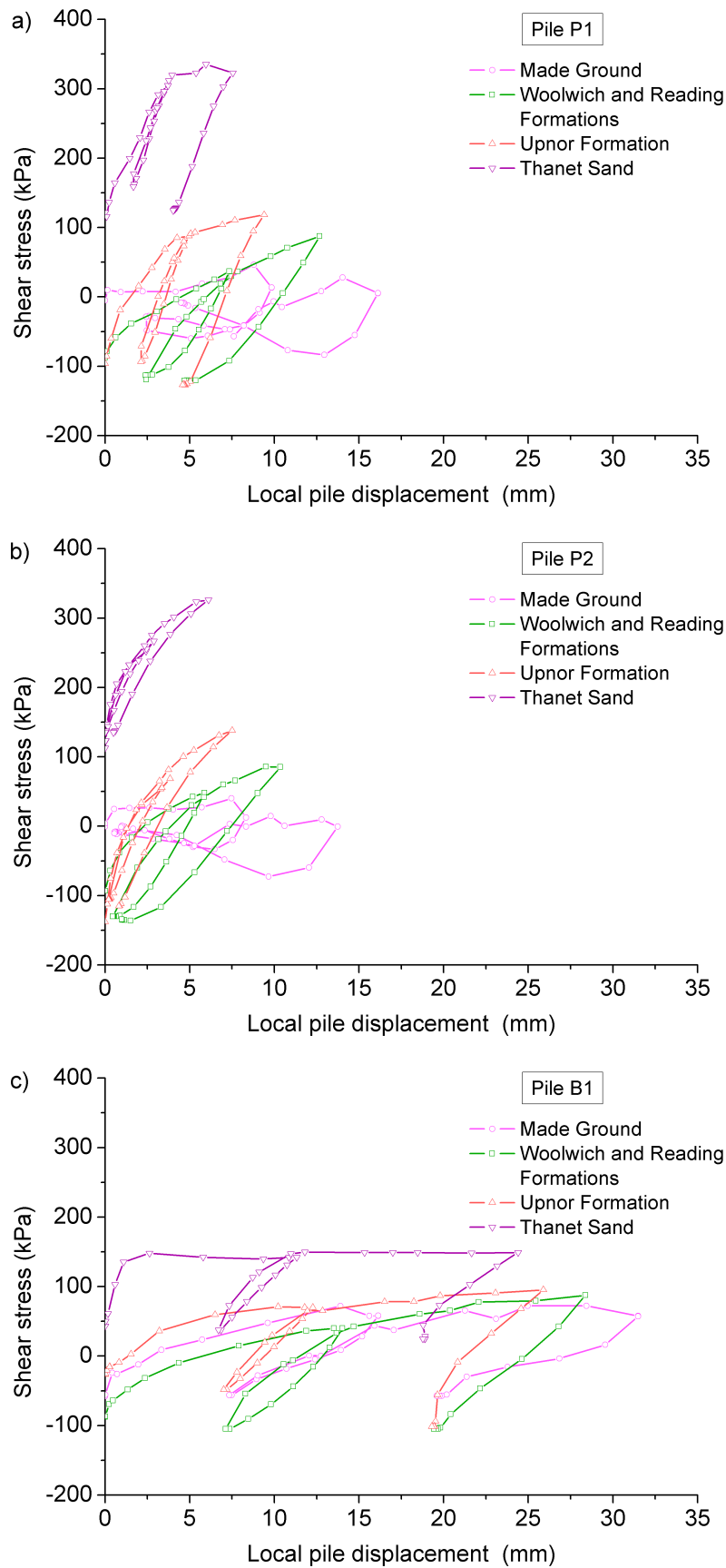


Figure 4.53: Mobilised shear stress of each soil layer against pile settlement corrected for the effect of creep: *a)* polymer pile P1; *b)* polymer pile P2; *c)* bentonite pile B1.

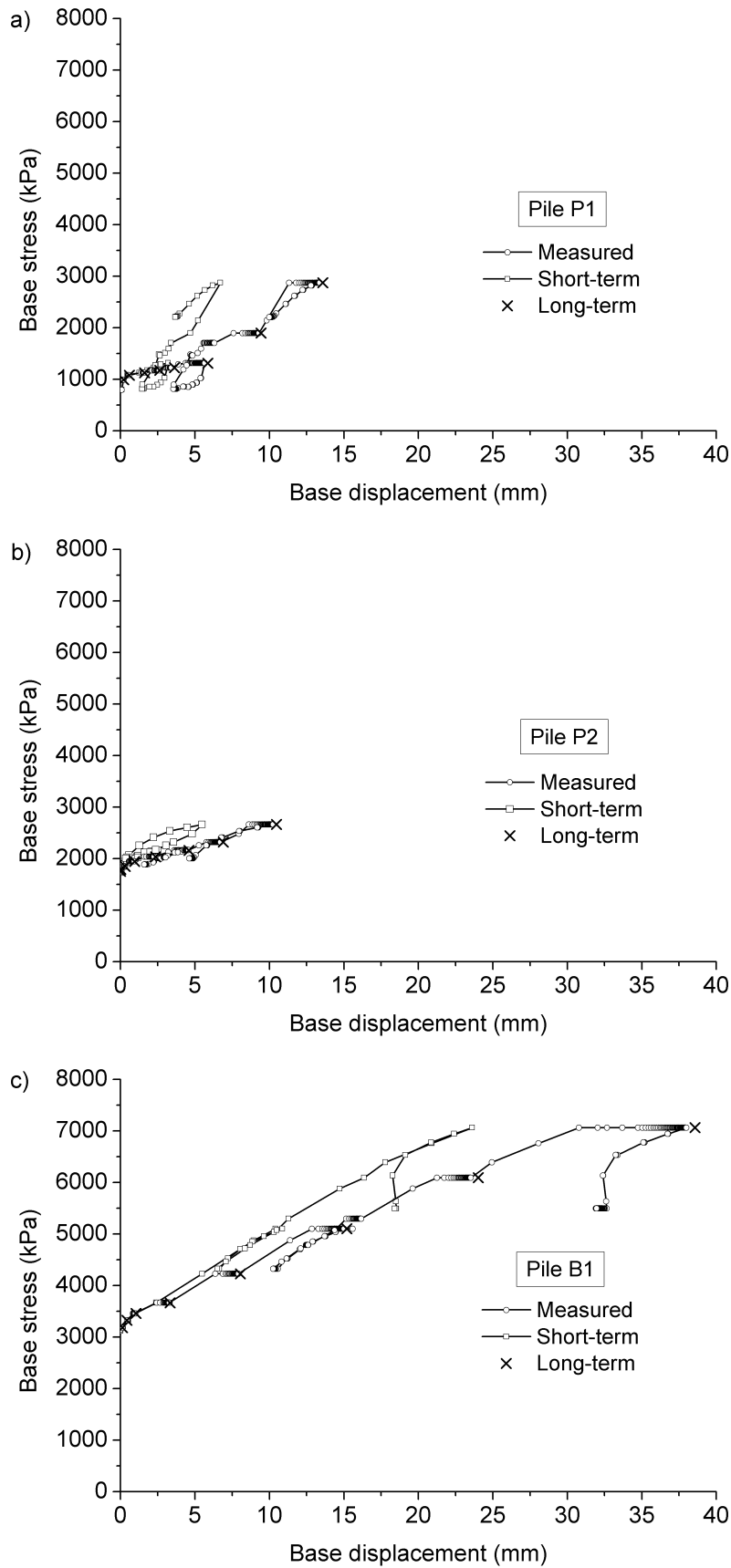


Figure 4.54: Mobilised base stress: *a*) polymer pile P1; *b*) polymer pile P2; *c*) bentonite pile B1.

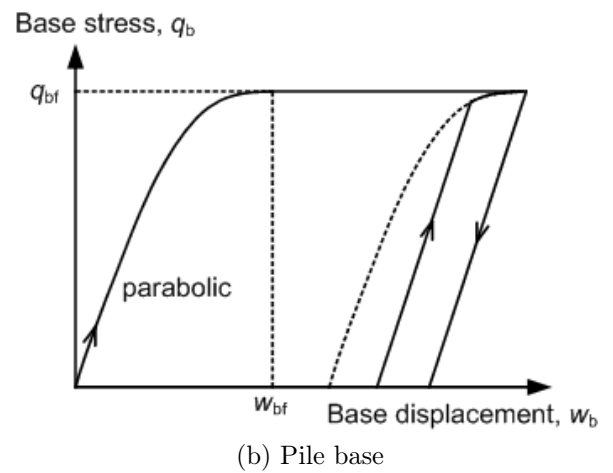
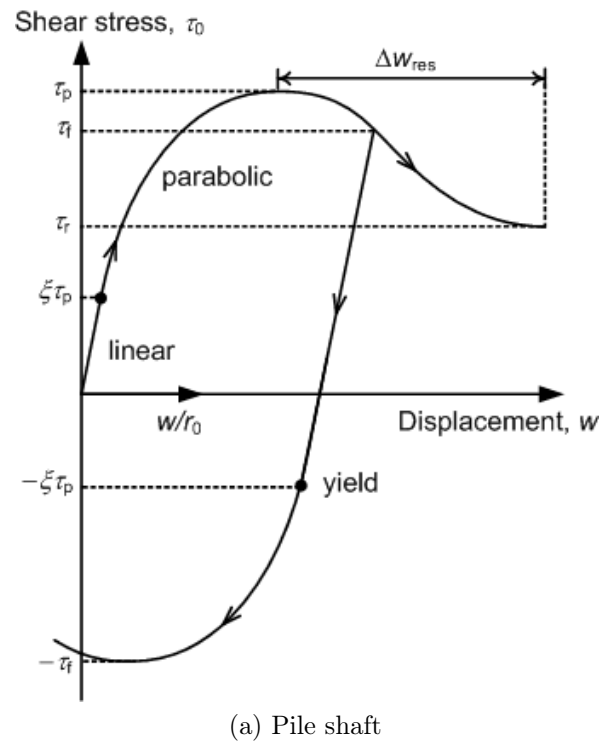
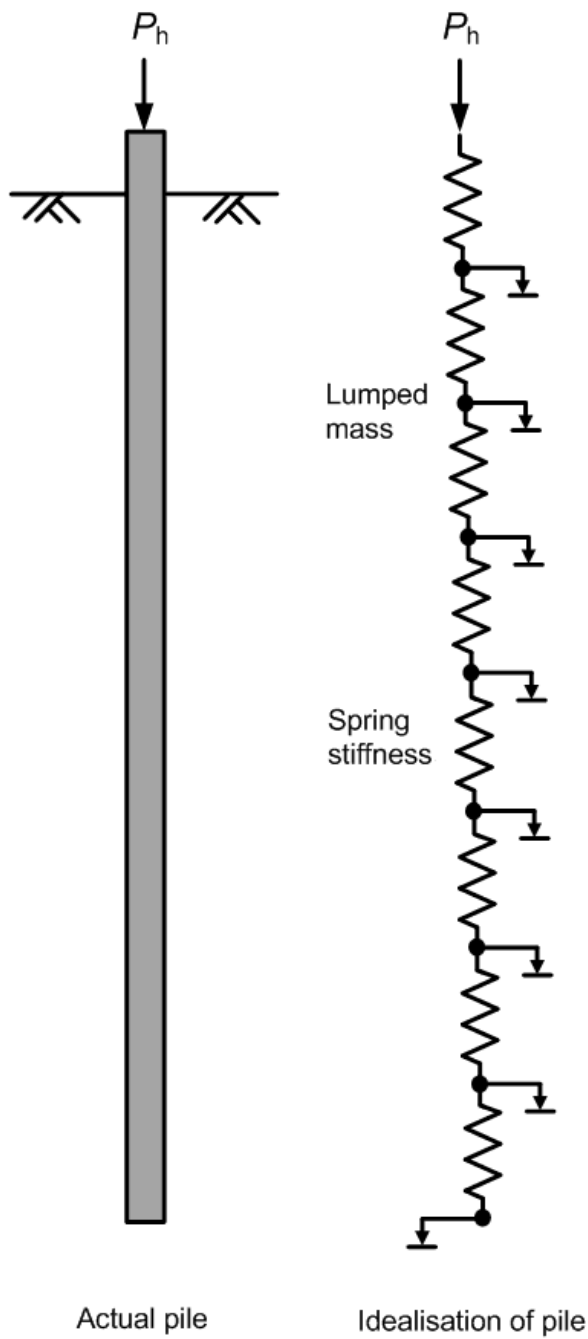


Figure 4.55: Idealisation of pile in load transfer analysis (after Randolph, 2003).

Figure 4.56: Load transfer curves: *a*) pile shaft; *b*) pile base (after Randolph, 2003).

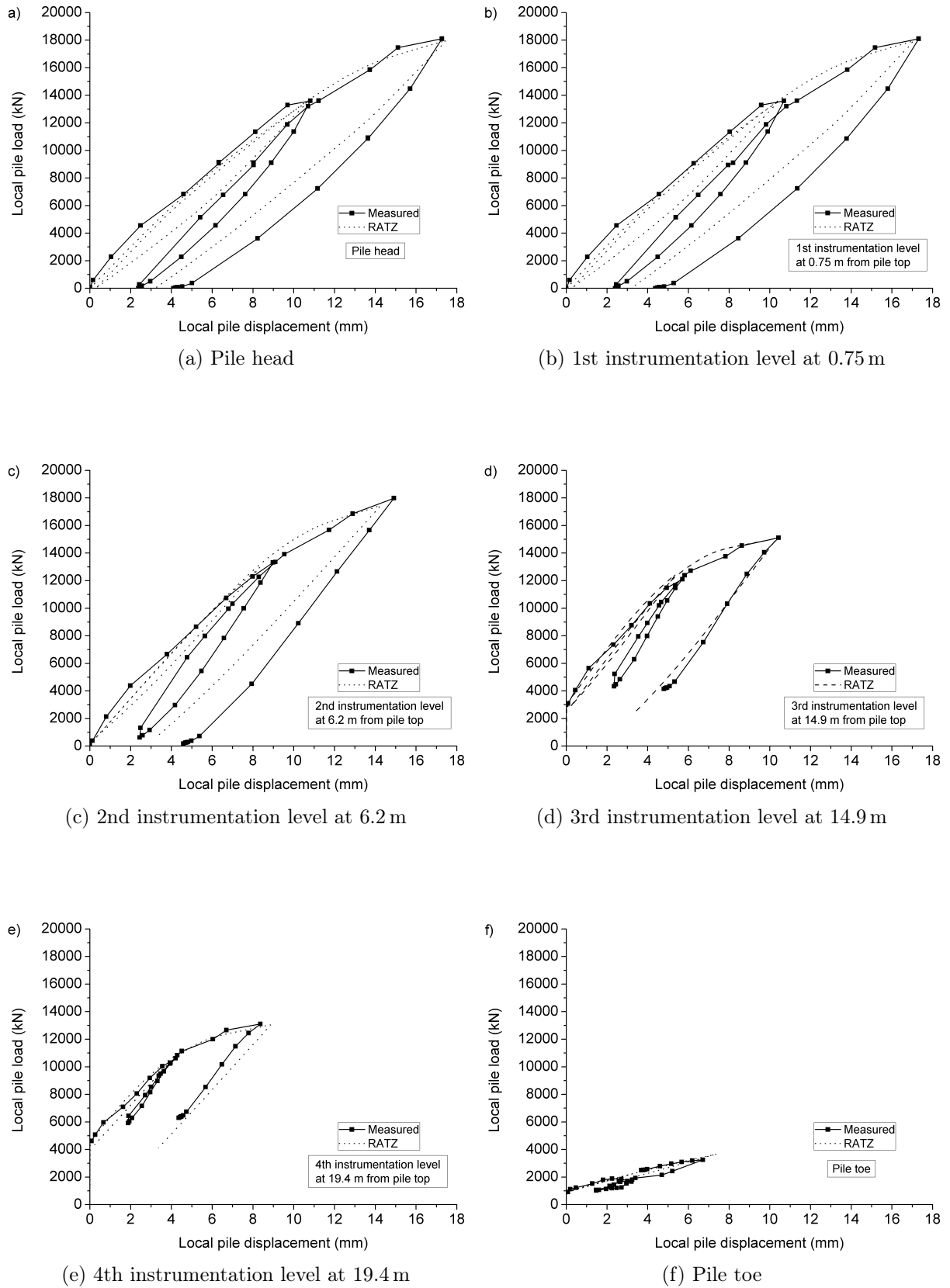


Figure 4.57: Measured and back-analysed local load–displacement response of pile P1.

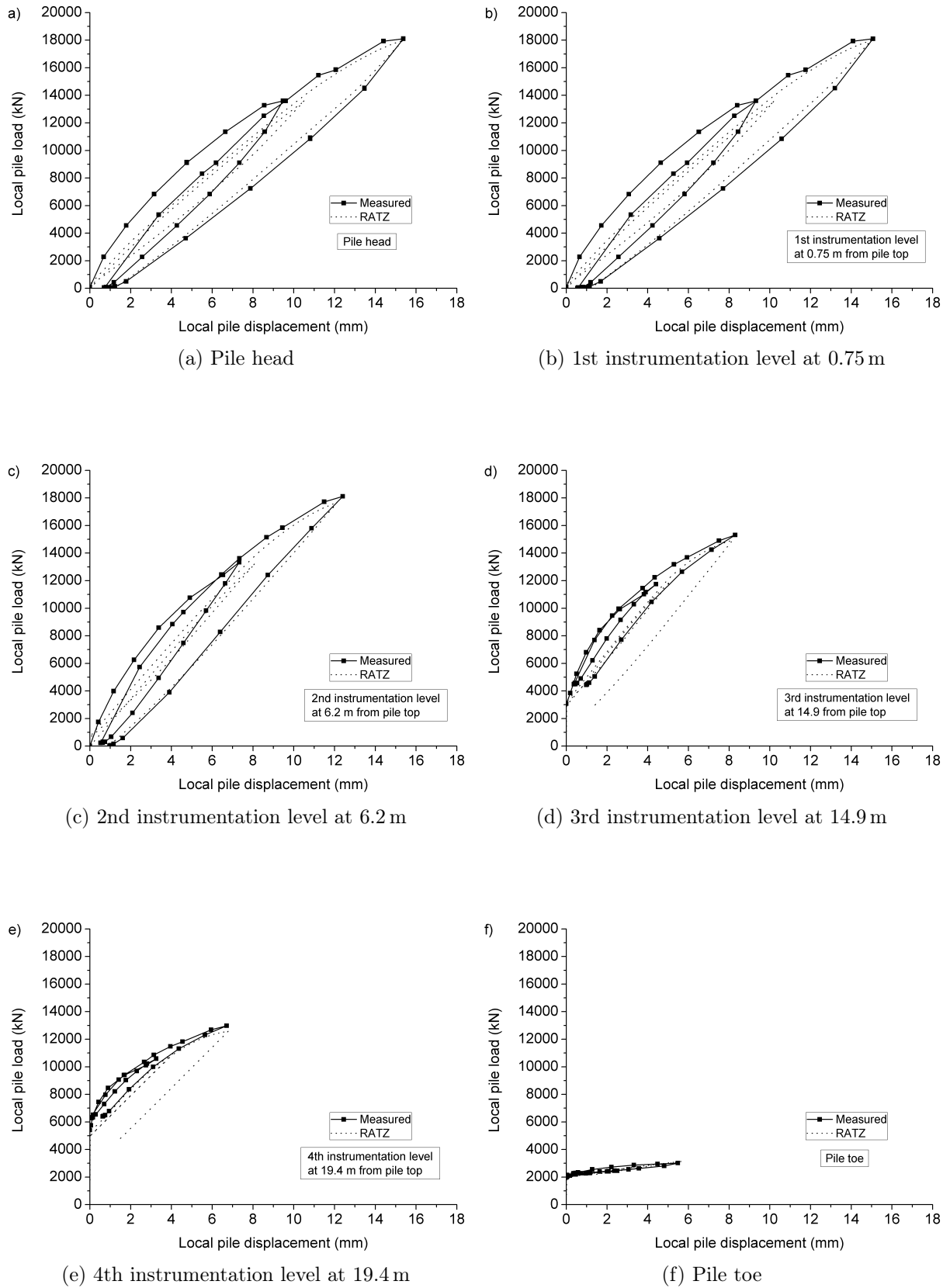


Figure 4.58: Measured and back-analysed local load–displacement response of pile P2.

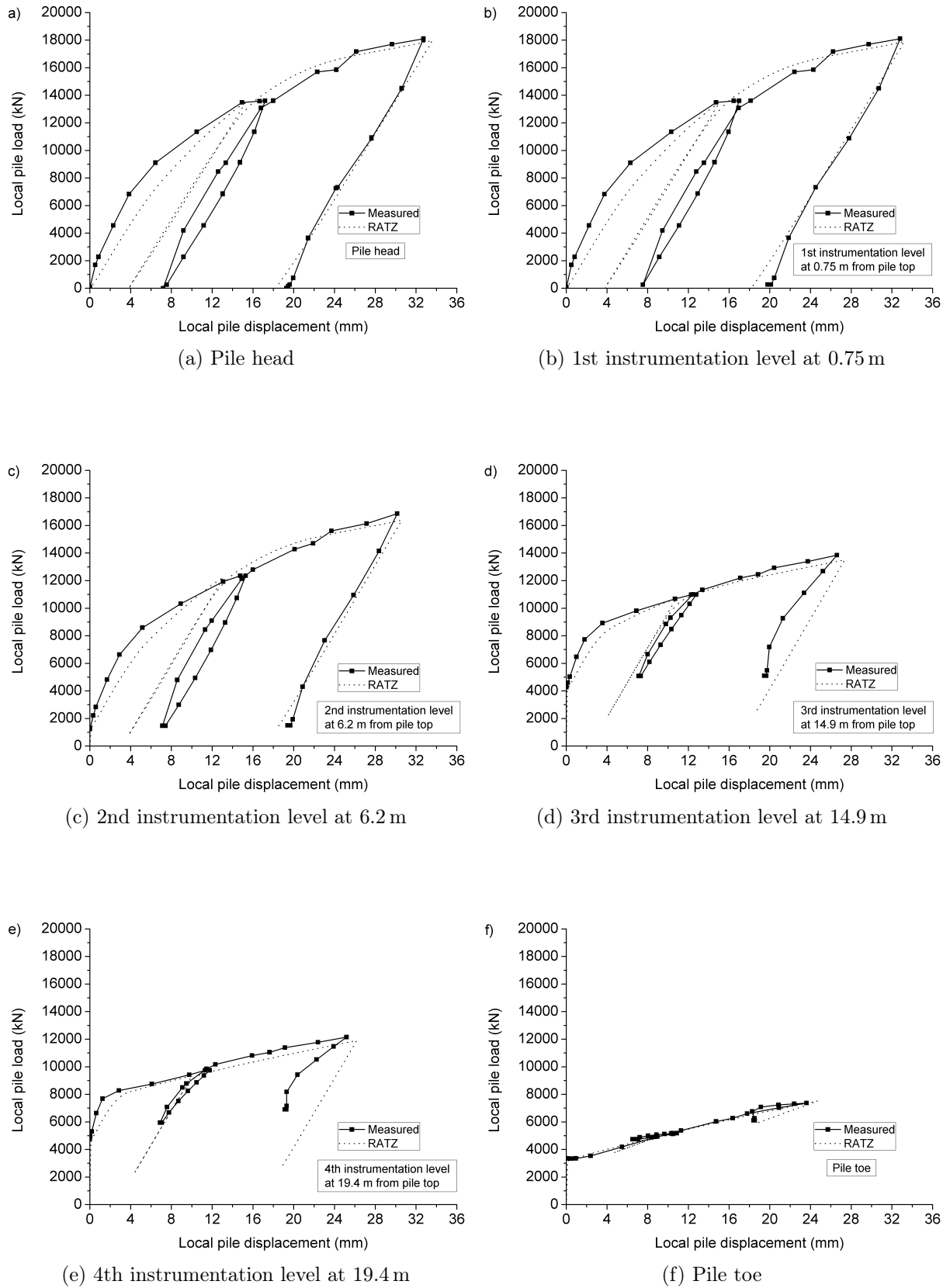


Figure 4.59: Measured and back-analysed local load–displacement response of pile B1.

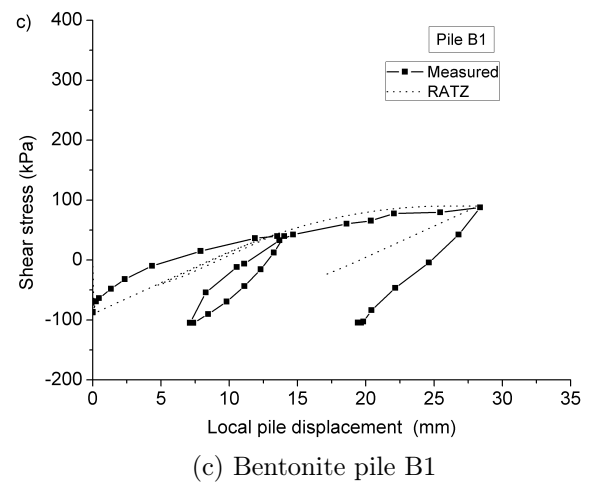
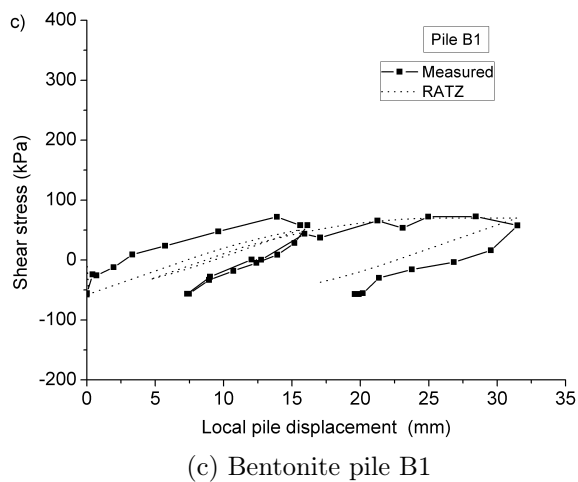
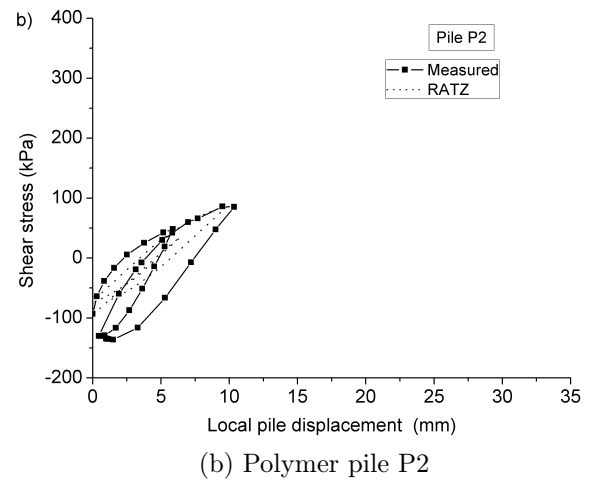
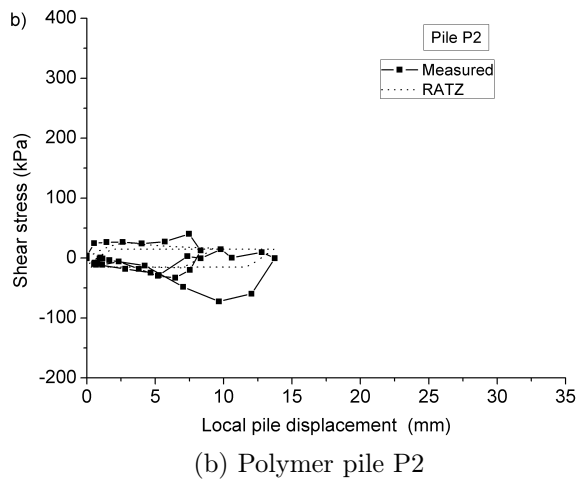
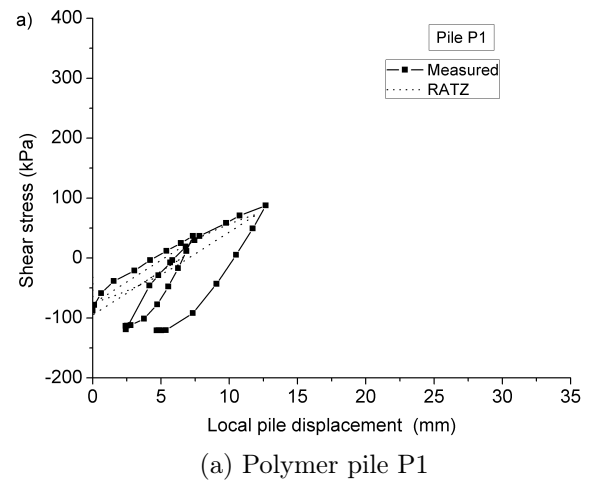
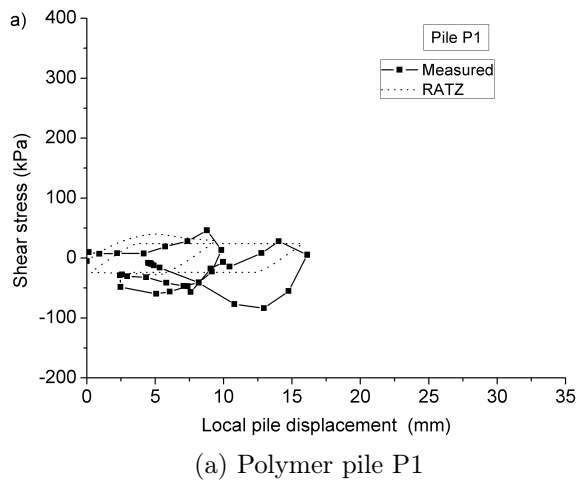


Figure 4.60: Measured and back-analysed mobilised shear stress in made ground.

Figure 4.61: Measured and back-analysed mobilised shear stress in Woolwich and Reading Formations.

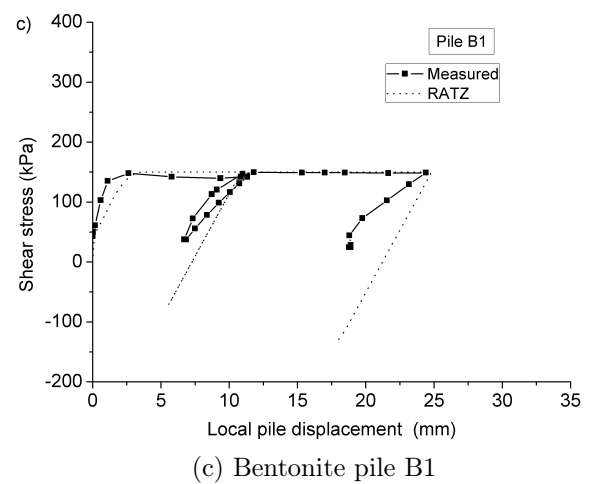
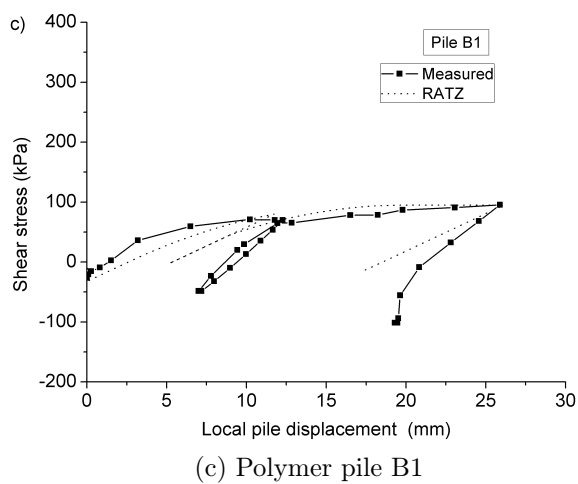
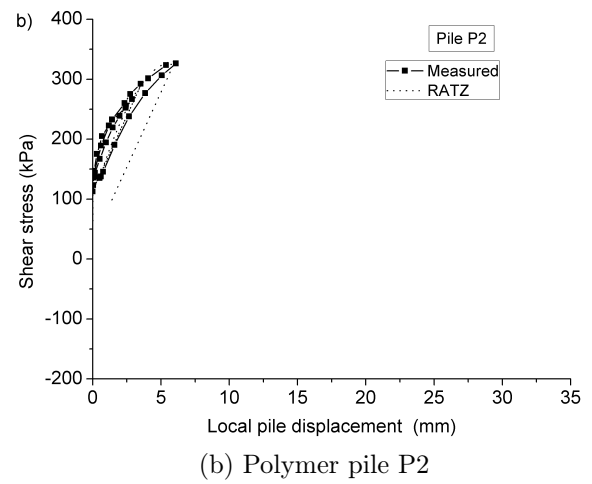
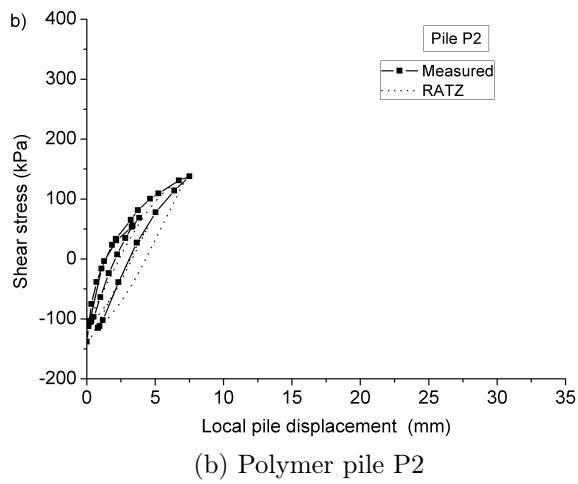
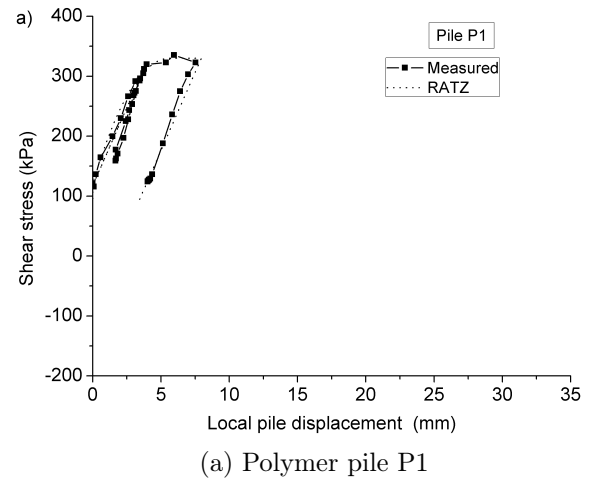
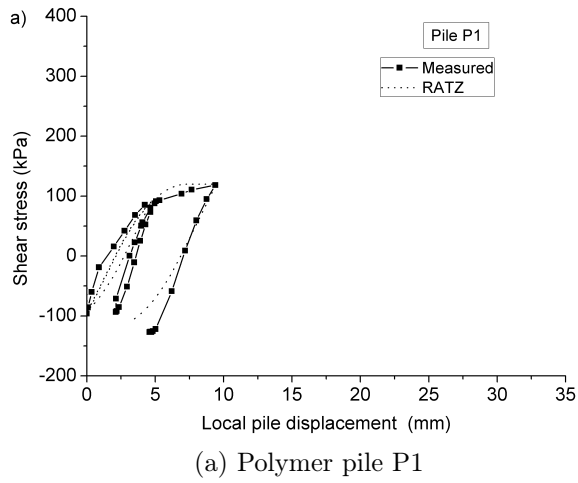


Figure 4.62: Measured and back-analysed mobilised shear stress in Upnor Formation.

Figure 4.63: Measured and back-analysed mobilised shear stress in Thanet Sand.

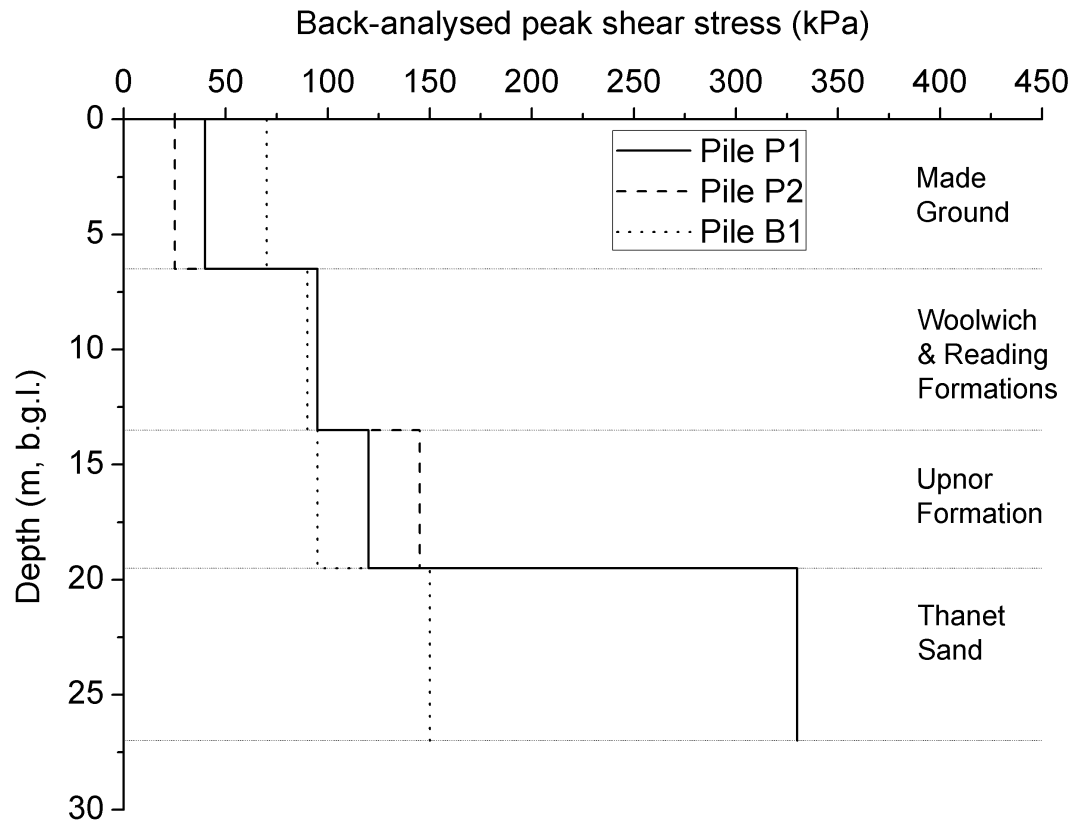


Figure 4.64: Back-analysed peak shear stress profile.

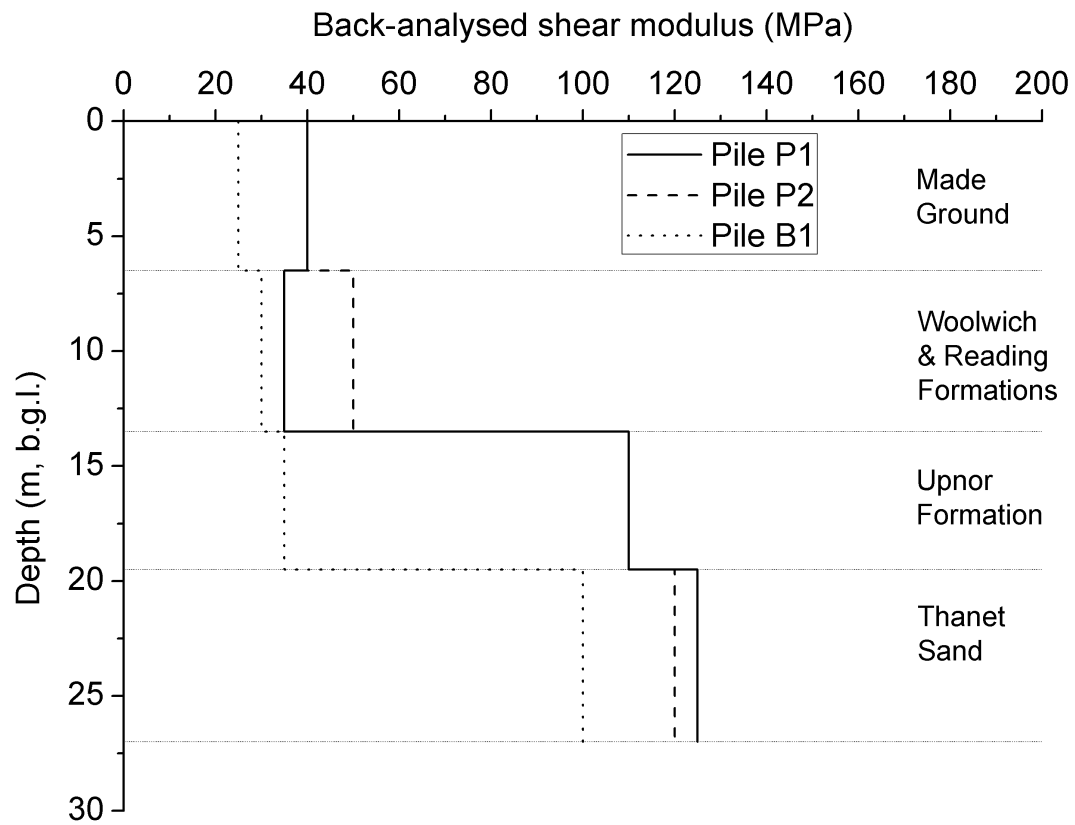


Figure 4.65: Back-analysed shear modulus profile.

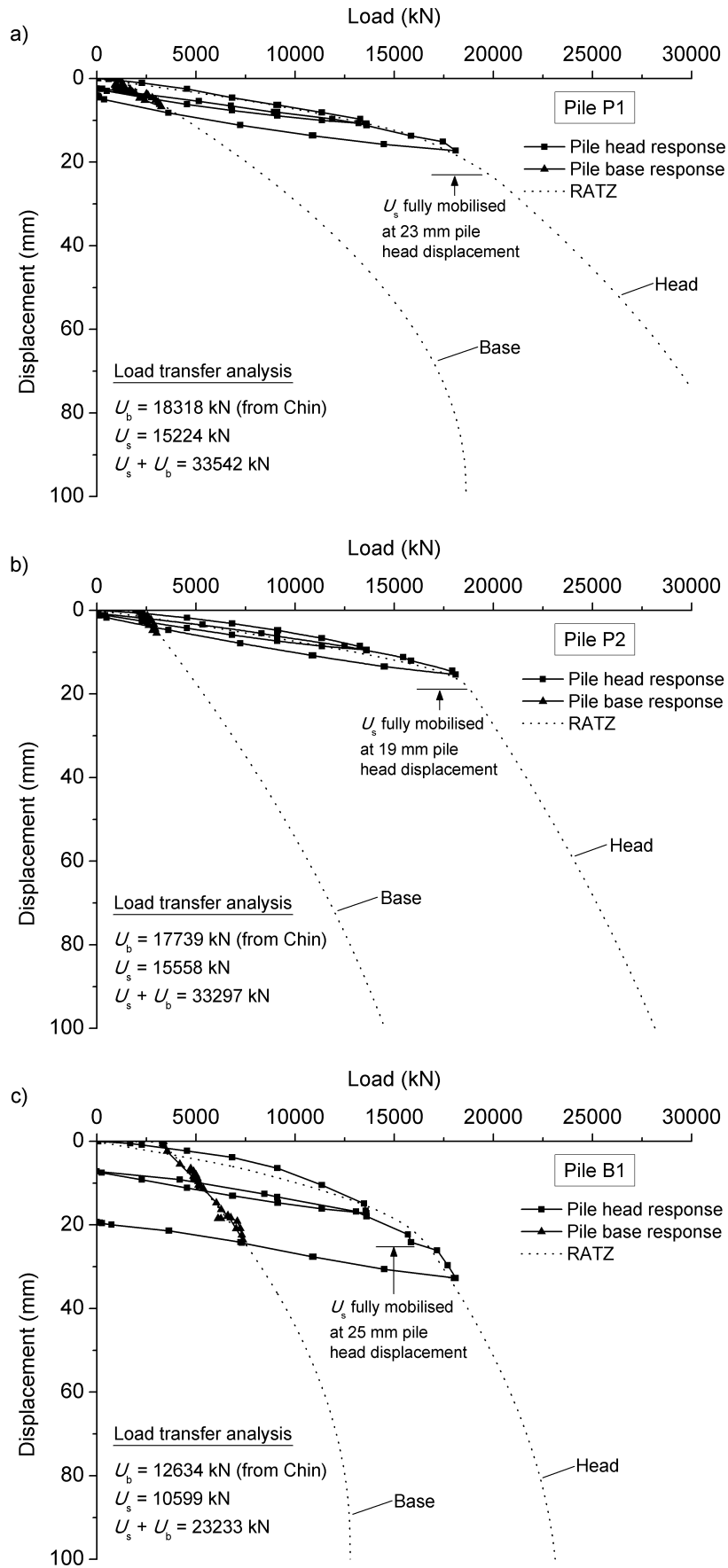


Figure 4.66: Measured and simulated response of test piles: a) polymer pile P1; b) polymer pile P2; c) bentonite pile B1.



Figure 5.1: Strengthening effect of HPAM on dry Leighton Buzzard sand.

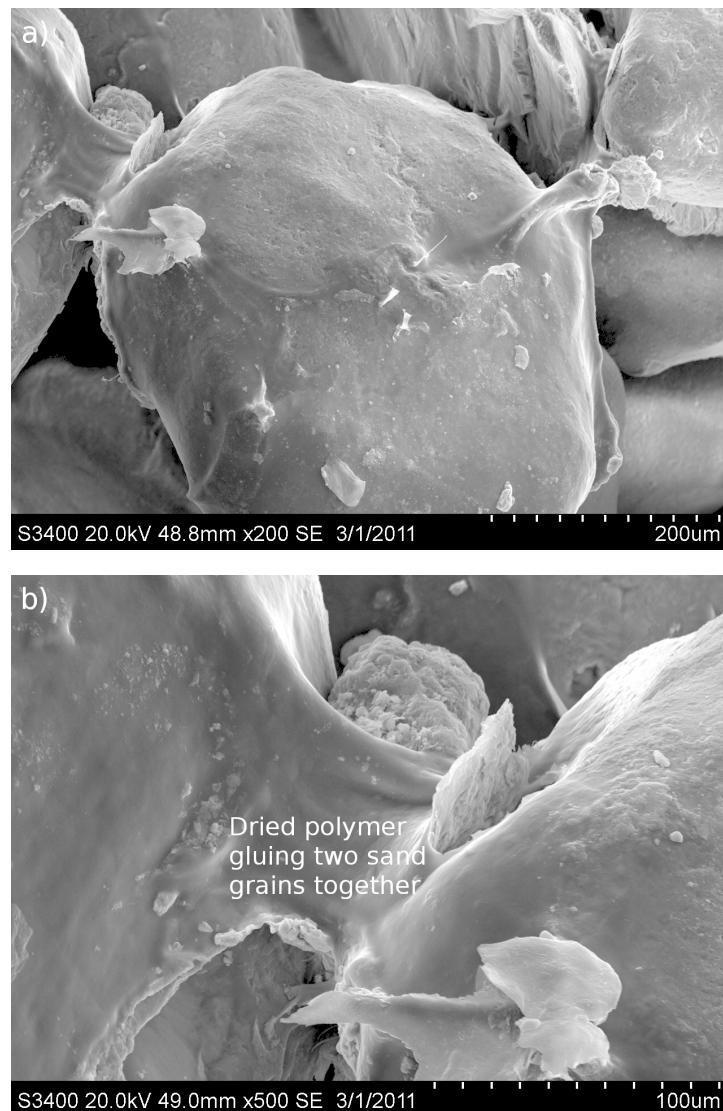


Figure 5.2: SEM micrographs of dry Leighton Buzzard sand previously saturated with HPAM polymer fluids: *a*) $\times 200$ magnification; *b*) $\times 500$ magnification.

Tables

Table 1.1: Required functions of an ideal polymer support fluid and the related fluid properties

Stage	Functions	Related chemical or physical fluid properties and underlying control mechanisms	Other non-fluid considerations
Excavation	Keep hole or trench open with little fluid loss	Rheological properties ^a – steady state and transient seepage Fluid density – hydrostatic pressure	Support fluid and groundwater levels; soil strength; salinity of pore water; choice of fluid additives; excavation tool
	Minimise soil softening at walls and base	Chemical reactivity (degree of hydrolysis/substitution) – polymer-clay sorption Polymer chain length (molecular weight) – particle encapsulation	Pile bore or trench open time; effect of other electrolytes
	Keep excavated material dry by preventing dispersion	Ditto	Excavation tool design
	Can be cleaned in situ to reduce the amount of suspended particles before concreting	Sand content; density – initial conditions Chemical reactivity – reaction with cleaning chemicals Rheological properties – particle-holding ability	Waiting time after in-hole treatment
Concreting	Can be easily displaced by concrete	Rheological properties – flow behaviour during displacement by concrete	Rate of concrete placement; embedment depth of tremie pipe
	Enable solid concrete – no inclusions	Ditto	Ditto; plus flow behaviour of fresh concrete; stability of side wall (no collapsing soil)
	Does not react with fresh concrete	Chemical reactivity – possible reaction with concrete	–
Fluid handling	Fluid performance can be maintained during the course of project	Density; sand content – initial conditions Rheological properties – particle-holding ability chemical reactivity – consumption rate of active polymer	Tankage; experience of site operatives; soil condition; excavation tool; in-service polymer dosage
	Does not contain hazardous chemicals	–	–
Loading	Can be broken down to water at end of project	Density; solid/sand content; pH – initial conditions Chemical reactivity – reaction with cleaning/oxidising chemicals	Choice of cleaning agent
	Does not reduce concrete bonding to reinforcement	Chemical reactivity – affinity with steel/rust and fresh concrete Rheological properties – flow behaviour during displacement by concrete	–
	Give clean concrete-soil interface on pile shaft (thin or no filter cake)	Density, sand content – physical blocking of soil pores on wall surface	–
	Give clean pile base (no ‘soft toe’)	Rheological properties – particle-holding ability controlling settling rate of particles	Base checking procedure; discharge rate of first load of concrete

^a The rheological properties of a fluid include its viscosity (shear and extensional), viscoelasticity, yield stress, etc. See Chapter 3 for details.

Table 2.1: Summary of commercial polymer products for foundation construction

Supplier	Application	Base polymer type	Base polymer (emulsion)	Base polymer (granules)	Mix water treatment	Membrane formation	Fluid loss control (mineral based)	Fluid loss control (polymer based)	Viscosity booster	Weighting / increased stability	Particle settlement acceleration	Cleaning before disposal	References
Baroid Industrial Drilling Products		HPAM	EZ-MUD Plus	POLY-BORE & EZ MUD DP	Soda ash	—	—	—	—	—	—	5% sodium hypochlorite solution	Baroid Industrial Drilling Products (2011)
	Piling & diaphragm walling	HPAM	Shore Pac liquid (replaced by Shore Pac XX-Poly)	Shore Pac (formerly Shore Pac GCV)	Soda ash, De-Chlor	—	Sand Sealant, Stone Stop	Shore Pac Sure Seal	Shore Pac liquid (replaced by Shore Pac XX-Poly)	Anchor-Load, Ice Pak, Shore Pac Stabilizer	Shore Pac Insta-Clear Dry	Slurry Buster or Slurry Buster Dry	CETCO Drilling Products (2011)
Geo-Tech Services		HPAM	—	Novagel	Sodium hydroxide or soda ash	—	Mineral Seal	—	—	Novadense & Soilplex	Sand Magnet	Sodium hypochlorite & acid	GeoTech Services (2011)
KB International (formerly KB Technologies)	Piling & diaphragm walling using grabs	Vinyl (modified HPAM)	—	SlurryPro CDP	ProTek 100	SlurryPro LA-1-D, ImPac & SlurryPro MPA	SandSeal MMS	KobbleBlok & SlurryPro MPA	—	WeightIT, InstaFreeze & InstaFreeze C2	SlurryPro MPA	Klean-Break or Calcium hypochlorite	Goodhue & Holmes (1995); O'Neill & Reese (1999); International (2011)
		Vinyl (modified HPAM)	—	SlurryPro CDP	—	—	EnhancIT 100 and EnhancIT 200	—	—	—	—	—	—
		SlurryShield PBS & SlurryShield FGP : base polymer and additives combined in a single package											
		SlurrySMART : base polymer and additives combined in a single package											
Polymer Drilling Systems	Diaphragm walling using hydromills	HydroCUT : base polymer and additives combined in a single package											
		HPAM	Super Mud	Super Mud Dry	Water Treat	—	—	Aquasorb	—	—	—	Quik Floc	5 % sodium hypochlorite or 3 % hydrogen peroxide solution
Süd-Chemie	Piling & diaphragm walling	HPAM	SC MUD L	SC MUD P	Soda ash	—	Bentonite	—	—	—	—	—	Süd-Chemie (2011)
		CMC	—	SC VIS HVP / GSTP	—	—	—	—	—	—	—	—	

Table 2.2: Summary of elemental analysis results by EDX

Polymer type	Product name	C (%)	N (%)	O (%)	Na (%)	Al (%)	Si (%)	S (%)	Cl (%)	K (%)	Ca (%)	SEM micrograph reference
Commercial dry HPAM	SlurryPro CDP	48.02	—	41.74	10.24	—	—	—	—	—	—	Figure 2.10
	Shore Pac	43.18	—	43.74	13.08	—	—	—	—	—	—	Figure 2.11
	POLY-BORE	45.02	—	44.91	10.07	—	—	—	—	—	—	Figure 2.12
	EZ MUD DP	45.99	—	44.73	9.28	—	—	—	—	—	—	Figure 2.13
Commercial emulsion HPAM ^a	Super Mud	53.54	—	42.82	3.64	—	—	—	—	—	—	—
	Shore Pac XX-Poly	47.42	—	41.65	10.93	—	—	—	—	—	—	Figure 2.22
	FloSet MUD VL2	49.48	—	40.15	10.37	—	—	—	—	—	—	—
Natural biodegradable polymer	SC GUM	41.79	—	58.21	—	—	—	—	—	—	—	Figure 2.16
	DUO-VIS PLUS NS	42.65	—	52.78	0.43	—	—	—	—	3.81	0.33	Figure 2.17
	Aqualon CMC 7H3SF	36.69	—	52.30	11.01	—	—	—	—	—	—	Figure 2.18
		27.23	—	21.33	18.19	—	0.59	7.38	25.28	—	—	Figure 2.26a
Blended polymer		25.8	—	33.15	13.25	—	3.29	10.14	14.37	—	—	Figure 2.26b
	SlurrySMART	25.79	—	47.51	1.8	—	14.79	—	10.12	—	—	Figure 2.26c
		37.45	—	50.27	1.67	—	10.07	—	0.53	—	—	Figure 2.26d
		35.38	—	48.37	3.96	—	10.53	—	1.75	—	—	Figure 2.26e
Blended additives		40.22	—	47.79	10.52	—	1.01	—	0.47	—	—	Figure 2.26f
		38.13	12.13	36.49	3.65	—	1.37	6.1	1.87	—	0.26	Figure 2.27a
	EnhancIt 100	37.76	—	55.58	2.62	0.25	3.78	—	—	—	—	Figure 2.27b
		39.30	—	43.35	4.48	0.41	9.46	—	3.0	—	—	Figure 2.27c
EnhancIt 200		48.21	5.5	9.67	10.7	—	—	—	25.61	—	0.3	Figure 2.28a
		61.86	3.62	10.11	0.66	—	—	—	23.74	—	—	Figure 2.28b
		46.04	15.24	27.03	0.8	—	—	—	10.7	—	0.19	Figure 2.28c

^a EDX was carried out on the precipitates of these emulsion polymers to avoid the influence of the mineral oil. See Section 2.5 for further details about the precipitation process.

Table 2.3: Summary of test results of four fundamental polymer properties: solid content; solubility; degree of hydrolysis; molecular weight.

Polymer type	Product name	Solid content (%) ^b	Amount of 'fish eyes' (g/g)	Degree of hydrolysis (%) ^b	Molecular weight ($\times 10^6$ g/mol) ^b
Commercial dry HPAM	SlurryPro CDP	89	3.2	38 (40) ^c	18 (20) ^c
	Shore Pac	90	3.4	39	22
	POLY-BORE	93	17.2	28	16
	EZ MUD DP	86	0	26 (30) ^d	12 (10) ^d
Commercial emulsion HPAM	Super Mud	33	0	13	3
	Shore Pac XX-Poly	53 (50) ^e	0	23	7
	FloSet MUD VL2 ^a	38	0	23	8
Dry HPAM standard ^a	HPAM standard no. 1	85	1.1	36 (40)	14 (> 10)
	HPAM standard no. 2	95	0	6 (10)	Used for calibration (0.2)
	HPAM standard no. 3	87	0	70 (70)	Used for calibration (0.2)
Dry PAM standard ^a	PAM standard no. 1	89	4.3	0 (0)	Used for calibration (18)
Natural polymer ^a	SC GUM	90	0	—	—
	DUO-VIS PLUS NS	89	0	—	—
	Aqualon CMC 7H3SF	93	0	—	—
Blended polymer	SlurrySMART	93	2.2	—	—

^a These products are not included in Table 2.1. Please refer to Appendix A for details about them.

^b Numbers in brackets are published or known/claimed values.

^c After Goodhue & Holmes (1995).

^d After Liao (1995).

^e After CETCO Drilling Products (2011).

Table 3.1: Summary of viscosity parameters of different types of polymers

Polymer type	Polymer concentration c_p (kg/m ³)	Product name	Zero-shear viscosity η_0 (cP)	Infinite-shear viscosity η_∞ (cP)	Flow behaviour index n (-)	Time constant λ (s)	Marsh funnel viscosity t_{MF} (s)
Commercial dry HPAM	0.8	SlurryPro CDP	12500	7	0.34	250	72
		Shore Pac	47000	7	0.26	520	94
		POLY-BORE	34000	8	0.29	470	102
		EZ MUD DP	7500	3	0.38	160	60
Dry HPAM standard	0.8	HPAM standard no. 1	11500	3	0.37	180	66
Emulsion HPAM	2.0	Super Mud	510	6	0.40	2.1	48
		Shore Pac XX-Poly	13200	4	0.39	150	70
		FloSet MUD VL2	4250	6	0.40	70	59
Natural polymer	3.0	DUO-VIS PLUS NS	156000	1	0.26	650	37
		Aqualon CMC 7H3SF	103	1	0.65	0.16	47
Blended polymer	2.0	SlurrySMART	15500	7	0.37	260	82

Table 3.2: Summary of viscosity parameters of a commercial HPAM at different concentrations

Polymer concentration c_p (kg/m ³)	Infinite-shear viscosity η_∞ (cP)	Zero-shear viscosity η_0 (cP)	Flow behaviour index n (-)	Time constant λ (s)	Critical shear rate $\dot{\gamma}_{\text{crit}} (= 1/\lambda)$ (s ⁻¹)	Marsh funnel viscosity t_{MF} (s)
0.2	2.2	1300	0.42	180	0.42	53
0.4	4	10100	0.32	360	0.32	68
0.6	5	19500	0.3	380	0.3	80
0.8	7	47000	0.26	520	0.26	94
1.0	8	75000	0.25	550	0.25	113
1.2	9	135000	0.24	580	0.24	125

Table 3.3: Recommended dosage of the commercial HPAM ‘Shore Pac’ for different soil conditions and the expected Marsh funnel viscosity values (from CETCO Drilling Products, 2011)

Soil type	Polymer concentration c_p (kg/m ³)	Marsh funnel viscosity t_{MF} (s)
Clay and Shale	0.4 to 0.5	35 to 50
Silt and fine to medium sand	0.5 to 0.6	45 to 55
Coarse sand to pea gravel	0.7 to 0.8	55 to 65
Gravels to cobbles	1.0 to 1.2	65 to 95

Table 3.4: Summary of viscoelastic parameters of a commercial HPAM from creep tests

Polymer concentration	Instantaneous compliance	Discrete compliances ^a									Equilibrium compliance	Steady-state viscosity (Burgers Model)	Steady-state viscosity (Ninomiya)
c_p (kg/m ³)	J_0 (Pa ⁻¹)	J_1	J_2	J_3	J_4	J_5	J_6	J_7	J_8	J_9	J_e (Pa ⁻¹)	η (cP)	η (cP)
0.2	0	0	0	5.612	39.11	0	109	0	2243	0	2396.72	3106	1442
0.4	0	0	1.095	5.921	0	9.132	16.94	105.8	52.86	0	191.75	9155	8543
0.6	0	0	2.373	1.944	0	6.923	11.86	47.82	94.49	0	165.41	23210	28660
0.8	0	0	2.471	0	1.059	5.384	9.526	35.02	21.04	0	74.50	49650	33970
1.0	0	0	1.711	0	0.721	0.7743	0.129	9.58	15.71	0	28.62	99140	67720
1.2	0	0	1.073	0	0.838	0	1.119	5.736	19.13	0	27.90	121500	101000

^a For the computation of the individual compliance, J_i , the retardation time, Λ_i , is calculated as logarithmically spaced over the creep test period. Therefore, for a creep test which lasts for 8000 s, $t_1 = 0.09384$ s, $t_2 = 0.3879$ s, $t_3 = 1.603$ s, $t_4 = 6.628$ s, $t_5 = 27.4$ s, $t_6 = 113.3$ s, $t_7 = 468.2$ s, $t_8 = 1935$ s, and $t_9 = 8000$ s.

Table 4.1: Summary of peak effective stress strength parameters of Lambeth Group after Hight *et al.* (2004)

Soil type	Effective cohesion c' (kPa) ^a	Peak angle of shearing resistance ϕ'_p ($^\circ$) ^a	Remarks
Woolwich Formation - Laminated beds	0	21.5	triaxial compression
Woolwich Formation - Lower Shelly Clay	0	24	triaxial compression – ϕ'_p reducing to 16°
Reading Formation - Upper and Lower Mottled Clays	10 (0)	28 (14.5)	triaxial compression – lower bound
	230	28	triaxial compression – upper bound
	50 (25)	26.5 (21)	direct shear box – U100 samples
	110	28	direct shear box – block samples
Upnor Formation	0	33	triaxial compression – ϕ'_p reducing with increasing clay content

^a Values in brackets correspond to samples with high plasticity ($I_p > 30\%$).

Table 4.2: Properties of polymer and bentonite fluids used for the Stratford field trial

Pile number	Fluid details and exposure time				Fluid test results ^b			
	Product name and supplier	Dosage (kg/m ³)	Total fluid–soil exposure time (h)	Time of fluid in pile bore during sampling (h)	pH (-)	Sand content (%)	Density (g/cm ³)	Marsh funnel viscosity (s)
P1	SlurryPro CDP	1.0 (CDP)	7.5	5	11.2/11.2/11.2	0/0.2/0.2 (1.3) ^c	1.00/1.02/1.03	70/56/56
P2	/ KB Int.	0.08 (LA-1-D) ^a	26	23	11.0/11.0/11.0	0/0.1/0.1 (1.4) ^c	1.00/1.02/1.02	69/50/50
B1	Berkbent 163 / Tolsa	40.0	7.5	3	10.5/10.3/10.3	0/6.8/10	1.02/1.10/1.15	34/40/41

Figure 4.24

^a Before the addition of CDP and LA-1-D, ProTek 100 (potassium hydroxide) was used to treat the mix water. At the end of the excavation, 10 L of 1% SlurryPro MPA solution were added to the base of the polymer-supported bores.

^b Fluid test results are given in this format: fresh fluid / used fluid from middle of pile bore / used fluid from bottom of pile bore.

^c Sand content results given in brackets were obtained from samples taken near the bottom of bores immediately after excavation but before the addition of 1% MPA solution.

Table 4.3: Summary of soil parameters used for the estimation of residual load in polymer pile P2

Soil	Depth (m, b.g.l.)	Bulk density ρ_b (Mg/m ³)	Undrained shear strength S_u (kPa)	Adhesion factor α (-)	Interface friction angle δ (°)	Coefficient of earth pressure for the shaft K_s (-)
Made Ground	0 to 6.5	1.85	—	—	30	0.5
Woolwich Reading Formations (Lambeth Group)	6.5 to 13.5	1.95 + 0.0231z ^a	67.5 + 18.64z ^a	0.6 ^b	26 ^c	1.15
Upnor Formation (Lambeth Group)	13.5 to 19.5		198.0 + 6.34z ^a	0.6 ^b	33	1.15
Thanet Sand	19.5 to 27.0		—	—	32.5 ^d	1.15

^a Depth in metres from the top of the relevant soil layer.

^b Estimates based on case histories reported in Hight *et al.* (2004).

^c Approximate average value of the ϕ'_p values from Table 4.1 considering the thickness of the various sub units.

^d See Figure 4.49.

Table 4.4: Summary of back-analysed soil parameters from load transfer analyses

Pile number	Soil layer	Pile shaft					Pile base		Other parameters	
		Shear modulus G (MPa)	Peak shear stress τ_p (kPa)	Yield parameter ξ (-)	Ratio of residual to peak shear stress τ_r/τ_p (kPa)	Displacement to residual Δu_{res} (mm)	Strain-softening parameter η (-)	Ultimate unit base resistance q_{bf} (kPa) ^b		Displacement to mobilise q_{bf} w_{bf} (mm)
Pile P1	Made Ground ^a	40	30	0	0.6	10	1	—	—	$\zeta = 4$ $r_0 = 0.63$ m (0 to 8 m b.g.l.) $r_0 = 0.60$ m (8 to 27 m b.g.l.) $E_p = 27.5$ GPa
	Woolwich and Reading Formations	35	95	0	1	—	—	—	—	
	Upnor Formation	110	120	0	1	—	—	—	—	
	Thanet Sand	125	330	0.7	1	—	—	16197	100	
	Made Ground ^a	40	25	0	0.6	10	1	—	—	
Pile P2	Woolwich and Reading Formations	50	95	0	1	—	—	—	—	
	Upnor Formation	110	145	0	1	—	—	—	—	
	Thanet Sand	120	330	0.7	1	—	—	15685	200	
	Made Ground ^a	20	70	0	0.6	10	1	—	—	
	Woolwich and Reading Formations	30	90	0	1	—	—	—	—	
Pile B1	Upnor Formation	35	95	0	1	—	—	—	—	
	Thanet Sand	100	150	0.7	1	—	—	11171	110	

^a The made ground was supported by a temporary casing during construction and had no direct contact with the support fluids (Figure 4.9). The parameters of this layer are only given here for completeness.

^b From Chin analyses.

Table 4.5: Summary of interpreted soil and design parameters based on the results of load transfer analyses

Basic information				Initial soil stiffness and local strain			Pile design parameters	
Soil layer	Pile number	Depth-averaged S_u (kPa) ^a	Depth-averaged σ'_v (kPa)	Rigidity index G/S_u (-)	Normalised stiffness $E_u/p'^b, c$ (-)	Approximate shear strain γ_0 (%) ^d	Adhesion factor α (-) ^e	Interface friction angle δ (°) ^f
Woolwich and Reading Formations	P1			263	667	0.035	0.71	35
	P2	133	118	376	953	0.011	0.71	35
	B1			226	572	0.045	0.68	34
Upnor Formation	P1			507	1120	0.009	0.55	25
	P2	217	221	507	1120	0.009	0.67	30
	B1			161	356	0.100	0.44	20
Thanet Sand	P1			—	—	0.008	—	32
	P2	—	455	—	—	0.008	—	32
	B1			—	—	? ^g	—	16

^a Based on the results given in Figure 4.8. $S_u \approx 67.5 + 18.64z$ for the Woolwich and Reading Formations; $S_u \approx 198 + 6.34z$ for the Upnor Formation, where z is the depth in metre from the top of the relevant soil layer.

^b $E_u = 2(1 + \nu_u)G = 3G$, where $\nu_u = 0.5$ and the soil is assumed to be ideal isotropic porous elastic material.

^c $p' = \sigma'_v(1 + 2K_0)/3$, where K_0 is assumed to be 1.50 for Lambeth Group and 1.15 for Thanet Sand.

^d For the Woolwich and Reading Formations, γ_0 was estimated using the stiffness degradation curves presented in Linney & Page (1996). As no information can be found about the stiffness degradation characteristics of the Upnor Formation, the curves of Linney & Page (1996) were also used for the purpose of estimating γ_0 in the Upnor Formation. For the Thanet Sand, γ_0 was estimated using the approximate G_0 value of 150 MPa and the stiffness degradation trend given in Ventouras (2005).

^e $\tau_p = \alpha S_u$

^f $\tau_p = \sigma'_v \tan \delta = K_s \sigma'_v \tan \delta$, where K_s is taken as 1.15 for all soil layers.

^g As discussed in Section 4.6.6, when bentonite fluids were used the interface shearing resistance in the Thanet Sand was governed by the consolidated filter cake at the pile-sand interface. Therefore, no estimation of the shear strain in the Thanet Sand was made.

Table 5.1: Some pros and cons of bentonite and polymer fluids

Bentonite fluid – dispersion (suspension) of sodium or calcium bentonite clay
Pros
• Proven technology with a wealth of past experience; • Low material cost per unit volume of fluid; • Can become a gel or semi-solid if left undisturbed. This allows bentonite fluids to stabilise very coarse soil by rheological blocking and also to powerfully suspend dislodged soil particles up to coarse sand size; • Can form filter cakes on the surface of coarse soil thus creating a low-permeability membrane; reducing fluid loss with time; • Less sensitive to mix water quality than some polymers; once mixed bentonite is a stable fluid system.
Cons
• Natural material with a depleting resource; • Higher dosage required; • Require bulky ancillary plant for mixing and cleaning; • Used fluids may become difficult to clean due to accumulation of fines; • Used fluids are classified as liquid wastes in the UK and need to be treated by specialist waste contractor; • Cannot be used in environmentally sensitive areas as it can cause harm to the aquatic life; • Filter cakes formed on the soil surface can significantly reduce the resistance of piles.
Polymer fluid – solution of water-soluble polymers which are usually of the high molecular-weight type
Pros
• Man made material with controllable properties; • Lower dosage required; • Smaller site footprint as it does not require bulky mixing equipment; • Used fluids can be treated on site; treated fluids may be directly discharged into sewer with water undertaker's consent leading to significantly reduced disposal cost; • More environmentally friendly as it is solution based; • Can reduce the swelling and slaking of water-sensitive shale; • Can give a cleaner concrete-soil interface and hence higher shaft resistance.
Cons
• Modern technology; some countries and companies have little or no previous experience; • High material cost per unit volume of fluid; • Does not become a gel or form filter cakes on the surface of coarse soil; poor fluid loss control if no additives are used; • More prone to damage caused by ionic species in water and by pumping equipment; • Polymer is an active system and sorbs onto soil; hence replenishment is necessary if the fluid is to be re-used repeatedly; • Cannot hold coarse particles in suspension for long time; hence a higher chance of causing the 'soft toe' problem.

Appendix A

List of Polymer Products and Their Published Chemical Information

Baroid Industrial Drilling Products

EZ-MUD PLUS (liquid) emulsion HPAM

POLY-BORE dry solid HPAM

CETCO Construction Drilling Products

Anchor Load (liquid) a heavy liquid containing 38% dissolved solids, possibly sodium silicate (now discontinued)

De-Chlor inorganic salt

Ice-Pak (liquid) unknown, to be used with Anchor-Load to form a soft grout (now discontinued)

Sand Sealant mineral clay

Shore Pac formerly known as Shore Pac GCV, copolymer of sodium acrylate and acrylamide (dry solid HPAM)

Shore Pac Liquid (liquid) emulsion HPAM (now replaced by Shore Pac XX-Poly)

Shore Pac Insta-Clear Dry dry blend of clay, inorganic salts and organic polymers

Shore Pac Stabilizer (liquid) formerly known as Stabilite according to the supplier's old literature, a highly charged cationic polyelectrolyte solution

Shore Pac Sure Seal formerly known as Macro-Fill according to the supplier's old literature, lightly crosslinked sodium polyacrylate. Note that this chemical is commonly used as a superabsorbent polymer in consumer products such as baby diaper.

Shore Pac XX Poly (liquid) copolymer of sodium acrylate and acrylamide in mineral oil (emulsion HPAM)

Slurry Buster (liquid) sodium hypochlorite solution

Slurry Buster Dry calcium hypochlorite, calcium chlorate, calcium hydroxide, calcium acid and calcium salt (1:1)

Stone Stop bentonite clay

Geo-Tech Services

Mineral Seal undisclosed natural material

Novadense (liquid) a mixture of Novagel, consistency modifiers and Mineral Seal.

Novagel acrylate/acrylamide copolymer (dry solid HPAM but advertised as a vinyl polymer)

Sand Magnet (liquid) proprietary

Soilplex (liquid) two-component product consisting of Soilplex 1 and Soilplex 2, chemical information undisclosed

KB International (formerly KB Technologies)

CDP modified HPAM marketed as ‘Vinyl’ polymer

EnhancIT 100 trade secret, carboxylated vinyl polymer

EnhancIT 200 trade secret, carboxylated vinyl polymer

HydroCUT trade secret, hydrophobically modified polymer, vinyl polymer

ImPac polyacrylamide

InstaFreeze (liquid) blend of alkaline salts, sodium hydroxide

InstaFreeze C2 (liquid) to be used with InstaFreeze to form soft grout, chemical information undisclosed (now discontinued)

KleanBreak calcium-based oxidizing agent

KobbleBlok proprietary polymer

ProTek 100 (liquid) potassium hydroxide (caustic potash)

SandSeal MMS natural kaolinite/silica mineral

SlurryPro LA-1 (liquid) emulsion form of SlurryPro LA-1-D (now discontinued)

SlurryPro LA-1-D dry vinyl polymer

SlurryPro MPA (liquid) proprietary polymer

SlurryShield FGP proprietary vinyl polymer

SlurryShield PBS proprietary vinyl polymer

SlurrySMART trade secret, hydrophobically modified polymer, polyacrylamide

WeightIT (liquid) blend of alkaline salts

Polymer Drilling Systems (PDSCo)

Aquasorb anionic polyacrylamide

Quik Flocc (liquid) nonionic polyacrylamide in water-in-oil emulsion

Super Mud (liquid) anionic polyacrylamide copolymer in water-in-oil emulsion

Super Mud Dry anionic polyacrylamide copolymer

Water Treat sodium carbonate (soda ash)

Süd-Chemie

SC MUD L (liquid) anionic polyacrylamide (emulsion HPAM)

SC MUD P anionic polyacrylamide (dry solid HPAM)

SC VIS HVP / GSTP sodium carboxymethyl cellulose (natural)

The following products are not listed in Table 2.1 but appear elsewhere in the thesis

Aqualon CMC 7H3SF sodium carboxymethyl cellulose (natural) supplied by Ashland Inc., formerly Hercules Inc. The classification code '7H3SF' represents a degree of substitution of about 0.7, high-viscosity type, an upper viscosity limit of about 3000 mPa·s for a 1% solution, smooth solution characteristics, and food grade.

DUO-VIS PLUS NS xanthan gum (natural) supplied by M-I SWACO Norge AS. Unlike many other commercial xanthan gums this product is not coated with glyoxal for improved dispersibility/solubility

FloSet MUD VL2 (liquid) anionic polyacrylamide (emulsion HPAM) supplied by SNF(UK) Ltd (SNF Inc, 2011)

Geomud-15 Unknown, possibly a high-molecular-weight HPAM

HPAM standards dry solid HPAM supplied by Polysciences Inc.

PAM standard dry solid unhydrolysed PAM supplied by Polysciences Inc.

SC GUM guar gum (natural biodegradable) supplied by Süd-Chemie via Bentofrance