THE UNIVERSITY OF NOTTINGHAM

DEPARTMENT OF MINERAL RESOURCES ENGINEERING

THE EXAMINATION AND PREDICTION OF OPENCAST BACKFILL SETTLEMENT

by

Christopher W.W. Hills
Bsc., Msc.

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ABSTRACT

With ever increasing demands on land use large areas of land are being considered for development which due to past or present mining activities are covered by substantial depths of fill materials. The problem faced with the development of land infilled with opencast mining backfill is in predicting the behaviour of the fill upon development. To be able to design foundations suitable to withstand the movements that occur within opencast backfill or design a backfilling operation that produces land suitable for a proposed after development, a means of predicting backfill settlement is required.

From the analysis of a considerable quantity of data collected from the monitoring of backfilling operations and backfill movement at a range of opencast coal sites located within the UK, the behaviour of opencast backfill has been examined and better understood. This information has enabled a method of predicting backfill settlement to be developed which has been subsequently implemented as a computer program running under the Windows operating system.

Factors taken into consideration during the prediction process are the timing of backfilling operations, the compactive state of the backfill, the inundation of the backfill and the influence of surrounding material and a means of predicting differential settlement due to backfill heterogeneity is proposed. Examples are given demonstrating the significance of these factors upon settlement predictions made at a hypothetical site. Finally, a comparison is made between predicted settlements and those monitored at an actual site to demonstrate the validity of the method proposed.

AFFIRMATION

The work submitted in this thesis is my own work and has not been previously submitted for any other degree.

The following publication has been based in this research:

Hills C.W.W. & Denby B. (1995)
The prediction of opencast backfill settlement.
In Preperation.

It is anticipated that other publications will be shortly forthcoming based on the research covered in this thesis.

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INTRODUCTION

1.1 General Introduction

A consequence of opencast coal mining in the UK is the production of large areas, typically up to 350 hectares, of backfilled land. Traditionally, backfill placement was simply carried out by the machinery used for the excavation of the overburden; draglines casting overburden directly into the dump area, truck-transported spoil end-tipped over a loose-wall or scraper-transported spoil being placed in layers at various levels. The result of this approach is large volumes of backfill having varying degrees of compaction dependant upon the machinery and technique used. For agricultural after use, drainage considerations apart, this is of limited consequence, but with the growing pressures on land use such sites are being considered more and more, following restoration, for development purposes sensitive to ground movements.

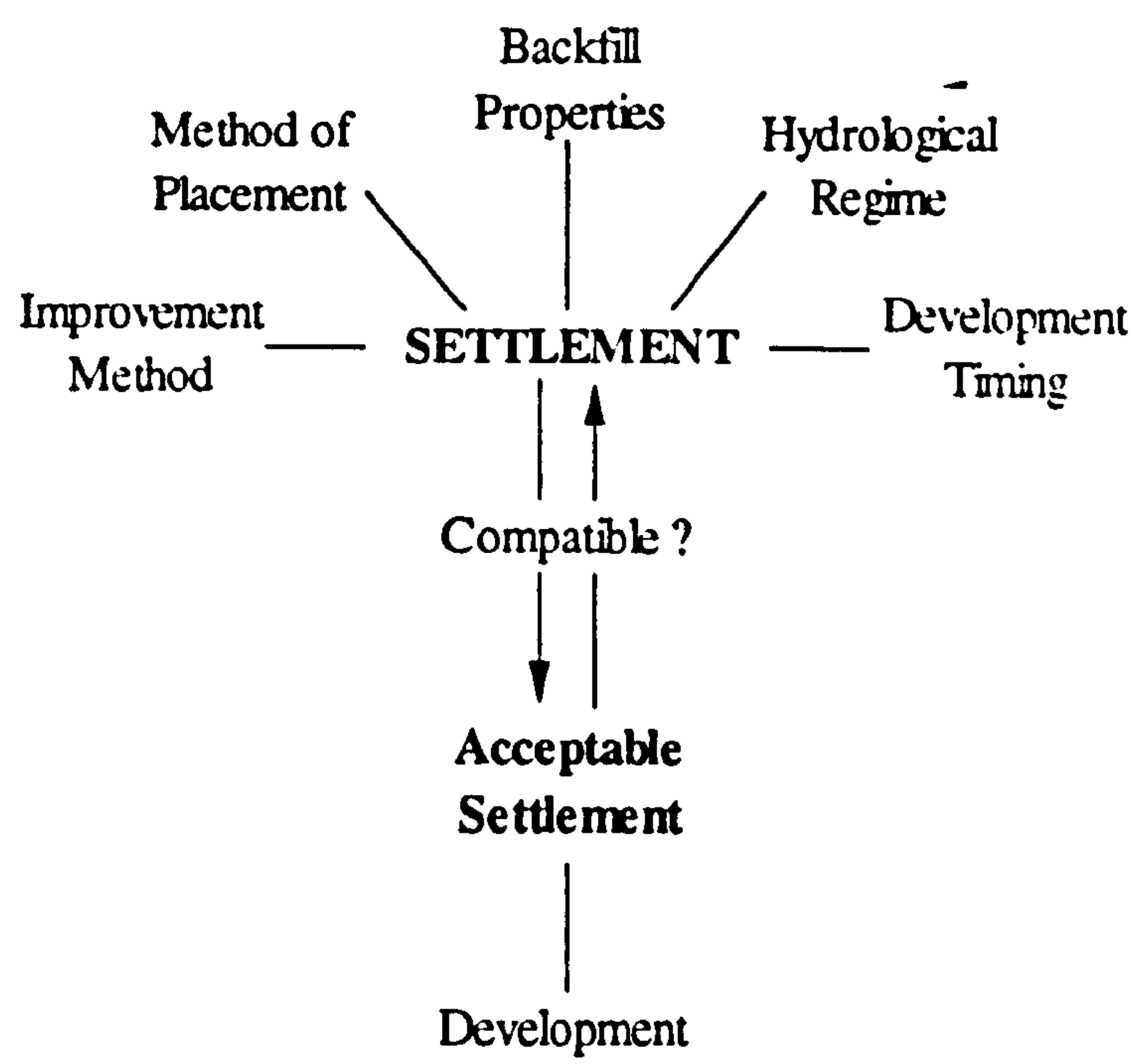


Figure 1.1. Representation of the factors influencing opencast backfill settlement.

The construction of roads, housing or light structural development on opencast backfill encounters several geotechnical problems, but the major one is that of settlement. This has led to methods of controlled backfill placement being adopted when subsequent structural development of a site is required. The controlled methods carried out currently are that of compaction in accordance with specifications based on either the Department of Transport (DTp) Specification for Road and Bridgeworks, 1976 or the more recent DTp Specification for Highway Works, 1986.

To design a controlled method of backfill placement best suited to the after use of a site, a thorough knowledge of the mechanics involved in backfill settlement is required; settlement must be understood in terms of the backfill placement method, constituent material properties, hydrological regime and development timing. The problem is demonstrated in Figure 1.1 which shows the factors that influence the settlement of a backfill and how a particular development defines acceptable settlement limits within which a backfill must lie for site development to proceed. Factors that are open to manipulation are those of the method of placement, any improvement methods employed and in certain situations the type of development. The problem of opencast backfill development is therefore one of designing a backfilling operation that is compatible with the given development or a development that is compatible with the settlement characteristics of a given backfill.

To enable this compatibility to be achieved a means of predicting the settlement of a backfill is required. It is the aim of this work, through the examination of the settlement behaviour and characteristics of both compacted and uncompacted backfill and also those of earth/rockfill embankments, to recommend a method by which backfill settlement can be calculated.

Information examined by this work includes a large amount of compaction, settlement and groundwater monitoring data from 11 British Coal opencast sites located within the UK. This data was made available to the author during his involvement in the report, 'A State of the Art Review of the Compaction of Opencast Backfill' (SARCOB 1993) which was produced for British Coal Opencast by Scott Wilson Kirkpatrick Limited, Consulting Engineers in collaboration with Nottingham University, Department of Mineral Resources Engineering.

In this work the expression "uncompacted" will be used to describe fills that have not been placed and systematically compacted by a scheme of controlled backfilling. However, depending on the method of excavation and backfilling, most backfills will have received some compaction.

1.2 Research Objectives

The research objectives of this work can be summarised as below:

- To better understand the mechanics by which opencast backfills settle.

- To establish parameters that can be used to define the settlement characteristics of a backfill.
- To determine relationships between these parameters and methods of backfill placement.
- To examine the way in which backfill settlement is influenced by the hydrological regime.
- To develop a method by which opencast backfill settlement can be predicted dependant upon the many parameters that define both a given backfill and backfilling operation.
- To implement this method through the development of a PC based computer program.
- To demonstrate the validity of the proposed predictive method through a comparison between predicted and actual settlements at an opencast backfill case study site.

1.3 Synopsis of Thesis

Chapter 2 : describes the behaviour and properties of opencast backfill as well as the performance of earth and rockfill embankments due to their comparable nature. The information given comes from the findings of a review of relevant literature.

Chapter 3 : summarises and gives an analysis of the large body of compaction, settlement and groundwater monitoring data made available to the author, providing a better insight into the behaviour of opencast backfills. The data comes from 11 British Coal opencast sites located within the UK.

Chapter 4 : examines methods by which backfill can be placed in a controlled manner and methods available for improving the performance of a backfill upon placement. The controlled methods of placement described are those recommended by SWK Limited which are detailed within SARCOB 1993. The information from the previous chapters enables these methods to be examined in terms of the settlement characteristics of the backfill upon placement or improvement. Forms of development, typical to opencast backfill restoration projects, are also examined to give an indication of acceptable settlement characteristics that the backfilled land must comply with for development to proceed.

Chapter 5 : reviews presently available methods by which backfill settlement may be predicted followed by an outline of the method proposed by the author. The way in which this method is implemented through the development of a PC based computer program known as OBSett (Opencast Backfill Settlement Prediction Package) is also discussed.

Chapter 6 : summarises the results generated during the calibration and testing of the developed settlement prediction program, OBSett. Calibration ensured that settlement predictions were within realistic limits. Testing gave a degree of validation to the method proposed by demonstrating that predictions made for a range of different backfilling scenarios where representative in terms of the observations made in Chapters 2 and 3.

Chapter 7 : further validates the proposed method of settlement prediction by making a comparison between predicted and actual monitored settlements at the case study site, Site A.

Chapter 8 : provides the conclusions reached by this work and points to areas in which additional research would be beneficial in furthering understanding in this field.

OPENCAST BACKFILL BEHAVIOUR AND PROPERTIES

2.1 Introduction

To be able to design for and build upon opencast backfill a thorough understanding of its behaviour is required. Factors determining opencast backfill behaviour include the properties and packing state of the constituent fill material, the shape of a backfill as a structure in its own right, the susceptibility of a backfill to becoming saturated from either surface water infiltration or a rising groundwater table and the presence of external structures such as overburden mounds and settlement ponds. This chapter discusses these points as found from an extensive review of the literature that has been published on the performance of backfills which have been restored by controlled compaction as well as those placed without specific compaction. Information was also acquired on the performance of earth and rockfill embankments because of their comparable nature.

2.2 Typical UK Opencast Backfill

Opencast coal mining operations in the United Kingdom work exclusively with strata belonging to the Carboniferous Coal Measures. Most opencast coal sites occur in outcrop regions which may or may not be concealed by younger rocks or superficial deposits. The rock types intervening the seams are usually mudstones, siltstones, seatearths and sandstones which are present in different thicknesses from locality to locality and in some instances may be completely absent from the stratification sequence (Hassani *et al* 1979). Superficial deposits generally consist of boulder clay and sands and gravels.

The solid strata shows a variability in in-situ hardness and density and reacts differently to exposure and weathering effects. A wide range of geotechnical properties can therefore be encountered from massive durable sandstones, requiring blasting to excavate, to highly weathered mudstones, consisting mainly of clay sized material. A typical opencast backfill will therefore be of a heterogeneous nature with a considerable variation in the dimensions of the constituent fragments, up to boulder size (Buist and Dutch 1984). This is further complicated by the fact that opencast workings are commonly associated with land reclamation and improvement schemes thus necessitating the incorporation of made ground, old foundations and re-worked backfill.

In the UK, opencast backfills typically cover an area of between 7 and 300 hectares to a depth of up to 100 metres. The overall density of an opencast backfill will be largely dependant upon the method of backfill placement. Traditional approaches result in large volumes of very heterogeneous backfill having greatly varying degrees of compaction. Where controlled restoration of the site is required, the backfill material is placed in relatively thin, uniform, layers and compacted to a level dependant upon the after use of the site. Such a method of placement produces a backfill having a more uniform compactive state and greater homogeneity.

2.3 Movements in Opencast Backfill

Movements in opencast backfills most commonly occur as a result of creep settlement, collapse settlement and heave. Mechanisms that often act in combination depending upon the site circumstances. The imposition of loads from overburden mounds or the construction of roads or structures can generate further vertical movements and in areas where backfill overlies buried excavation walls or side walls lateral movements can also occur. Parallels can be made between the movement of opencast backfills and those of earth and rockfill embankments.

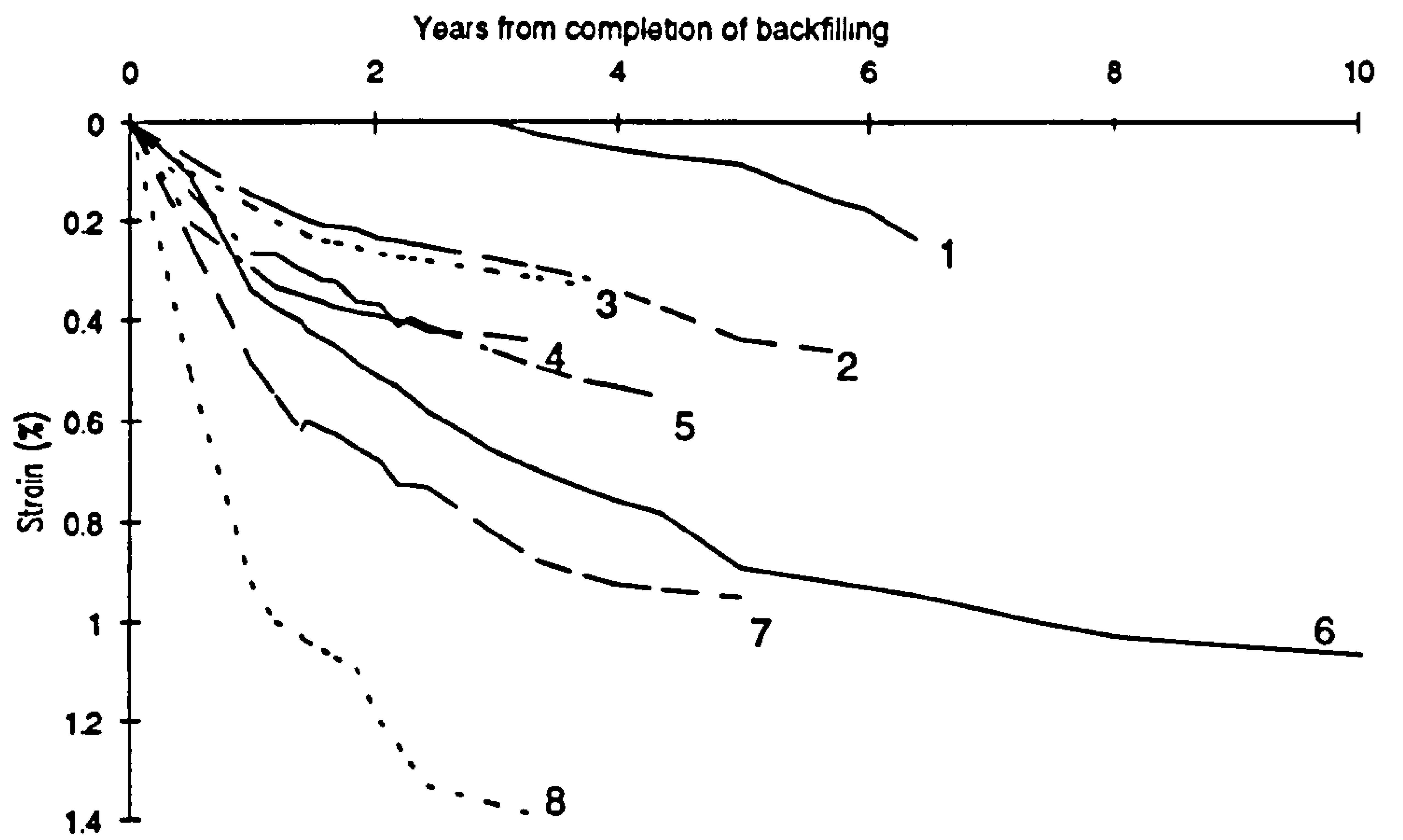
2.3.1 Creep

On completion of the placement of a body of fill significant settlement occurs under conditions of constant stress and moisture content known as creep settlement. Creep settlement has been observed in both opencast backfill and earth and rockfill embankments.

The mechanism for creep settlement is one of gradual re-arrangement of the material fragments resulting in a reduction in voids ratio due to the crushing of highly stressed contact points. Compressive stresses within a body of fill are concentrated at the contact points between material fragments. When such stresses exceed the compressive strength of the material, contact point failure occurs enabling movement of fragments thus producing a reduction in voids.

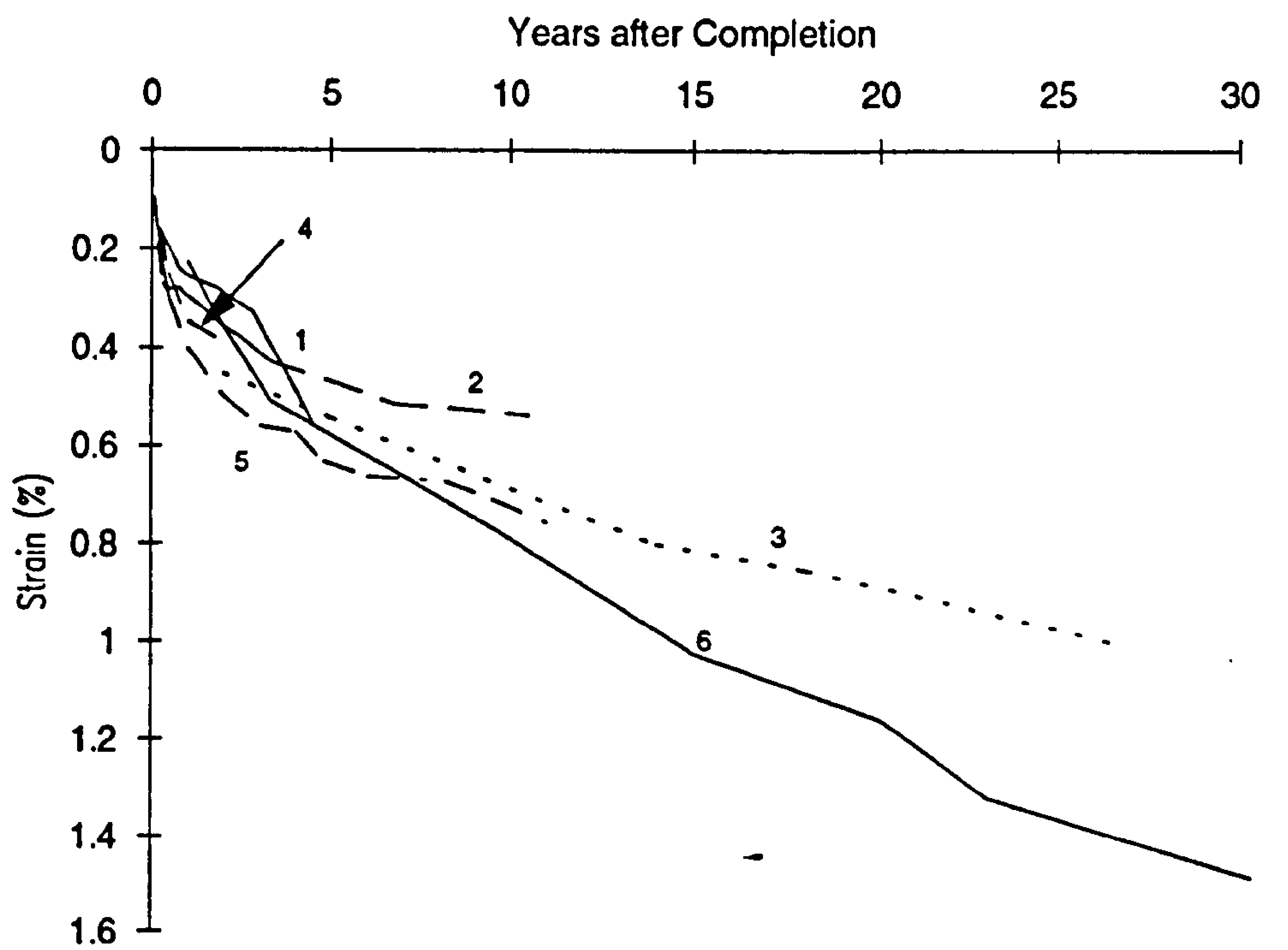
It is generally accepted that creep strain can be expressed in terms of the creep compression parameter (alpha) which is defined by Sowers *et al* (1965) as the gradient of the straight line graph of creep strain plotted against log time.

Examples of results obtained by monitoring of both earth and rockfill embankments and opencast sites as presented in the literature are reproduced in Figures 2.1 & 2.2. These indicate the scale and nature of creep settlement for a range of different sites.



Site	Average Depth (m)	Groundwater / backfill depth(%)	Material Type	Degree of Compaction
Clockfields (8)	20-50	dry	Mudstone fragments in a stiff clay matrix	Uncompacted
Chibburn (6) Horsley (1)	- 70	? 55	? Mudstone and sandstone fragments	Relatively uncompacted (placed by dragline/end tipping)
Dibdale West (2) Clockfields (4) Amblecote House (5) Turners Lane (7)	30 25-50 20 30	>50 dry dry dry	As Clockfields " " "	Spread in layers using box-scrapers
Amblecote North (3)	25-40	dry	As Clockfields	Spread in layers (<300mm) and compacted with vibrating rollers (min 4 passes)

Figure 2.1. Settlement against time for various opencast coal sites. (Knipe 1979, Charles *et al* 1987, Kilkenny 1968)



Dam	Crest Height	Fill Type
El Infiernillo (1)	148	Compacted Diorite and Conglomerate
Bear Creek (2)	66	Dumped Schist
Salt Springs (3)	100	Dumped Granite
Muddy Run (4)	76	Compacted Micaceous Schist
Selset (5)	43	Compacted Boulder Clay
Dix River (6)	84	Dumped limestone

Figure 2.2. Post construction crest settlements of some embankment dams (Penman 1971)

Location	Method	Max. depth of fill (m)	Type of Backfill	Settlement on Inundation			
				Years after backfilling	Maximum Settlement (m)	Settlement/ saturated fill depth (%)	Method of Inundation
Horsley	Truck and shovel, dragline	70	Mudstone and sandstone fragments	4	0.75	2 (0.8 ave)	Rising groundwater table
Ilkeston	Truck and shovel?	12	Clay	14	0.3	3	Ingress from surface-suspected via trenches
Corby	Dragline	24	Boulder clay overlying oolitic limestone	-	0.31	6 (2.5 ave)	Ingress from surface via trenches
West Auckland	?	18	Clay with shale fragments	24	0.36	5	Rising groundwater table
Dibdale West	Scraper placed	30	Mudstone fragments in a clay matrix	4	0.15	0.5 (0.2 ave)	Rising groundwater table
Cogswell Dam	Rockfill tipped without compaction	38	Hard durable rock fill	During construction	0.6	6	Rainstorm plus watering through infiltration wells
Radcliffe	Dragline/truck and shovel	35	Mudstone, sandstone and drift	2.5	0.1	?	Rising groundwater
Coldrife	Dragline/truck and shovel	30	Mudstone, sandstone and drift	14	0.1	1.8	Rising groundwater
High Vale	Dragline	20	Sandstone, mudstone and till	2	0.04	1	Rising groundwater

Table 2.1 Occurrences of inundation settlement described in the literature (Based on Charles & Burford 1987 and Reed and Hughes 1990)

The opencast sites have been restored by various different methods. Monitoring at the Horsley site did not begin until approximately 3 years after backfilling, consequently a significant proportion of the strain has not been monitored.

2.3.2 Collapse

Collapse settlement consists of a volumetric change under constant total stress due to an increase in water content. In the case of opencast backfill an increase in water content can occur at depth on cessation of any pumping operations or closer to ground level as surface water penetrates into the fill. Field observations and research has shown that inundation is a major cause of fill settlement in both compacted and uncompacted opencast backfills. Numerous case histories have been documented, a selection of which are described within Appendix A, with basic data and references summarised in Table 2.1.

Rock Type	Average compressive strength, dry N/mm^2	Average wet strength/dry strength (%)
Granite - Austria	145	88
Granite - Sweden and Germany	240	94
Crystalline Limestone	97	90
Dense quartz-mica schist Tennessee	96	47

Table 2.2. Compressive strength of rock, wet and dry (after Penman 1971)

One of the earliest examples of collapse settlement was that of Cogswell Dam (see Appendix A). No clear explanation of this was given until Terzarghi (1960) suggested that the settlements were caused by the reduction in the strength of rock on saturation. He quoted values for the strengths of rock when dry and saturated as published in 1945 (summarised in Table 2.2) which demonstrated the effect of saturation on rock strength. Terzaghi had carried out experiments which showed that wetting unpolished rock surfaces did not reduce the coefficient of sliding friction of rock on rock and he concluded therefore, that the settlement was not caused by any lubricating effect of the water but was due to a reduction of strength of the rock fragments at their points of contact.

Further, more detailed work, on the influence of moisture content on the compressive strength of rocks has been carried out by Colback and Wiid (1965).

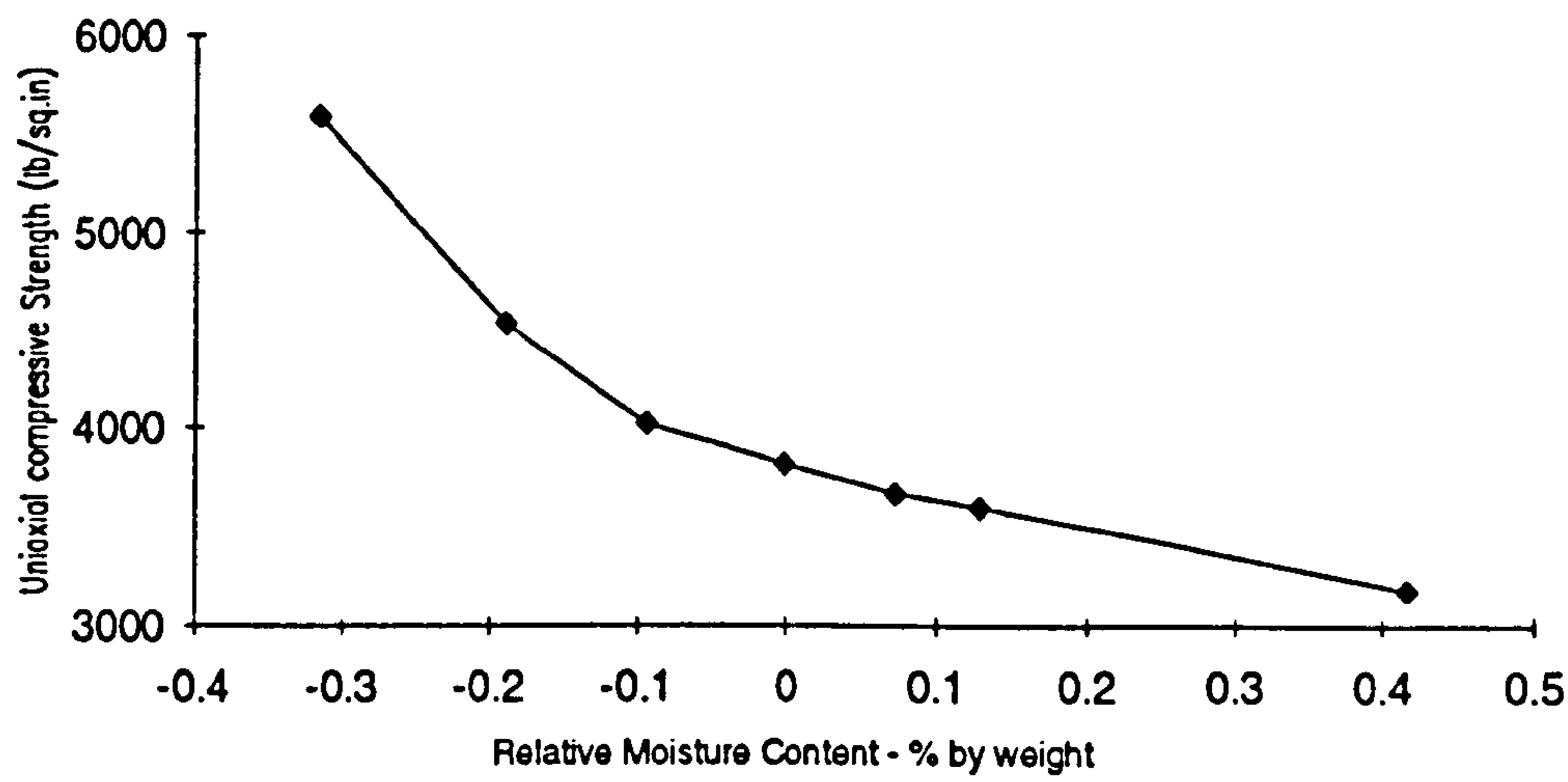


Figure 2.3. Relationship between uniaxial compressive strength and moisture content for quartzitic sandstone specimens (Colback & Wiid 1965)

Two quartzitic rock types were tested having appreciably different porosities, a quartzitic shale (0.28% porosity) and a quartzitic sandstone (15% porosity). Both showed a reduction in compressive strength with increasing moisture content following a general curvi-linear relationship as shown for the quartzitic sandstone in Figure 2.3. A range of moisture contents were achieved by placing the samples in environments of different constant humidity until a stable state was attained. This stable state was reached when periodic weighing of each specimen showed its weight remained practically constant with time. Saturation was achieved by submerging the specimens in water, again until a stable state was attained.

A similar behaviour was noted by Hassani *et al* 1979 who investigated the strength characteristics of rocks associated with opencast mining in the UK. A summary of dry and saturated compressive strengths are given in Table 2.3.

Examination of the Mohr fracture envelopes of rocks tested under triaxial compression showed that the coefficient of internal friction (the slope of the Mohr envelope) was not sensibly altered by the moisture content since for different moisture content conditions the envelopes were displaced parallel to each other. Colback and Wiid therefore tentatively concluded that the reduction in strength with increasing moisture content is primarily due to a reduction in uniaxial tensile strength which in turn is a function of the molecular cohesive strength of the material.

Sowers *et al* (1965) examined the compressibility of broken rock by carrying out compressive tests on samples of greywacke, granite and sandstone. Tests carried out prior to compression testing, to determine the physical properties of the three rock types, showed the unconfined compressive strength for both the granite and sandstone to be unaffected by saturation. The effect of jetting whilst under compression on the greywacke and sandstone is shown in Figure 2.4. Here it can be seen that when the dry rock under load was jetted, the settlement increased immediately by about 20%. Similar increased settlement was produced by merely wetting the rock, without jetting. The increases were the same in both the relatively porous sandstone and the non-porous greywacke.

Rock Type	Uniaxial Compressive Strength (MN/m^2)	
	Dry (σ_c ± S.Dev.)	Saturated (σ_c ± S.Dev.)
Mudstone	45.0 ± 5.0	5.0 ± 2.0
Seatearth	29.0 ± 8.0	1.25 ± 0.1
Laminated Siltstone	64.0 ± 13.0	45.0 ± 9.0
Massive Siltstone	77.0 ± 17.0	65.0 ± 12.0
Fine grained Sandstone	85.5 ± 9.0	56.0 ± 3.0
Medium grained Sandstone	68.0 ± 14.0	50.0 ± 15.0
Coarse grained Sandstone	34.0 ± 3.5	23.0 ± 4.0

Table 2.3. General strength characteristics of Coal Measures rocks from opencast sites (Hassani *et al* 1979)

This effect of wetting observed in the sandstone, which exhibited the same compressive strength wet or dry and the sudden crushing produced by wetting the impervious greywacke are not adequately explained by the above mentioned mechanism of the molecular cohesive strength weakening (mineral bond softening), as proposed by Colback and Wiid (1965). Sowers *et al* therefore put forward the explanation that micro fissures, produced in the highly stressed contact points, are entered by water due to capillarity causing a local increase in stress and additional failure.

Two theories therefore have been put forward to explain the collapse settlement of rockfills, that of the weakening of mineral bonds by wetting and that of additional failure due to water entering micro fissures. It is probable that a combination of these mechanisms is largely responsible for the collapse settlement of rockfill.

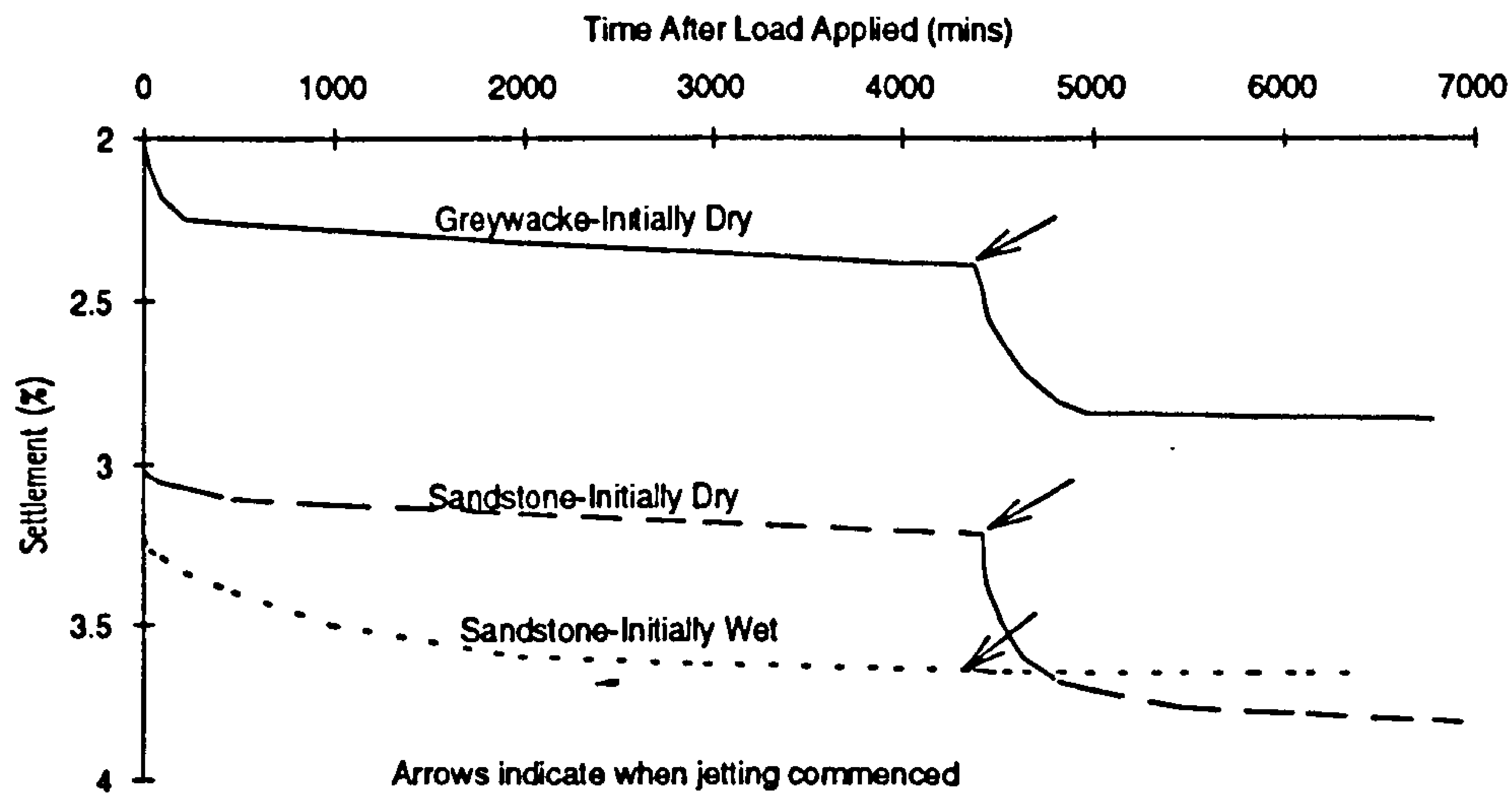


Figure 2.4. Effect of jetting dry rock under load. (Sowers *et al* 1965)

A further mechanism which will be influential when Coal Measures mudstones and shales (including seatearths) make up a large proportion of the fill, is that of breakdown of such rocks in water. Taylor and Spears (1970) examining this characteristic, put forward the mechanism of air breakage. Air, contained within the pores of the rock, becomes pressurised due to capillarity on saturation. If this pressure is great enough, failure of the mineral skeleton along planes of weakness ensues. This mechanism relies on the presence of air within the rock pores and can therefore only occur with saturated rocks if they are excavated and allowed sufficient time to dry.

To examine air breakage Taylor and Spears (1970) undertook slake tests in both air and in a vacuum. The "in vacuum" values demonstrated that breakdown can be arrested by removal of air thus confirming air breakage as a mechanism for breakdown of Coal Measure mudstones and shales. It was further demonstrated that the extent of breakdown in water was generally inversely proportional to grain size. As capillarity is inversely proportional to capillary size (pore radius) the above relationship is to be expected if air breakage is a significant factor of breakdown in water.

The "in vacuum" results did not however completely arrest water breakdown and work carried out by Nakano (1967) found no substantial difference between slaking in air and in a vacuum when testing Japanese mudstones. He attributes breakdown to chemical dissolution chiefly by hydrogen bonding of originally absorbed water molecules around clay particles with newly absorbed ones, thus causing inter-particle swelling and weakening of mineral bonds. This type of water expansion is characteristic of montmorillonite which is a major constituent of the Japanese mudstone. Hence inter-particle swelling is likely to be the dominant mechanism, largely blanketing any capillarity effects when high concentrations of montmorillonite are present.

In Coal Measures in the UK, montmorillonite is present in only very small concentrations or concentrated in a few thin seatearths (such as the Staffordshire Brock and Park seams) therefore the mechanism of inter-particle swelling can be considered to have little significance.

If a significant proportion of clay is present within a fill the effect of clay swelling/collapse must be considered, as discussed by Cox (1978). Cox points out that clay has the ability to swell or to collapse dependant upon moisture content and the level of overburden pressure. Testing of clay samples showed that as the initial moisture content rises (for a constant initial dry density), the amount of swelling first increases then decreases. This is because swelling occurs at a lower moisture content than collapse. Increasing the total (overburden) pressure produces an increase in the amount of collapse settlement.

2.3.3 Heave

Heave has been observed in a number of opencast mine backfills and can be associated with the removal of overburden mounds and in some cases with the saturation of the upper few metres of the fill due to surface water ingress. Magnitudes of heave associated with surcharge removal as measured by Knipe (1979) and Charles and Burford (1978) are shown to be in the order of 0.2% fill thickness. A similar value was recorded by Reed (1986) when monitoring a mudstone backfill. Heave upon removal of overburden mounds generally decreases with time. Heave associated with surface water ingress is typically complete within a few months of backfilling, however the rate at which these movements occur depends on the rate of surface water saturation (SARCOB 1993).

Heave due to the removal of overburden is generally explained by the elastic expansion of rock fragments upon a reduction in effective stress (Knipe 1979). Heave as a result of saturation is a little less clear and a number of mechanisms have been proposed. Overburden material at depth, prior to excavation, will be subjected to large confining stresses which upon excavation and placement at shallow depths will no longer act. This will result in the development of tensile stresses within the material and in the case of mudstone and clay fragments which contain a system of fine capillaries, negative pore pressures can develop. Volume increase may then occur upon saturation as tensile stresses are released upon disintegration of the material due to weakening of molecular cohesive bonds and water entering micro fissures. In the case of clays and some instances mudstones, the negative pore water pressures which may be present can lead to swelling (Cox 1978). It has been noted that heave upon saturation is only evident in compacted fills subjected to low confining stresses at shallow depths (SARCOB 1993).

Other mechanisms thought to be responsible are physio-chemical in nature; the swelling of montmorillonite and the oxidation of pyrite. As the proportion of true swelling clay minerals in UK backfills is very low to non existent the mechanism of clay swelling is considered to be negligible (Taylor and Spears 1970). The oxidation of pyrite to form gypsum involving an increase in volume is thought to be responsible for foundation failures in several small developments. Such a reaction will only occur in an oxidising environment and will therefore generally be limited to the top 1 to 3 metres of the backfill. If oxidising conditions could be identified by careful investigations, then it would account for the constant rate of heave with time as these reactions are known to occur over a protracted length of time. However, these conditions would need to exist at depths greater than 2 to 3 metres since backfill within this depth of the restored surface is typically made up by pyrite-deficient sub-soils and top-soils (Saunders 1988).

2.3.4 Differential Settlement

It can be considered that differential settlements are the cause of most damage to any structures or roads constructed upon opencast backfill. The differential, or relative, settlement between one part of a structure and another is of greater significance to the stability of a given structure than the magnitude of the total settlement (Tomlinson 1986). Differential settlements occur within opencast backfills as a result of the following:

- Variations in the depth of the backfill which generally occur where the fill overlies any buried walls or high/side walls.
- Lateral variations in the constituent materials making up the backfill.

Site	Stations	Change in Fill Depth (m)	Distance between Stations (m)	Buried Slope Gradient vertical:horizontal	Differential Settlement (mm)	Rotation	Method of Backfilling
Holly Bank	25 to 29	3.0 to 22.5	9.0	2.2:1	22	1 in 410	Compacted [1]
	42 to 45	1.7 to 40.0	6.1	6.3:1	22	1 in 280	
	16 to 18	2.3 to 25.0	3.5	6.5:1	109	1 in 32	Uncompacted
	50 to 52	19.8 to 52.5	4.6	7.1:1	9	1 in 510	Compacted [1]
	8 to 6	23.0 to 52.3	11.3	2.6:1	10	1 in 1130	
	36 to 38	16.7 to 30.0	3.6	3.7:1	2	1 in 1800	
Hurst	13 to 15	2.5 to 20.7	15.0	1.2:1	29	1 in 517	Compacted [2]
	7 to 9	0 to 16.4	14.7	1.1:1	22	1 in 668	
	1 to 3	3.3 to 16.8	14.9	0.9:1	22	1 in 677	
	19 to 22	5.5 to 26.2	25.6	0.8:1	48	1 in 533	

Table 2.4. Differential Settlements as Measured by Patterson (1986)

[1] Backfill compacted by method compaction. Settlements monitored for 18 months after backfilling

[2] Backfill placed by scraper below 15 m depth and by method compaction in upper 15 m. Settlements monitored for 2 years after backfilling

- Lateral variations in the compactive state of the backfill.
- Non-uniform saturation of the backfill from surface water ingress or groundwater table rebound.

Values for differential settlement as measured by Patterson (1986) are summarised in Table 2.4.

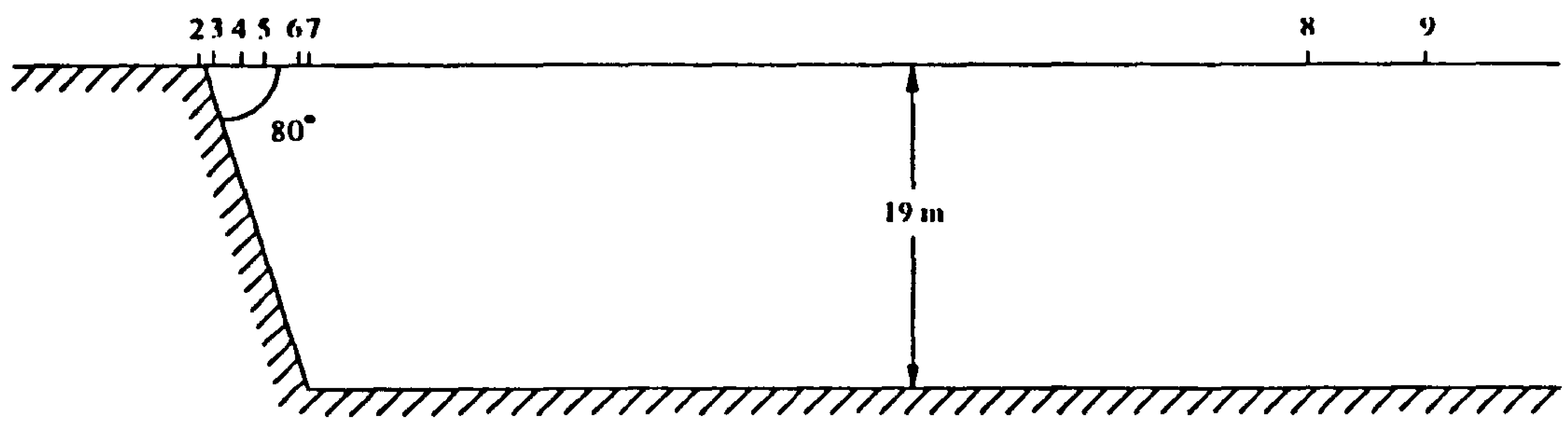
2.3.5 Lateral Movements

The potential for damage of any development sited on opencast backfill is considered in terms of the movements that occur due to settlement or heave. However, observations made by Reed (1986) show that lateral fill movements can be considerable and in some instances exceed vertical fill movements. At one of the sites examined by Reed, a net horizontal movement of 62mm was measured at a shallow uncompacted backfill, whilst only 29mm of vertical settlement was observed. Measurements were carried out over a steep highwall (80^{o}) and are summarised in Table 2.5 showing the movement between settlement stations. The fill was only 20 metres deep and it is expected that these movements may be amplified within greater thicknesses of fill. Horizontal movements must have a relationship to vertical settlement and this becomes of importance where rapid changes in fill depth occur (generally over high walls or buried walls) and at sites where different systems of compaction have been used. Selective compaction under proposed roads and buildings may successfully reduce vertical movements, but horizontal movements could occur as a consequence of the movement of the uncompacted fill adjacent to the construction (Reed 1986).

The measurement of lateral fill movements across high walls at Holly Bank, Essington, (Patterson 1986), indicated them to be minimal for compacted fill but significant for uncompacted fill. A summary of the lateral fill movements for the compacted material as shown in Table 2.5, indicate no real patterns of movement. The uncompacted material, however, showed a distinct pattern of movement, with tension (up to 20mm over 10 months) occurring over the top of the high wall and compression (up to 10mm over 8 months) over the base of the high wall.

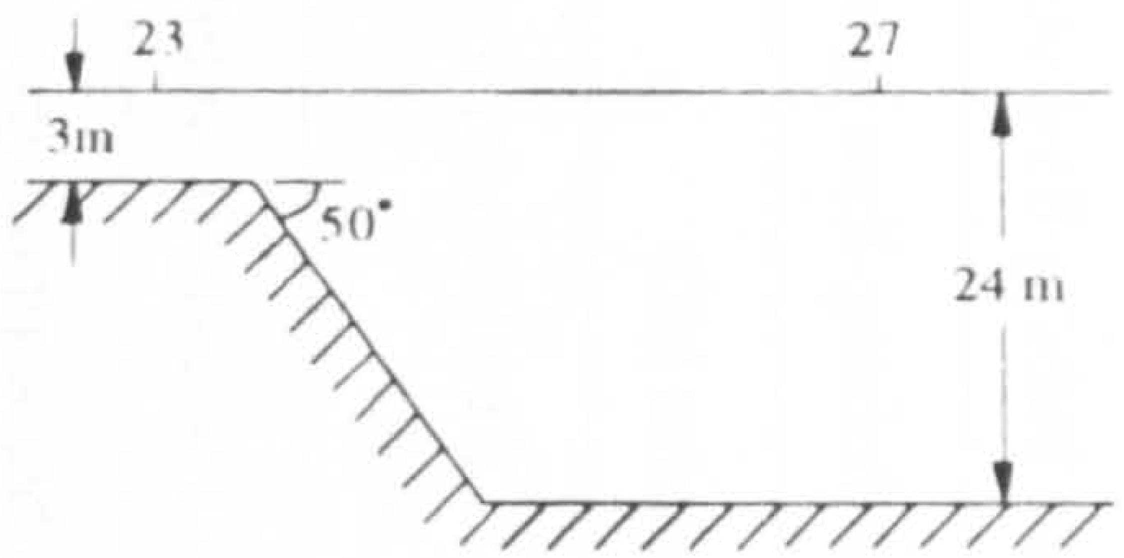
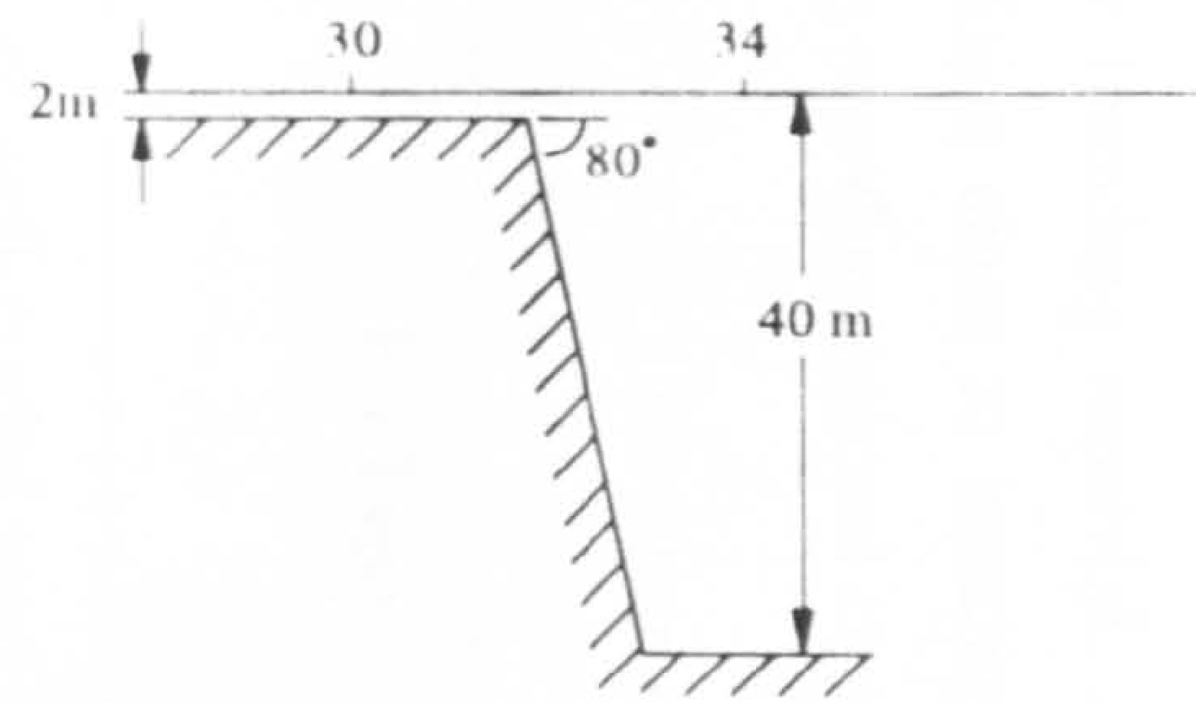
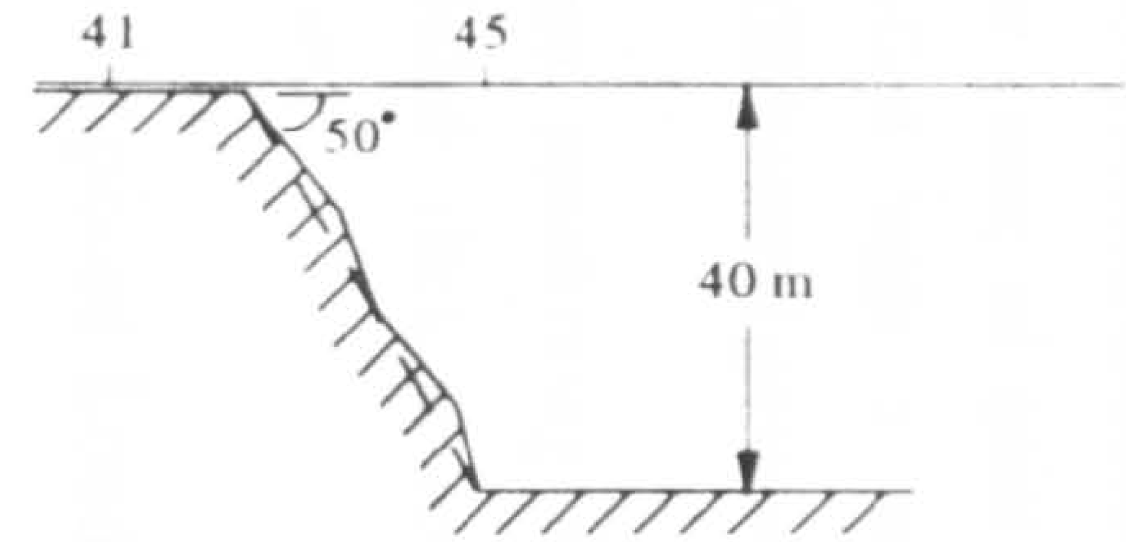
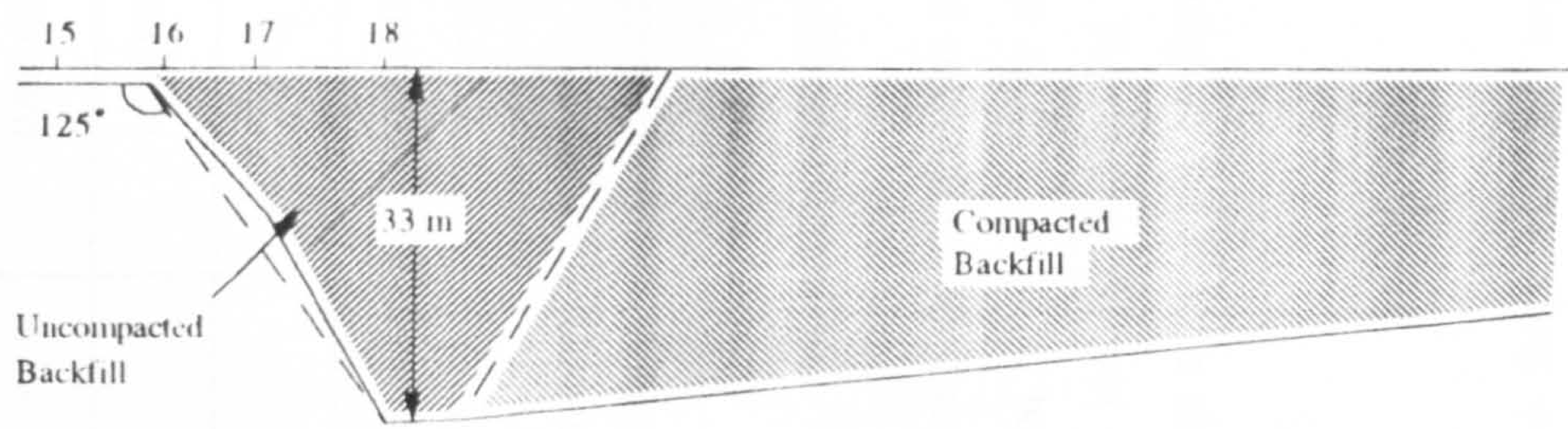
The removal of horizontal constraint by the excavation of an opencast mine may lead to the relaxation of the solid walls. In addition the solid slope may move due to the effects of faulting or other factors contributing to mine slope instability. Damage to off-site structures have been recorded at a number of sites probably occurring as a result of either or both of these phenomena. These two factors may induce horizontal

Site G Uncompacted (Reed 1986)



Interval	Distance (m)	Lateral Movement	Period of Monitoring
2 to 3	1.0	3.9mm compression	29 months
3 to 4	1.4	5.3mm elongation	29 months
4 to 5	1.0	7.2mm elongation	29 months
5 to 6	1.4	17.0mm elongation	29 months
6 to 7	1.0	25.7mm compression	29 months
7 to 8	46.0	0.6mm elongation	29 months
8 to 9	3	7.2mm compression	29 months

Table 2.5. Lateral Fill Movements as measured by Patterson (1986) and Reed (1986)

<table>
<tr><td colspan="7">Holly Bank, Essington (Patterson 1986)</td></tr>
<tr><td rowspan="2"></td><td colspan="6">Compacted</td></tr>
<tr><td colspan="2">
</td><td colspan="2">
</td><td colspan="2">
</td></tr>
<tr><td>Lateral Movement</td><td colspan="2">23 to 27 : +/- 5mm, compression and elongation</td><td colspan="2">30 to 34 : 8mm compression</td><td colspan="2">41 to 45 : 12mm elongation*</td></tr>
<tr><td>Period of Monitoring</td><td colspan="2">13 months</td><td colspan="2">12 months</td><td colspan="2">16 months</td></tr>
<tr><td rowspan="2"></td><td colspan="6">Uncompacted</td></tr>
<tr><td colspan="6">
</td></tr>
<tr><td>Lateral Movement</td><td colspan="3">15 to 16 : 20mm elongation</td><td colspan="3">17 to 18 : 10mm compression</td></tr>
<tr><td>Period of Monitoring</td><td colspan="3">10 months</td><td colspan="3">8 months</td></tr>
<tr><td colspan="7">Table 2.5. (Cont.)</td></tr>
</table>

*Movement considered to be influenced by site traffic rather than ground movement movements in a backfill adjacent to the rock slope (Condon 1986) and together with fill movements induced by differential vertical settlements across side/high walls, could be the cause of quite large lateral fill movements. Such movements would be easily identified with a series of surface movement markers positioned across the backfill/natural ground interface.

Further monitoring of the scale and location of lateral fill movements in future backfilled sites is required to obtain a better understanding of the mechanisms involved.

2.3.6 Imposed Loads

Opencast backfill will be compressed by foundation loads in a similar many to any other founding material. Assuming the stress increments do not bring the fill to a condition close to bearing capacity failure, the settlements can be simply calculated using one-dimensional consolidation theory. The constrained modulus is defined as:

$$D = \text{Change in Stress/Change in Strain } (KN/m^2) \qquad 2.1$$

Fill Type	Compressibility	Typical Values of Constrained Modulus (KN/m^2)
Dense well-graded sand and gravel	very low	40 000
Dense well-graded sandstone rockfill	low	15 000
Loose well-graded sand and gravel	medium	4 000
Old urban fill	medium	4 000
Uncompacted stiff clay above water table	medium	4 000
Loose well-graded sandstone rockfill	high	2 000
Poorly compacted colliery spoil	high	2 000
Old domestic refuse	high	1 000 - 2 000
Recent domestic refuse	very high	-

Table 2.6. Compressibility of fills (Charles 1984a)

Some typical values of constrained modulus for a number of different fill types are given in Table 2.6 .It should be noted that the nature of opencast mine backfill would be expected to be of medium compressibility. It is generally considered that due to the relatively high permeability of opencast backfill, compression due to foundation loads will occur as the structure is being built. There will be some long term (creep) settlement but this will generally be insignificant in comparison to the self weight creep of the fill itself.

A direct method of determining the influence of foundation loads is to monitor the movement of a waste disposal skip, filled with damp sand and then covered with 75mm of concrete. This will give a direct indication of the performance of lightweight structures with strip footings, a method described by Charles and Burland (1982).

The possibility of bearing capacity failure can be determined from plate bearing tests and other direct measurements of ground strength. These usually demonstrate that opencast backfill is capable of supporting shallow foundation loads of at least 80 to 100 kN/m^2 (Knipe 1988). Plate bearing tests carried out by Patterson (1986) at a site compacted to The Department of Transport Blue Book Specification (1976) indicated a net allowable bearing capacity of at least $120kN/m^2$. It must be noted that damage to structures built on opencast backfill is rarely as a result of bearing capacity failure and more commonly due to either creep or collapse type settlements within the fill itself (Knipe 1988).

2.4 Factors Influencing Movement Behaviour

2.4.1 Fill Material Properties

The magnitudes of both creep and collapse settlements will be influenced by the properties of the fill material. Fill material properties having the most influence on the settlement behaviour of restored backfill or a rockfill structure, as described by Egretli and Denby (1988), are as follows:

- Particle size distribution
- Particle shape
- Particle strength
- Packing State

Particle size distribution

According to Penman (1971), settlements can be minimised by properly compacting a rockfill which has sufficient fine material to infill the void space between larger rock particles. It is thought that encasing the larger rock particles in the finer grained material will reduce point-to-point contact between particles and prevent rotation of larger particles due to breakage at the contacts. If particle reorientation can be prevented the magnitude of long-term settlement can be reduced. Kjaernsli and Sande (1963) showed that a well graded rockfill containing a large proportion of fines was less compressible than a uniformly graded rockfill.

Another point to consider is that the potential for breakage of a particle increases with its size. This results from the fact that the normal contact forces in a rockfill increase with particle size (the total contact area decreases with increasing particle size) and the fact that the probability of a defect in a given particle increases with its size (Hardin 1985).

Particle shape

The influence of angularity on settlement is shown by the work of Penman (1971). When sharp corners are absent compression of a fill is much reduced. This can be seen in the results of confined compression tests on quarried rockfill and river gravel derived from the same rock origin as shown in Figure 2.5.

Particle Strength

Creep and collapse settlement is primarily as a result of the crushing of contact points which will be directly related to the compressive strength of the fill particles. The compressive strength of rocks associated with UK opencast mining operations has been shown to be dependant upon weathering state, degree of saturation and due to bedding planes and other anisotropic behaviour, dependant upon orientation (Hassani 1979).

Packing State

The packing state is simply a measure of the density of a fill and hence inversely proportional to the voids ratio of the fill. The compressibility of a fill is clearly proportional to its voids ratio and thus inversely proportional to its packing state.

The relationship the above properties have with one another can be seen by the work of Marsal (1973) who examined the behaviour of granular materials and rockfill in the building of the El Infiernillo Dam in Mexico. It was determined that the principle cause of continuing settlement is the degradation of particles i.e. the crushing of contact points and the subsequent re-arrangement of particles into a denser state.

To assess degradation Marsal carried out a series of laboratory tests from which the following relationship was derived:

$$B = F.\left(\frac{\sigma}{Ns}.\frac{1}{q_u}\right) \qquad 2.2$$

where B is a measure of degradation hence settlement, F is a constant, σ the normal stress, Ns the no. of contacts/unit area and q_u the compressive strength. As Ns is dependant upon the grading of the material (Ns increases as the grading goes from narrow to broad) and inversely proportional to void ratio (Kilkenny 1968) the relationship between particle size distribution, packing state and particle strength and settlement can be clearly seen.

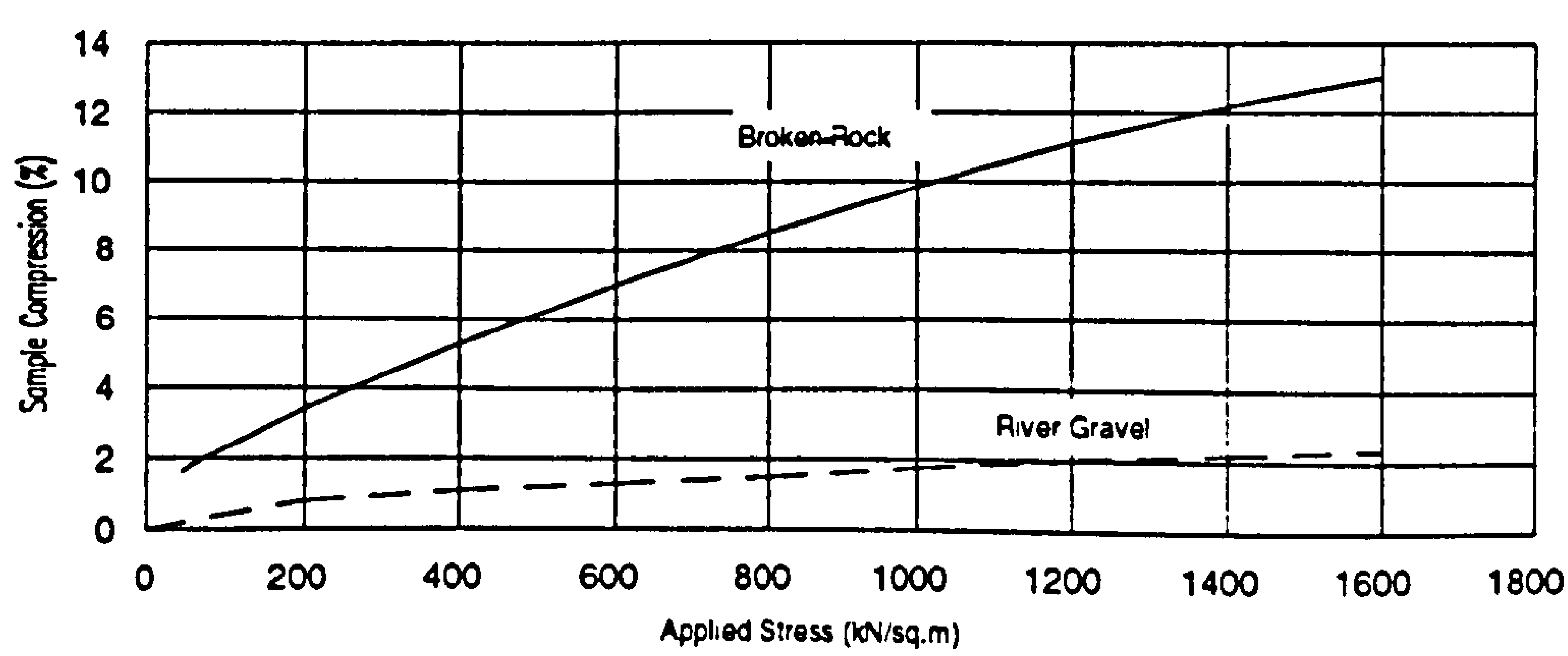


Figure 2.5. Compression of broken rock fill and river gravel (Penman 1971)

2.4.2 Mining Method and Restoration Procedure

The layout and design of any mining operation is largely determined by a number of certain site specific characteristics;

- The magnitude of operations
- Orientation, number and regularity of seams
- Thickness of overburden and interburden
- Diggability of the individual horizons

- Environmental constraints
- Economic constraints

Many machines and systems exist for the operation of an opencast mine. In the U.K. however, an operation can generally be described by one or a combination of the following;

- Dragline operations
- Truck and shovel operations
- Ripper and scraper operations

The mining method chosen will have an influence upon the size, shape and grading of the particles making up the backfill and the amount of compaction imparted upon then during their placement; factors which as described above, have considerable influence on the post mining behaviour of the fill.

Dragline cast and dump truck tipped backfills will have had little compactive effort imparted upon them and show both lateral and vertical variations in grading. Variations in grading are caused by segregation, as the fill tumbles down the loosewall larger particles fall to the base of the slope. This type of segregation has been noted in dumped rockfill embankments producing lifts changing from relatively fine grained and well graded material at the top to coarser and less well graded material near the base (Terzaghi 1960). Rehm *et al* (1980) examining the hydraulic properties of opencast backfill, discovered a similar phenomenon in dragline cast spoil but also noted a lateral variation related to the ridge and valley effect produced by this method of backfilling. This can again be explained by segregation as the fill tumbles down the loosewall.

Backfills of this nature will be prone to large amounts of post-reclamation settlements as demonstrated in Figure 2.1 which also shows the greater amounts of settlement of dragline cast and truck end-tipped placed backfills (uncompacted and relatively uncompacted fills) when compared with other methods of placement.

Backfills placed by scraper operations will have a large amount of plant running over them resulting in compactive energy being imparted, hence an increase in packing state is achieved, which has resulted in smaller post-reclamation settlements as demonstrated in Figure 2.1.

Where controlled compaction is adopted during backfilling the properties of the material placed prior to compaction and the properties of the backfill after compaction have to comply to a compaction specification. This ensures that a desired packing state is achieved and that the factors discussed above which influence post-reclamation settlements are within acceptable limits. Compaction specifications adopted for opencast backfilling operations are based upon the Department of Transport Specification for Road and Bridge Works (1976) and more recently for Highway Works (1986). These specifications outline acceptable methods of compaction and limits on particle size distribution, maximum particle size and the density (packing state) of the compacted backfill.

2.4.3 Mine Features

Not only does the mining method determine the type of machinery to be used but it also determines the position and presence of certain site specific features, the influence of which are summarised below:

(a) Spoil Mounds

Spoil mounds situated on previously backfilled ground will impose additional stress on the material beneath them, thus increase inter-particle contact stresses which will ultimately lead to settlement. Post-reclamation settlements can be much reduced upon their removal.

(b) Lagoons

Lagoons positioned over previously backfilled ground can in some instances induce collapse settlement in the underlying material due to water percolation. This was demonstrated at the Horsley site in Northumberland. Settlements measured at an extensometer located within fill over which a lagoon had been previously positioned were small when compared with settlements measured at extensometers located elsewhere within the site. This was explained by the fact that whilst the lagoon was in place, percolation of water into the backfill sufficiently "wetted" the material, hence inducing collapse, so as to prevent further settlement occurring during the subsequent rise in the water table (Charles *et al* 1984a).

(c) Haulage Roads

Large mining machinery tracking back and forth along haulage roads will impart compactive energy into the ground beneath the road. This will cause failure of inter-particle contact points and subsequent re-arrangement of the particles producing a denser material less prone to future settlements. The influence of the machinery on the fill will be similar to that of scraper laid fills which demonstrate improved settlement characteristics when compared with fills placed without machinery tracking over them (Knipe 1979).

(d) Pit boundary walls and internal walls

The steepness of any walls, either boundary or internal, will have a significant effect on the differential settlement of backfill material placed over these walls. Restricting slope angles, hence reducing differential settlements, can significantly increase the cost of an opencast mining operation as the amount of overburden material to be excavated increases with decreasing wall angle. Boundary wall slopes, especially, are therefore generally designed to ensure stable slopes and not to reduce differential settlement upon completion; as such quite large differential settlements are commonly associated within backfill material overlying these walls.

2.4.4 Groundwater Recovery

The inundation of a body of opencast backfill can have a significant effect on its stability due to collapse settlement as discussed above. The most common cause for inundation is the recovery of the groundwater table on completion of opencast operations. If the groundwater table intersects with the mined void above the base of the pit, it will be drawn down during mining operations, either by sump pumping within the excavation or by pumping from boreholes surrounding the site. When the mined void is backfilled and the de-watering pumps are withdrawn it is believed that the groundwater will rebound to the level prior to mining (Norton 1984) and the original flow system will not have been substantially changed (Straskraba 1986). The rate at which this rebound occurs is all important in the prediction of future ground movements. Monitored rates of recovery for sites examined by Reed (1986) and at the Horsley site (Charles *et al* 1984a) are summarised in Table 2.7 and give some indication of the range of recovery rates monitored at uncompacted opencast backfill sites.

The rate of groundwater recovery and the level of the re-established water table are dependant on the following inter-related factors (Singh *et al* 1984).

a) The Post-Mining Hydrological Regime.

Recharge may occur solely as a result of direct contact between the fill and a subsurface aquifer. In areas of previous mining the hydrology of the ground around the restored pit will be particularly complex. If the mined void intersects with any

Site	Depth (m)	Areal Extent	Compactive State	Recovery Rate		External Influences
				Water Table Rise (cumulative)	Time From Cessation of Pumping	
Site A (Reed 1986)	17	0.9 hectares	Uncompacted	9m	48 days	Deep mine workings discharging groundwater into the backfilled void
				18m	96 days	
				23m	144 days	
Site B (Reed 1986)	80	66 hectares	Uncompacted	5m	1 yr	Deep mine workings acting as a drain to the backfilled void
Site C (Reed 1986)	30-70	-	Uncompacted	To within 10m of pre-mining level	6 months	-
				To within 5m of pre-mining level	18 months-after which recovery virtually ceased	
Horsley (Charles et al 1984)	70	0.9 sq. km	Uncompacted	19m	1 yr	-
				30m	2 yrs	
				35m	3 yrs	
				38m	4 yrs	

Table 2.7. Groundwater Recovery Rates

abandoned deep mine workings then these may act as a drain for the fill, or conversely as a source of water inflow. The presence of external pumping operations will also greatly influence the hydrological regime.

b) The Permeability of the Backfill.

The permeability of an opencast backfill is dependant upon material properties and restoration method. The important material properties are particle size distribution and whether or not the particles constitute a soil or a rock.

Penman and Charles (1975) indicated that the permeability of a granular material is closely related to particle size and grading. Kenney *et al* (1984) showed permeability to be primarily dependant upon the sizes of the particles in the fine fraction producing an equation which calculates permeability from the size of the smallest 5% of the particle size distribution. It was further concluded that the permeability is essentially independent of the shape of the gradation curve.

As to whether the constituent material is a soil or a rock, the distinction made by Terzaghi and Peck (1948) can be used: "Soil is a natural aggregate of mineral grains that can be separated by such gentle means as agitation in water. Rock on the other hand, is a natural aggregate of minerals connected by strong and permanent cohesive forces". In the case of dam construction an earthfill dam is constructed from soil whilst a rockfill dam is constructed from rock.

A distinction useful in geotechnical terms is that a rockfill is considered to be a free-draining material in which no constructional positive pore pressures develop. The free-draining property of rockfill is largely a function of permeability and although the development of constructional pore pressures depends on several factors besides permeability (initial degree of saturation, compressibility, rate of construction), it would seem that 10^{-5} m/s might form a reasonable lower limit to the permeability for a material that can be accepted as rockfill (Penman and Charles 1975).

If this distinction is applied to opencast backfill, backfill constructed from soil (glacial deposits, weathered mudstones etc.) can be considered to be a non-free-draining material having a permeability less than 10^{-5} m/s.

The measured permeability of uncompacted opencast backfill varies from between 10^{-4} and 10^{-8} m/s, depending upon the argillaceous content of the original excavated rock (Norton 1983). A summary of measured permeabilities found in the literature for selected uncompacted opencast backfills and compacted rockfill dams are shown in Table 2.8.

In the case of compacted backfill, if it is assumed that the inter-particle voids within a backfill constructed from rock are connected, then permeability is directly related to the void ratio of the fill. As the process of compaction reduces voids, it must be considered that permeabilities will be lower for compacted fill than those for uncompacted fill. In the case of compacted backfill constructed from soil, it is less likely that the inter-particle voids will be connected, thus the permeability will approach that of the constituent soil particles.

Site	Coefficient of Permeability (m/s)	
Site B (Reed 1986) Uncompacted backfill dragline placed	Surface - 1m	3×10^{-4}
	1m - 6m	5.1×10^{-6}
	6m - 11m	6.7×10^{-6}
	11m - 14m	3.2×10^{-5}
Northern Great Plains (Rehm *et al* 1980) Uncompacted backfill dragline placed	Average	3×10^{-4} with a 1.5 order of magnitude S.D.
	Spoil Ridges	3×10^{-8} (range 3×10^{-7} to 5×10^{-9})
	Inter-ridge valleys	2×10^{-7} (range 5×10^{-5} to 3×10^{-9})
	Permeability's found to decrease with time, typically an order of magnitude in 17 months	
Llyn Brianne Compacted Rockfill Dam (Penman and Charles 1975)	Range	1×10^{-5} to 8×10^{-6}
Balderhead Dam Compacted Earthfill (Penman and Charles 1975)	Range	3×10^{-5} to 9×10^{-5}

Table 2.8. Measured permeabilities

Groundwater recovery rates can be predicted qualitatively from a knowledge of the criteria listed below (Reed 1986), but it must be recognised that any prediction can only be an approximation since recovery rates have been found to be extremely variable in practice.

- Pre-mining piezometric surface.
- Depth and areal extent of site.
- Position of old deep mine workings and whether they are discharging or draining water.
- Knowledge of external influences, e.g. adjacent opencast mines, underground mines.
- Hydraulic properties of the backfill.

2.4.5 Surface Water Infiltration

Surface water infiltration will increase the moisture content of near surface backfill material which as discussed above, can lead to collapse settlement. The main cause of surface water infiltration is from rainfall and leakage from drainage channels if any are constructed as a result of site development. The hydraulic properties of the near surface backfill material will determine the rate of infiltration.

2.5 Conclusion

It can therefore be seen from the information outlined above that a range of factors are responsible for the movement, relevant to structural development, of an opencast backfill. It is these factors that need to be more fully understood to be able to design for and predict backfill movement. In order to achieve this a joint research project instigated by British Coal Opencast was set up between Scott Wilson Kirkpatrick Consulting Engineers and The Department of Mineral Resources Engineering at Nottingham University. British Coal opencast provided all available information relevant to the controlled compaction operations carried out at British Coal opencast coal sites for review. The following chapter summaries the information obtained from the study sites and gives the conclusions reached upon its analysis.

ANALYSIS OF STUDY SITE DATA

3.1 Introduction

A considerable quantity of data has been reviewed by the author, and is summarised below. The majority of the data was made available by British Coal Opencast and comes from the monitoring of fill properties and the state of compaction achieved during placement and the subsequent backfill movements and groundwater table levels. It was not possible to include the monitoring data within this work, however it can be referenced within a report produced jointly between Nottingham University and Scott Wilson Kirkpatrick; SARCOB (1993).

In the following discussion creep strain will be discussed in terms of the creep rate compression parameter, alpha which is equal to the gradient of the graph of creep strain vs log time as discussed in Chapter 2. The behaviour of a given surface settlement station may not be simply made up of creep strain but may also include periods of heave and collapse. Therefore to calculate alpha for a given strain vs log time graph these other 'events' must be excluded from the calculation of gradient. Figure 3.1 demonstrates this by the exclusion of a period of collapse strain in determining the creep strain behaviour of three surface settlement stations at the Blindwells site.

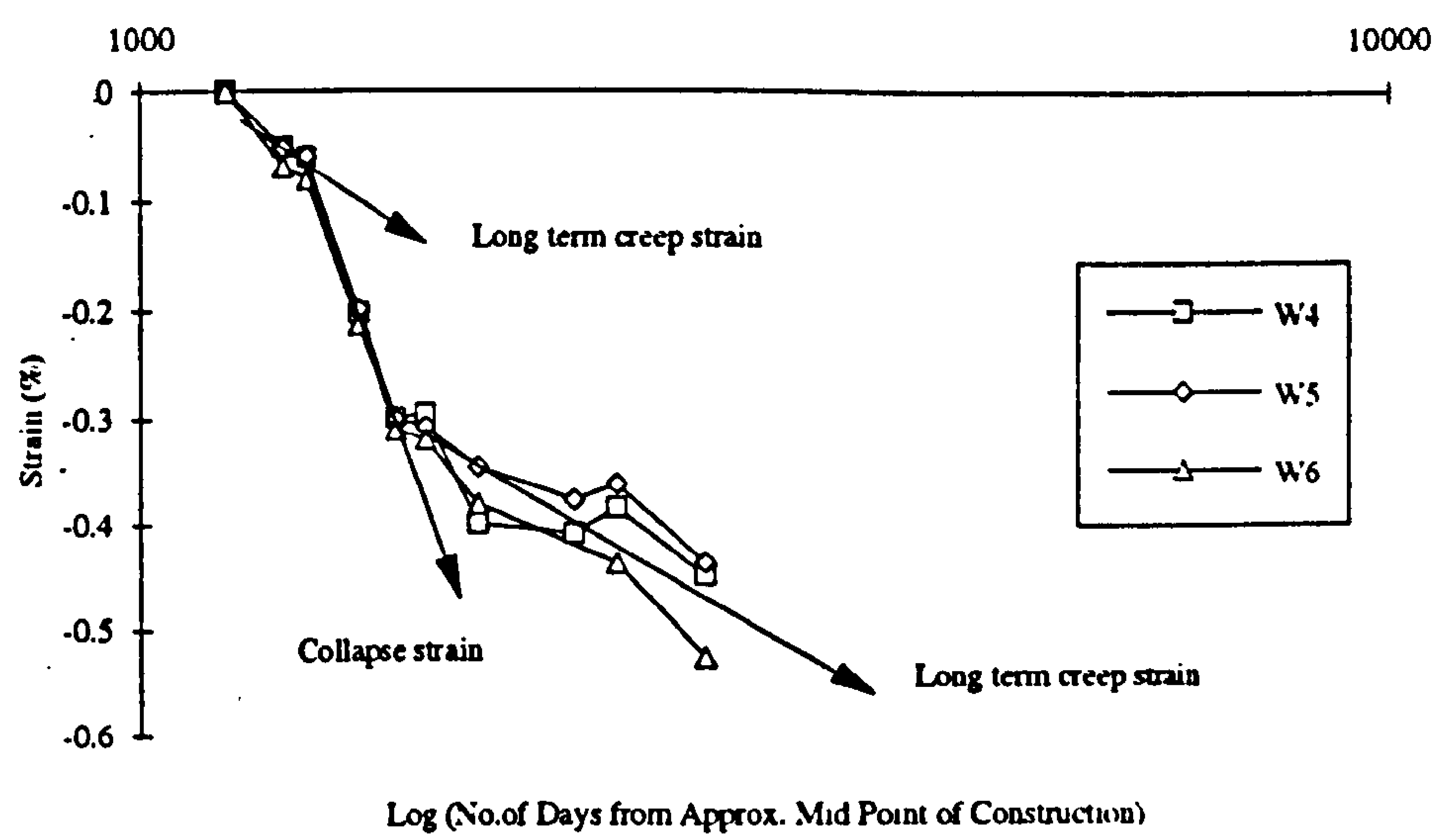


Figure 3.1. Blindwells surface settlement results demonstrating the calculation of creep strain.

The time period over which creep strain is measured has to be relative to the time at which creep strain commences (t_{zero}) for the straight line relationship proposed by Sowers *et al* (1965) and hence the alpha value obtained, to be valid. Sowers proposed that t_{zero} be taken as half the construction time for earth/rockfill dams for which the relationship was proposed. This is the approach taken by this work in the calculation of alpha, with t_{zero} taken as the time half the column of backfill beneath a given settlement station is constructed.

To be able to determine t_{zero} a schedule of compaction operations is required detailing the progress of backfill construction. In the case of the study sites only approximate details of the construction schedule were obtainable and it was therefore considered that alpha values should only be calculated from surface settlement stations and not from extensometers where information on the compaction schedule, more detailed than was available, would be required for an accurate determination of t_{zero} and hence alpha. The reason a greater accuracy in the schedule is required for the determination of alpha from extensometer results is that extensometers measure creep strain over a reduced column of backfill material and it is more difficult to determine the start and finish times for the construction of such a column without more accurate information on construction schedules.

The determination of the magnitude of collapse strain as discussed below is limited to extensometer results as these enable a more accurate measure to be made because the layer within which the collapse is occurring can be identified. Collapse strain determined from surface settlement stations will give misleadingly low results unless the full depth of backfill beneath a given station is subject to collapse over the monitoring period. As this is unlikely to occur surface settlement results are used only to identify a collapse 'event' and not in the calculation of the magnitude of the collapse. Figure 3.2 demonstrates the calculation of collapse strain between two magnets that become saturated due to a rise in water table.

3.2 Barnabas Opencast Coal Site

The Barnabas site is located on the Eastern outskirts of Clay Cross, Derbyshire, within the North East Derbyshire District. It has an approximate area of 25 ha and coal was worked at depths of between 9 and 25m below ground level. The site was worked as two areas A and B. The works were subsequently extended to include an area referred to as the Western Extension, or area C. The total volume of material excavated was 3,488,835 m^3, which yielded 305,950 tonnes of coal from the Deep Hard and underlying Piper seams giving a gross working ratio of 19 to 1. Mining operations

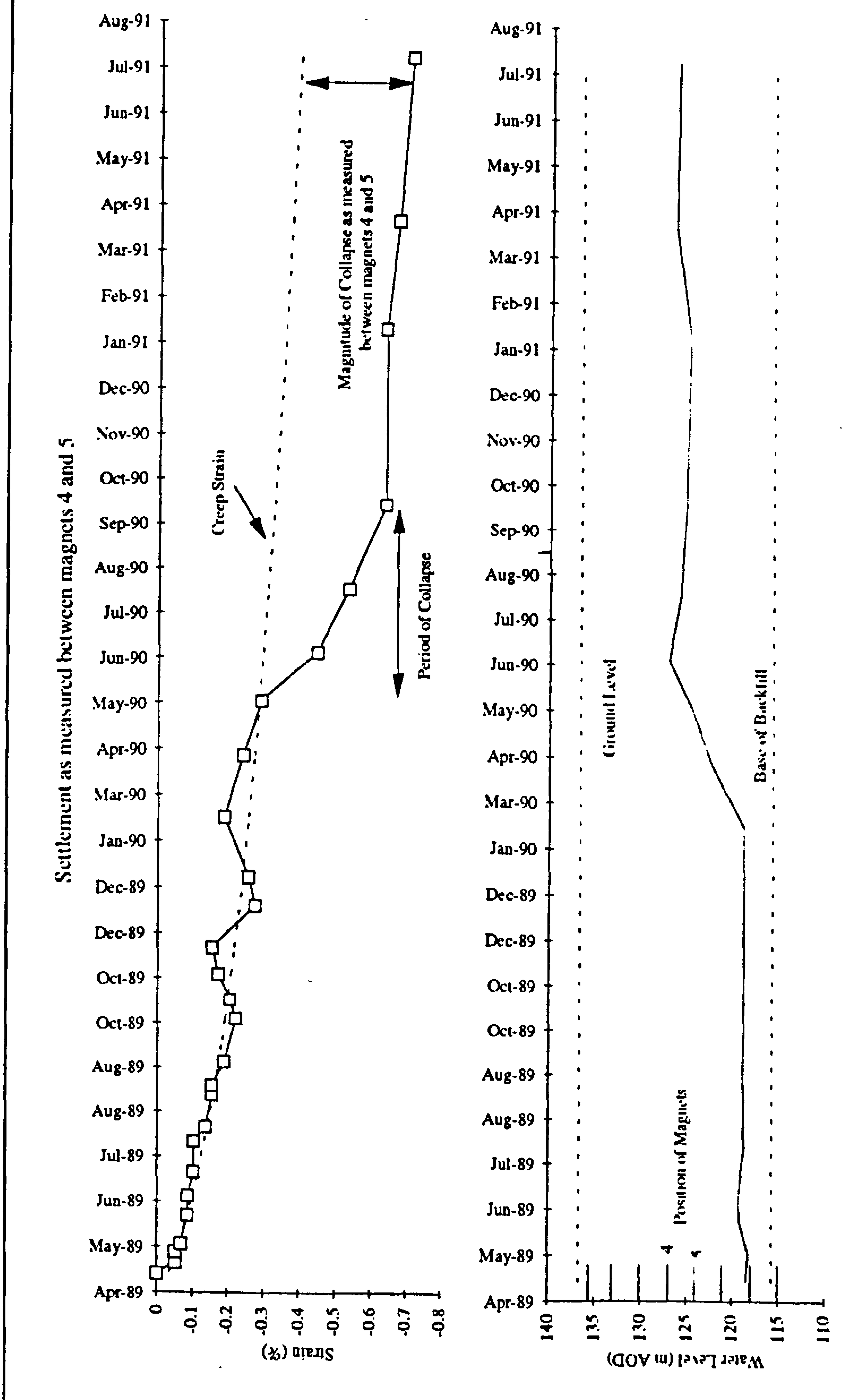


Figure 3.2. Newdale Extensometer Results Demonstrating the Calculation of Collapse Strain

commenced in 1986 and were completed in 1988. Excavation was by scrapers, face shovels, backactors and rippers.

Above the Deep Hard seam, excavated material was a weathered grey siltstone up to 13m thick. Between the Deep Hard and Piper seams, up to a maximum of 16m thickness, the excavated material was weathered mudstone with seatearths which changed downward into grey siltstone, becoming progressively sandier with depth. Underlying the siltstone was a persistent weathered sandstone; its average thickness was 3m. The base of the sandstone was usually about 4 m above the base of the Piper and was usually underlain by weathered siltstone. Point load tests on the sandstone indicated very low to low strength.

Material Type	In situ Testing				BS1377:1975 Test 12	
	Moisture Content (%)	Dry Density (Mg/m^3)	Air Voids (%)	Relative Compaction (%)	Optimum Moisture Content	Maximum Dry Density (Mg/m^3)
Weathered siltstone	9.9 [5-15] (84 results)	1.76 [1.6-2.1] (73 results)	-	91 [83-109]	10.4 [8-14] (29 results)	1.93 [1.69-2.05]
Weathered sandstone	11.5 [7-16] (58 results)	1.70 [1.56-1.87] (45 results)	-	89 [82-98]	12.4 [10-14] (18 results)	1.91 [1.84-1.97]
Weathered mudstone	10.3 [7-20] (77 results)	1.96 [1.42-2.11] (74 results)	-	99 [72-107]	12.6 [8-20] (24 results)	1.98 [1.68-2.03]

Table 3.1. Summary of the compaction monitoring data carried out at Barnabas.

The material excavated making up the backfill predominantly comprised of weathered siltstone and sandstone. It was on these two material types that compaction trials were carried out. The basis of the trials was to produce a compacted material having a dry density of 90% of the maximum dry density as measured by Test 12 described in BS1377:1975 - The British Standard Methods of Test for Soils for Civil Engineering Purposes. Using a 825B Tamping Roller, eight passes over a 500mm layer proved satisfactory for the siltstone and eight passes over a 300mm layer for the sandstone. A moisture content restriction of +/- 2% optimum moisture content was imposed for both materials. Hard sandstone was placed at the bottom of the excavation and mixed with siltstone to infill voids around the blocks. The weathered sandstone was easily fragmented and was placed in the upper 2m of the fill to enhance bearing capacity and drainage.

3.2.1 Compaction Monitoring

The average in situ compaction test results and laboratory test results are shown below in Table 3.1. In situ testing was carried out by sand replacement (large cylinder) BS1377:1975 Test 15B.

3.2.2 Instrumentation

Instrumentation involved the installation of 25 surface settlement monuments, 7 extensometers and 7 standpipe piezometers. These were installed and monitored, on average, some 60 days after the completion of backfilling. Monitoring results which have been made available to date cover an average period of some 1,300 days. The locations of the instrumentation can be seen in Figure B1.

3.2.2.1 Groundwater Recovery

The results of the ground water monitoring scheme generally indicate that a stable water table was attained from the start of monitoring but it cannot be concluded that this level is equal to pre-excavation levels as these levels are not known. Some slight variations in the water table were observed during monitoring. Piezometer 1 was dry from the start of monitoring, at the end of 1987, until the end of February 1988 at which time the groundwater level rose to a maximum of about 3m above the base of the backfill (35% of total fill thickness) and then fluctuated slightly before becoming dry again in June 1988. Piezometer 4 indicated a reasonably constant level of 2m above the base of the backfill but measured two rises of about 2 metres which were probably as a result of seasonal variations. Piezometer 7 measured a rise of 2.5 metres into the fill during June and July 1988.

3.2.2.2 Creep Strain

An average alpha value of 0.32 was obtained from surface settlement monument results with a s.d. of ± 0.23. This demonstrates that a considerable variation in measured alpha values was obtained and the overall average alpha value for the site is large in comparison to other study sites where a scheme of compaction has been carried out. The variation in alpha is considered to be the result of variations in the levels of compaction achieved during compaction. Insufficient detail as to the location of in-situ compaction results is available thus it is not possible to make a direct correlation between in-situ densities and alpha values; but as a considerable variation between measured in-situ densities and alpha values exists it is considered that such a correlation could be made given a complete set of monitoring data.

The high average alpha value achieved for the site is believed to be as a result of the low compaction targets set by the compaction specification. A target of 90% of the maximum dry density as determined by BS1377:1975 Test 12 was set whilst that recommended by the Department of Transport Specification for Highway Works (1986) is 95%. Also it is considered that as Test 12 is for light compaction it is not an appropriate comparison for material being compacted by large heavy plant as in the case of opencast backfill material.

Extensometer	Collapse Strain (%-age of layer thickness)	Layer Depth (m)	Ground Water (g) or Surface Water Infiltration (s)	Surface Response to Collapse at Depth (marker)
E1	0.90	5.5 - 7.5	s	Non
	0.40	7.5 -10.5	s	
E3	0.20	6.5 - 9.0	s	10mm (12)
	0.35	9.0 - 13.0	s	
E5	0.30	12.5 - 15.5	s	Non
	0.30	15.5 - 18.5	s	
E7	2.50	2.0 - 4.0	s	30mm (20) [1]
	0.80	7.0 - 9.5	s	
Average (s.d.) excluding E1 & E7	0.29 (0.06)	-	s	-
Average (s.d.) E1 & E7	1.15 (0.93)	-	s	-

Table 3.2. Summary of collapse strains as measured at Barnabas

[1] Response delayed by some 3½ months after collapse occurred at depth.

3.2.2.3 Collapse Strain

Periods of collapse settlement can be seen at a number of locations across the site at both surface settlement monuments and extensometers. As piezometer results indicate only a small proportion of the fill became inundated by rising groundwater it is considered that these collapse events were largely as a result of surface water ingress.

A summary of the measured collapse strains is given in Table 3.2 from which it can be seen that in two locations E1 and E7 considerable collapse strains where recorded. It is believed that at these two locations the backfill has a dry density equal to the lower end of the monitored range of values.

3.3 Bilston Opencast Coal Site

The opencast operations at Bilston commenced in March 1987 and were completed by July 1989. The site covered an area of some 38 ha with the excavation reaching a depth of 40 metres. Approximately 5.9 Mm^3 of overburden material was excavated consisting predominantly of Coal Measures strata and made ground. The Coal Measures consisted of mudstones, siltstones and sandstones. The mudstones and siltstones were mostly weak and weathered breaking down easily under heavy plant. The sandstones were believed to be extensively fractured and weathered from the pre-excavation investigation, however, during operations they were found to be more massive in places. The made ground consisted of blast furnace slags, ash and clinker and extensive foundations and other structures.

Material Type	In-situ Testing				BS1377:1975 Test 13	
	Moisture Content (%)	Dry Density (Mg/m^3)	Air Voids (%)	Relative Compaction (%)	Optimum Moisture Content	Maximum Dry Density (Mg/m^3)
Grade I Material	10.3 (333 tests)	1.902 [1.70-2.10] (333 tests)	8.9[1] (128 tests)	96[2] [86-106]	8.0[1]	2.05[1]
Grade II Material	12.0 (84 results)	1.80 [1.60-2.00] (84 results)	13.1[1] (41 test)	91[2] [81-101]	9.2[2]	1.98[2]

Table 3.3. Summary of the compaction monitoring data carried out at Bilston.

[1] Mudstone only. [2] All materials. Note: Approximately half of the backfill is mudstone.

Compaction of the backfill was carried out in accordance with the Department of Transport Specification for Road and Bridge Works (1976) which was relaxed for material below 30 metres by allowing for a 50% increase in layer thickness. Compaction plant used included a CAT 825C tamping foot compactor, a Stothert and Pitt T182 vibratory towed roller and a CAT 653 vibrating smooth wheeled roller.

3.3.1 Compaction Monitoring

Field density tests were carried out almost every day on material compacted within the void. They consisted of both sand replacement tests (BS1377:1975 Test 15B) and Nuclear Density Tests. Laboratory tests regularly carried out consisted of particle size analysis, plastic indices, compaction tests Nos. 12 and 13 (BS1377:1975), Californian Bearing Ratio, specific gravity and moisture content determination. The results of the density tests are summarised in Table 3.3.

3.3.2 Instrumentation

47 surface settlement stations, 9 extensometers (with piezometer tips) and 10 piezometers were installed and monitored between 12 and 24 months after the completion of compaction operations. Monitoring results made available to date cover an average period of 400 days. The locations of the instrumentation can be seen in Figure B2.

3.3.2.1 Groundwater Recovery

Water levels recorded by piezometers within the void indicate a water table of approximately 118 mAOD with a seasonal variation of about ± 2 metres. The seasonal variation measured is consistent with that measured by piezometers outside the void which to the south of the Lanesfield Fault also record a water table of about 118 mAOD. Prior to excavation this was measured at 121 mAOD and it can therefore be seen that the water table within the void had risen to within 3 metres of the level prior to the commencement of monitoring. During the monitoring period, excluding the seasonal variation, groundwater levels remained relatively constant with the majority of the groundwater table rise having occurred before monitoring commenced.

3.3.2.2 Creep Strain

The site can be divided into three distinct zones as shown in Figure B2 and summarised below:

i) the area subjected to a 15 metre high overburden mound left in place for approximately 12 months

ii) the central unsurcharged area

iii) the unsurcharged final cut area which was subject to various modifications including the accommodation of large lumps above 30m depth, of rockfill below 20m and a larger proportion of fine washery discard than incorporated elsewhere.

The average alpha value determined for the central area is 0.24 (0.08 s.d.) and that for the final cut, 0.24 (0.20 s.d.). It was not possible to determine a value for the surcharged area as heave movements masked any creep movement that may have been occurring. The average alpha values for the central area and the final cut area are therefore shown to be the same with the latter area showing a greater variation in alpha. This may be indicative of the relaxation in the compaction specification for this area which was necessary for the accommodation of large lumps, rockfill and fine washery discard all of which will lead to a more heterogeneous backfill in the final cut area than in the central area.

3.3.2.3 Collapse Strain

There is an absence of any periods of collapse strain from the extensometer results. This is to be expected, in terms of collapse as a result of groundwater rebound, as the groundwater table is shown to have largely re-established itself by the time monitoring commences. In terms of surface infiltration, again there is little evidence of any collapse from the extensometer results however there is some slight indication of relatively small collapse events from the surface settlement station results. The scale and apparent absence of collapse from surface infiltration is believed to be as a result of the delay between completion of compaction operations and the commencement of settlement monitoring. The majority of the collapse due to surface infiltration having occurred prior to monitoring.

3.3.2.4 Heave

Heave movements are evident within the surcharged area, they are however small with a recorded maximum of 10mm and an average of 4mm. The surcharged area was the first to be excavated and compacted and as such the greatest delay between completion and commencement of monitoring occurred within this area, approximately 24 months. It is therefore considered that the majority of the heave associated with the removal of surcharge was missed by the monitoring scheme.

3.3.2.5 Differential Settlements

A number of surface settlement markers where positioned such they could measure settlements across internal/side walls i.e. batter strings 1 to 10 Figure B2. Batter strings 1, 2 and 3 located within the surcharged area measured insignificant differential settlements which is probably as a consequence of both the large delay between completion of compaction and commencement of monitoring and the presence of a surcharge. Batter string 5, within the central area, shows settlement behaviour as would be expected, with settlements increasing with increasing fill depth across the batter. Differential settlements are still small though, with a maximum of only 1:2500. Batter strings 6, 9 and 10 are all located within the final cut and only batter string 10 measures settlements that would be expected. Batter string 6 measures its greatest settlement over the shallowest fill giving differential settlements of between 1:900 and 1:1300. Batter string 9 whilst measuring its largest settlement over the greatest fill depth, gives twice as much settlement at a station underlain by 6 metres of fill than at the adjacent station underlain by 22 metres of fill; differential settlements at batter string 6 range from between 1:1250 and 1:1800. Batter string 10 measures very small differential settlements of between 1:5500 and 1:10000 but does measure settlements increasing with increasing fill depth.

The settlement measured at batter strings 6 and 9 show that factors other than fill depth have significance on the magnitude of the absolute settlement. Only with well compacted homogeneous fills would one expect fill depth to be the only consideration. It can therefore be considered that fill heterogeneity in the vicinity of the batters causes the anomalous settlement behaviour seen. This can be expected due to difficulties of compacting material up to and against high/side walls and the possible build up of shear stresses between walls and fills, producing a slowing of settlement. The heterogeneous nature of the backfill within the final cut area will also contribute to this apparent anomalous behaviour.

3.4 Blindwells Opencast Coal Site

Blindwells opencast coal site lies 8 km East of Edinburgh and 1 km from the sea in the Lothian region of Scotland. Excavation of the site began in June 1978, the total site area covers 368 ha and was worked to a depth of approximately 60 metres with the removal of an estimated 55 million tons of overburden. The whole of the site was de-watered prior to the start of excavation with the water table being held down below the maximum excavation depth.

Material excavated consisted of boulder clay, varying in thickness from between 5 to 25 metres, with the deeper deposits contained within drift channels containing a large proportion of sandy material with sand and gravel lenses and fairly massive mudstones, siltstones, and sandstones of roughly equal proportions. The argillaceous rocks range from dark seat-earths and marine beds to sandy mudstones. The sandstones are generally white and vary enormously in grain size and hardness.

To accommodate the A1 trunk road (Trenent By-Pass) backfill material within 16 meters of final road level beneath the line of the proposed route was to be compacted. The contract specification called for the fill to be placed in layers not greater than 1m thickness between 16 and 6 m below finished road level (bfrl), not greater than 450mm thickness between 6 and 1.5 m bfrl and not greater than 150mm thickness between 0.5 and 1.5m bfrl. The fill was placed using a dozer not exceeding 14 tonnes and compacted using an 8 tonne towed vibratory roller with not more than eight passes per layer in bands of 2 m or less width.

3.4.1 Compaction Monitoring

No compaction monitoring was carried out during site operations, but borehole samples taken after compaction gave average air voids and dry densities values for compacted backfill of 29.5% and 1.57 Mg/m^3 respectively. Samples taken beneath and to the side of the compacted material gave average air void and dry density values of 25.4% and 1.56 Mg/m^3 respectively. These values indicate no improvement in density of the compacted material but this is probably due to sample disturbances as drilling and sampling difficulties were experienced.

3.4.2 Instrumentation

Four extensometers were installed within the backfill in two pairs with each pair lying on a line perpendicular to the proposed road. One extensometer from each pair was installed within the compacted material whilst the other lay within uncompacted material. A number of surface settlement stations were also installed along the length of the proposed road. Locations of the instrumentation installed are shown in Figure B3. Instrumentation was installed, on average, some 700 days after the completion of backfilling. Monitoring results made available to date cover an average period of 1300 days.

3.4.2.1 Groundwater Recovery

The instrumentation did not include any piezometers or standpipes, therefore there is no indication of the scale and timing of the groundwater recovery.

3.4.2.2 Creep Strain

The average alpha value obtained from the surface settlement station results is 1.0 ± 0.56 s.d., a value more indicative of uncompacted backfill than compacted. This is to be expected though, as a greater proportion of the fill is uncompacted than compacted. The influence of compaction on the alpha value can be seen in Figure 3.3, where the

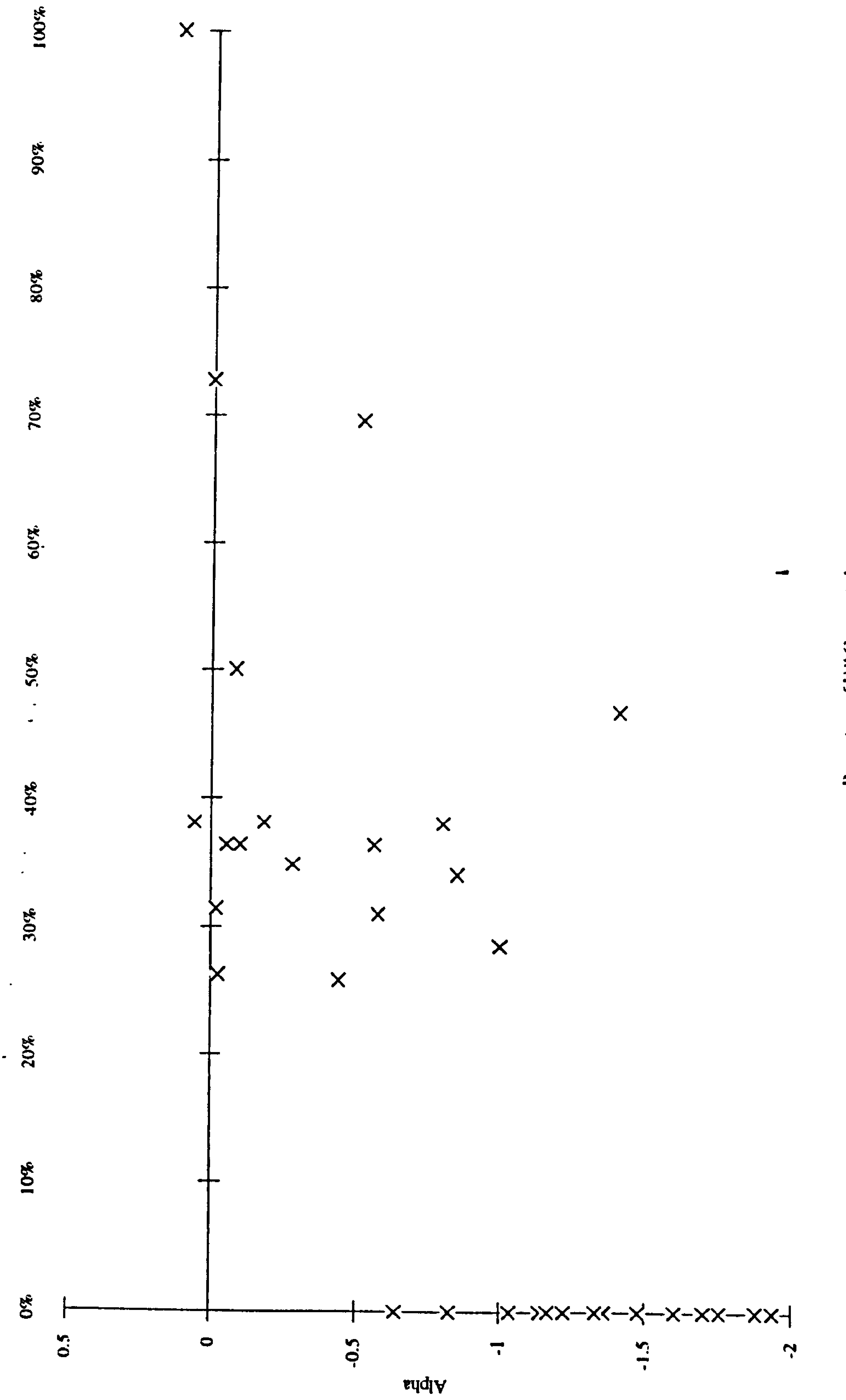


Figure 3.3. Blindwells. Relationship Between Alpha and Percentage of the Fill Compacted

percentage of fill compacted beneath a settlement station is plotted against the alpha value obtained at that location. Thus indicating the beneficial effect of compaction on the magnitude of creep strain as summarised in Table 3.4.

Percentage of Fill Compacted	Alpha (minimum to maximum)
0%	1.4 (0.64 to 1.94)
30-40%	0.4 (0.06 to 1.40)
>70%	0.1 (0.12 to 0.51)

Table 3.4. Alpha values compared to percentage of fill compacted

3.4.2.3 Collapse Strain

Examination of the surface settlement station results show periods of accelerated settlement, indicative of collapse, occurring in and around October during the years of 1984, 1985, 1986 and 1988. As no information is available on groundwater table depths or rainfall records, it is not possible to correlate these movements with increases in moisture content within the backfill. However, due to their cyclic behaviour and the high rainfalls typically associated with the time of year the movements are occurring it is postulated that they are as a result of surface water ingress.

Examination of the extensometer W8 indicates a period of accelerated settlement coincident with that of the surface settlement markers. The movement occurs both near surface and at depth therefore a rising groundwater table may be responsible for accelerated settlements occurring at depth. The magnitude of the movement is summarised in Table 3.5.

Extensometer	Collapse Strain (%-age of layer thickness)	Layer Depth (m)	Ground Water (g) or Surface Water Infiltration (s)	Surface Response to Collapse at Depth (marker)
W8	0.15	7.0 - 12.0	s	No adjacent settlement makers
	0.10	49.0 - 57.0	g	

Table 3.5. Summary of Collapse Strains as Measured at Blindwells

3.5 Dixon Opencast Coal Site

Dixon site is situated in Brimington, a north east suburb of Chesterfield, Derbyshire. The site covers an area of approximately 20 ha and was excavated to a maximum depth of 100m. The site is to be restored for industrial development (Area A) and the proposed A619 Brimington By-Pass will pass through Areas A and C. Approximately 16 million m^3 of material was excavated of which approximately 4 million m^3 will be compacted in the industrial zone and 1.6 million m^3 within the compaction zone for the By-Pass.

Material excavated consisted mainly of drift and Coal Measures. The drift material was comprised of gravel alluvium and silty clays, these deposits were not intended for use in the backfill compaction. The Coal Measures strata between the seams comprised of mudstones and silty mudstones with occasional sandstones and siltstones. Drilling prior to excavation estimated proportions to be 74% mudstone, 16% siltstone and 9% sandstone, the remaining percentage being made up of ironstone and seatearth. Made ground, mainly foundry sand and slag, forms a layer, on average 5.3m thick, on top of the alluvium and large tips. A proportion of this material is to be incorporated within the compaction zone of Area A.

Area of Site	In situ Testing			BS1377:1975 Test 12	
	Moisture Content (%)	Dry Density (Mg/m^3)	Relative Compaction (%)	Optimum Moisture Content	Maximum Dry Density (Mg/m^3)
A	8.9 [6.2-12.0]	1.97 [1.87-2.05]	98 [94-103]	10.5	2.01
C	8.4 [7.3-12.0]	1.86 [1.80-1.95]	94 [91-98]	10.5	1.98

Table 3.6. Summary of the compaction monitoring data carried out at Dixon.

A method specification was employed based upon the Department of Transport Specification for Road and Bridge Works (1976). The method of compaction for the three material types (mudstones, weathered and thinly bedded sandstone and foundry sand and slag) was determined from compaction trials.

3.5.1 Compaction Monitoring

Field density tests were carried out on a regular basis as backfilling progressed. In-situ density tests were carried out using BS1377:1975 Test 15 and relative compactions were determined from comparison with laboratory tests carried out in accordance with BS1377:1975 Test 12. Averages for the whole site are summarised in Table 3.6.

3.5.2 Instrumentation

Settlement and groundwater monitoring was carried out by means of 21 simple surface levelling stations, 3 magnetic extensometers and 3 piezometers commencing, on average, 40 days after the completion of compaction operations. Monitoring results available to date cover an average period of 1200 days in area C and 700 days in area A. The locations of the instrumentation can be seen in Figure B4.

3.5.2.1 Groundwater Recovery

Monitoring commenced between 1 and 3 months after the completion of compaction whilst pumping operations were still in progress in other areas of the site. The influence of this pumping is clearly shown by the piezometers at locations R2 and R3. The timing of the shut down and commencement of pumping operations correlates clearly with a rising and falling groundwater table respectively. Piezometer R4 remained dry during monitoring. On the completion of pumping during June to August 1991 water levels are seen to rise for the following 6 to 8 months of monitoring thereafter. The rate of rise is initially about 2 m/month for the first 2 months and then it is seen to decrease to about 1 m/month thereafter. This is to be expected as the water table begins to approach its pre-excavation levels, these were however not attained during monitoring. At location R2 the water level rose to within 40% of the pre-excavation level within 9 months and at location R3 to within 50% within 7 months.

3.5.2.2 Creep Strain

Average alpha values of 0.28 (0.08 s.d.) and 0.17 (0.08 s.d.) were obtained from the surface settlement marker results in areas A and C respectively. These values are believed to be indicative of a well compacted backfill and the small variation in the alpha values obtained shows the backfill in both areas to be relatively homogenous.

3.5.2.3 Collapse Strain

Collapse strains are evident from both the surface settlement marker and the extensometer results. It is believed that these are as a result of inundation due to a rising groundwater table as each observed collapse 'event' correlates clearly with a rise in the groundwater table. There is little evidence of any collapse due to surface water infiltration.

Extensometer	Collapse Strain (% of layer thickness)	Layer Depth (m)	Ground Water (g) or Surface Water Infiltration (s)	Surface Response to Collapse at Depth (marker)
R2	0.45	19.1 - 25.7	g	60mm (M)
	0.45	25.7 - 31.5	g	
	0.20	25.7 - 31.5	g	Non
R3	0.20	23.0 - 25.0	g	Non
	0.52	23.0 - 25.0	g	25mm (J)
Average (s.d.)	0.36 (0.15)	-	g	-

Table 3.7. Summary of Collapse Strains as Measured at Dixon (Area C)

3.5.2.4 Heave

Heave was recorded within area A at stations 7, 8 and 9 for a period of 3 months from the start of monitoring. This is believed to be due to a combination of elastic rebound following the completion of the compaction operation and near surface saturation due to surface water ingress and as such is considered to be typical behaviour where compaction is carried out. It is however behaviour that is rarely seen from the settlement data obtained from the study sites as monitoring generally commences after this period of heave has occurred.

3.6 Flagstaff Opencast Coal Site

The Site is located on the eastern outskirts of Ashby-de-la-Zouch in Leicestershire. The alignment of the A42 route runs through the site and an interchange has been constructed within the Site's boundaries. The Site is divided into three areas of opencast excavation A, B and C as shown in Figure B5. Excavation in area A commenced in mid 1985 with backfilling of the Controlled Zone completed by June 1986. Backfilling within the controlled zone of area B was completed by November 1986. Area C was the last to be excavated and backfilled. The depth of the excavation was up to 28 metres and the site covered some 126 hectares. Excavation was split into three separate zones. The overburden generally consisted of Lower Coal Measures strata; mudstones siltstones and occasional sandstones. The mudstones ranged in weathering grade from a clay through to moderately weak mudstone.

Three types of backfilling and compaction were specified at the site. Within the Controlled Zones beneath the sections of road and beneath proposed development, backfilling and compaction was in accordance with the Department of Transport Specification for Road and Bridgeworks (1976). Over the remainder of the site the fill was brought up in lifts no greater than either 5m or 1m, by layer tipping and generally no compaction was applied. However, because of the close proximity of the 1m tipping zone to the controlled zones the contractor chose to conduct some compaction of layers less than 1m thickness, in these areas.

Compaction plant used by the Contractor consisted of the Stothert and Pitt 72T and T182 vibratory rollers and CAT 825C compactors.

A large proportion of the mudstone was classified unsuitable as it was drier than the moisture limits specified. It was impractical for this material not to be incorporated into the controlled zone therefore the specification was modified allowing end product compaction to be carried out on material dry of the specified limits. The end product was set at a dry density of 1.85 Mg/m^3.

Material Type	In situ Testing			BS1377:1975 Test 13	
	Dry Density (Mg/m^3)	Air Voids (%)	Relative Compaction (%)	Optimum Moisture Content	Maximum Dry Density (Mg/m^3)
Controlled Zone Only	1.93 [1.85-2.05]	5.0 (2.0-9.0)	100 [96-106]	12.0	1.93

Table 3.8. Summary of the compaction monitoring data carried out at Flagstaff.

Note: Average air voids value for dry mudstones was 9.0%

3.6.1 Compaction Monitoring

Field density tests were carried out on a regular basis as backfilling progressed. In-situ density tests were mainly carried out using a nuclear density gauge but a small number of tests were carried out in accordance with BS1377:1975 Test 15. Relative compactions were determined from comparison with laboratory tests carried out in accordance with BS1377:1975 Test 13. Averages for the whole site are summarised in Table 3.8.

3.6.2 Instrumentation

Surface settlement stations, extensometers, piezometers and standpipes were installed and monitored, on average, some three months after the completion of compaction operations. Up to 3 years monitoring results have been made available for examination. The location of the instrumentation can be seen in Figure B5.

3.6.2.1 Groundwater Recovery

Monitored groundwater levels generally show only very small rises indicating that a large proportion of the groundwater recovery occurred prior to monitoring. The groundwater level is shown to have reached between 5 and 10 metres of its pre-excavation level and is therefore expected to continue rising, but only at a very slow rate, similar to that which has already been monitored.

3.6.2.2 Creep Strain

An average alpha value of 0.19 (0.18 s.d.) was obtained within the compacted zone. This compares with a value of 0.22 (0.11 s.d.) for the 1 metre tipping zone, there was insufficient data for a value to be obtained from the 5 metre tipping zone. It is considered that the low value obtained for the 1 metre zone is not typical of backfill material placed in this manner. This is believed to be as a consequence of an insufficient data set, data enabling the calculation of alpha was obtainable from only 4 settlement markers within this zone as compared to 26 within the compacted zone.

3.6.2.3 Collapse Strain

Periods of accelerated settlement can be seen in both the extensometer and surface settlement marker results. As there is generally little groundwater table rise occurring during the monitoring period it is considered that the majority of the observed collapse is due to surface infiltration. It is apparent from the results that the majority of this collapse strain is complete within 19 months from the completion of backfilling. A summary of the collapse strains measured from the extensometer results are shown in Table 3.9 indicating the considerably larger collapse strains that occurred within the 1 metre tipping zone than in the compacted zone.

3.7 Ketley Brook Opencast Coal Site

The Ketley Brook opencast site lies in the County of Shropshire within the District of Wrekin, about 1km north of Lawley. The site occupies about 50 ha and is split into

Extensometer	Collapse Strain (%-age of layer thickness)	Layer Depth (m)	Ground Water (g) or Surface Water Infiltration (s)	Surface Response to Collapse at Depth (marker)
E9[1]	0.35	8.0 - 11.0	s	Insufficient data
	0.40	17.5 - 20.0	g	
E10[1]	0.15	12.0 - 15.0	s	Insufficient data
	0.10	15.0 - 18.0	s	
	0.30	18.0 - 20.5	s	
E2[2]	0.60	1.0 - 4.0	s	Insufficient data
	0.60	7.0 - 10.0	g	
	0.25	13.0 - 16.0	g	
	0.70	19.0 - 21.0	g	
E7[2]	1.20	1.0 - 3.0	s	30mm (44) [3]
	0.50	3.0 - 6.0	s	
	0.40	6.0 - 10.0	s	
	0.70	10.0 - 12.0	s	
Averages (s.d.)	0.23[1] (0.12)	-	s	-
	-		g	
	0.57[2] (0.40)		s	
	0.52[2] (0.24)		g	

Table 3.9. Summary of Collapse Strains as Measured at Flagstaff

[1] compacted zone, [2] 1 metre tipping zone
[3] Response delayed by some 3½ months after collapse occurred at depth.

two areas, A (uncompacted) and B (compacted). Compaction operations commenced in January 1989 and were completed by about June 1990.

The material excavated consisted broadly of drift, old opencast backfill and Coal Measures. The drift varied in thickness from about 2 to 4m in the south to over 20m in the north and east. It consisted of boulder clay or till interleaved with fluvio-glacial sand and gravel's. The clay was stiff in-situ but was easily softened by water issuing from adjacent 'running sand'.

Old opencast backfill consisted of firm to stiff silty clay with sand, abundant gravel sized fragments of coal, sandstone and shale and occasional cobbles or boulders and was found to a maximum depth of 28 metres.

Siltstones and sandstones represented a significant proportion of the total volume of Coal Measures if not the majority. Their hardness and blocky nature meant that a large proportion of it was unacceptable 'Coarse Granular Material' due to its large particle size and was therefore accommodated within a 5 metre tipping zone. The rest of the measures consisted of mudstones and seatearths. The seatearths were stockpiled outside the compaction zone.

Faulting within the area is dominated by the Ketley Fault which occupies a zone some 22 to 45 metres in width with a down throw of approximately 40m. This coincides with the alignment of batters especially in the southern area of Area B which resulted in the batter face comprising a smooth face of hard rock.

Material placed within Area B was compacted in accordance with the Department of Transport Specification for Highway Works (1986) with a method specification being chosen. Compaction trials were carried out on all material types to be compacted prior to their inclusion within the compaction zone. Compaction plant employed during trials consisted of a Stothart & Pitt T182 towed vibratory roller, a CAT 825 four wheeled tamping roller, and CAT 653 and VIBROMAX 1802 self propelled vibratory rollers.

Modifications to the specification were necessary during operations to take into account the differences between opencast operations and highway works and the variability of the material to be compacted. This resulted in the inclusion of some unacceptably soft clay material below 20m depth within the compaction zone. Also of note is that an area of fill was placed rapidly by 24 hour working to buttress a potentially unstable slope. It is considered that due to the urgency with which the backfill was placed, compaction targets may have been compromised resulting in slightly less well compacted backfill in this area than elsewhere within the compaction zone. Figure B6 shows the extent of both of these areas.

3.7.1 Compaction Monitoring

Field density tests were carried out on the compacted materials while backfilling was progressing. These took the form of both sand replacement tests (BS 1377 Test 15B) and nuclear density tests. Summaries of both the in-situ test results and the laboratory results are show in Table 3.10. This shows that, generally, adequate levels of compaction were achieved with the an average air voids value for all material types of 6%.

Material Type	In situ Testing			BS1377 Test 13	
	Dry Density (Mg/m^3)	Air Voids (%)	Relative Compaction (%)	Optimum Moisture Content	Maximum Dry Density (Mg/m^3)
Class 2A	1.80 [1.62-2.02]	5.0 [0 - 18]	-	-	-
Class 2B	2.03 [1.80-2.15]	9.3 [5 - 16]	-	-	-
Class 2C	1.88 [1.65-2.10]	5.3 [0 - 12]	-	-	-
All Materials	1.87 [1.62-2.15]	6.0 [0 - 18]	93 [80 - 107]	10.2 [6.6 - 15.4]	2.02 [1.86-2.24]

Table 3.10. Summary of the compaction monitoring data carried out at Ketley Brook.

3.7.2 Instrumentation

Settlement and groundwater monitoring was carried out by means of simple surface levelling stations, magnetic extensometers and piezometers. Instrumentation from which results have been made available for analysis are located mainly within the area of 24 hour working or soft clay infill as mentioned above. Monitoring commenced some 2 months after the completion of all compaction operations. The location of the instrumentation can be seen in Figure B6.

3.7.2.1 Groundwater Recovery

The monitored groundwater level is approximately 130 mAOD across the site dropping to about 160 mAOD in the southern corner. The levels measured are reasonably constant during the period of monitoring with no indication of any rapid fluctuations, except at the piezometer located within extensometer E1. This shows large fluctuations in the order of a 30m fall in 5 months. Such a change in water level is unlikely to represent the true ground water table and therefore may be more indicative of perched water. Piezometers 1 to 9 give some evidence for the presence of perched water with the upper piezometer tips measuring a slightly higher water table than the lower tips. This may be as a result of impermeable layers within the backfill or as a consequence of uneven groundwater recharge from the perched water within the natural strata.

3.7.2.2 Creep Strain

Average alpha values for settlement stations within the zones of deposition of unacceptable clay infill and 24hr working and those outside of these zones are 0.23 (0.07 s.d.) and 0.12 (0.06 s.d.) respectively. These values are believed to be indicative of compacted backfill however it can be tentatively concluded that the inclusion of soft clay below 20m and the enforcement of 24hr working produced a backfill having inferior creep characteristics than backfill placed without these modifications. There is however insufficient data from settlement stations outside of these zones to fully substantiate this reduction in backfill quality.

3.7.2.3 Collapse Strain

There is some evidence of minimal collapse occurring at the surface settlement markers with the majority showing two distinct periods of accelerated settlement. As there is no corresponding rise in the groundwater table during these periods it is assumed that these settlements are as a result of surface water infiltration. The absolute magnitude of these settlements is in the region of 10mm.

There is little evidence of any strains on inundation from the extensometer results, groundwater measurements indicate a stable level, therefore any strains due to groundwater recovery may have been missed by the monitoring scheme having occurred whilst this stable level was being attained.

3.7.2.4 Heave

Small heave movements where recorded at two locations; at settlement marker 4B32 and at extensometer E4. The depth of backfill beneath station 4B32 was only 3m and as such monitoring commenced at this location relatively soon after the placement of the full depth of backfill. The measured heave of 8mm is therefore believed to be as a result of elastic rebound following compaction and surface water ingress heave. At location E4, the fill had been surcharged prior to the installation of the extensometer and the resulting heave is believed to be as a response to this surcharge. Negligible creep movements were monitored after heave movements finished.

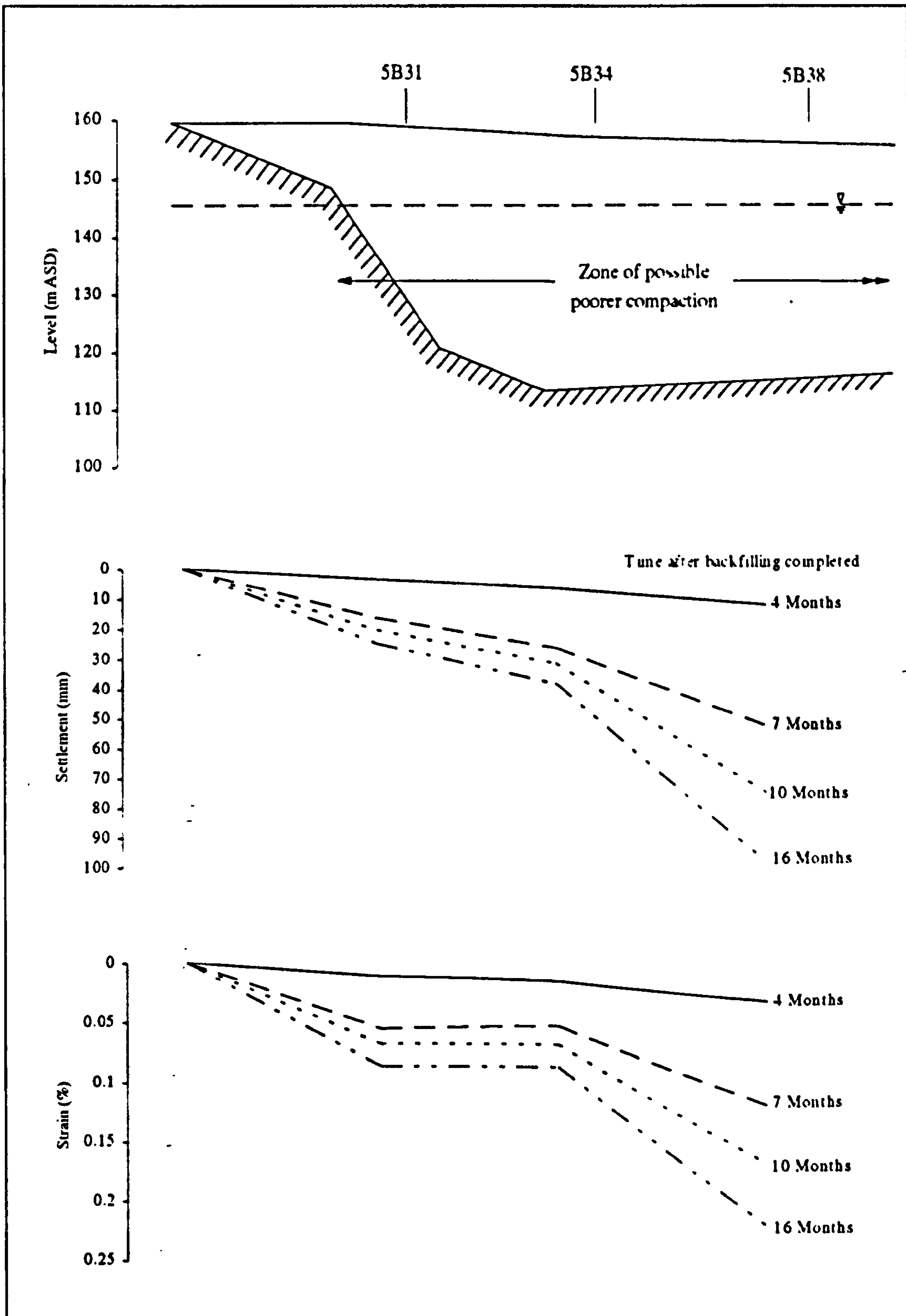


Figure 3.4. Ketley Brook differential settlements, markers 5B31 to 5B38

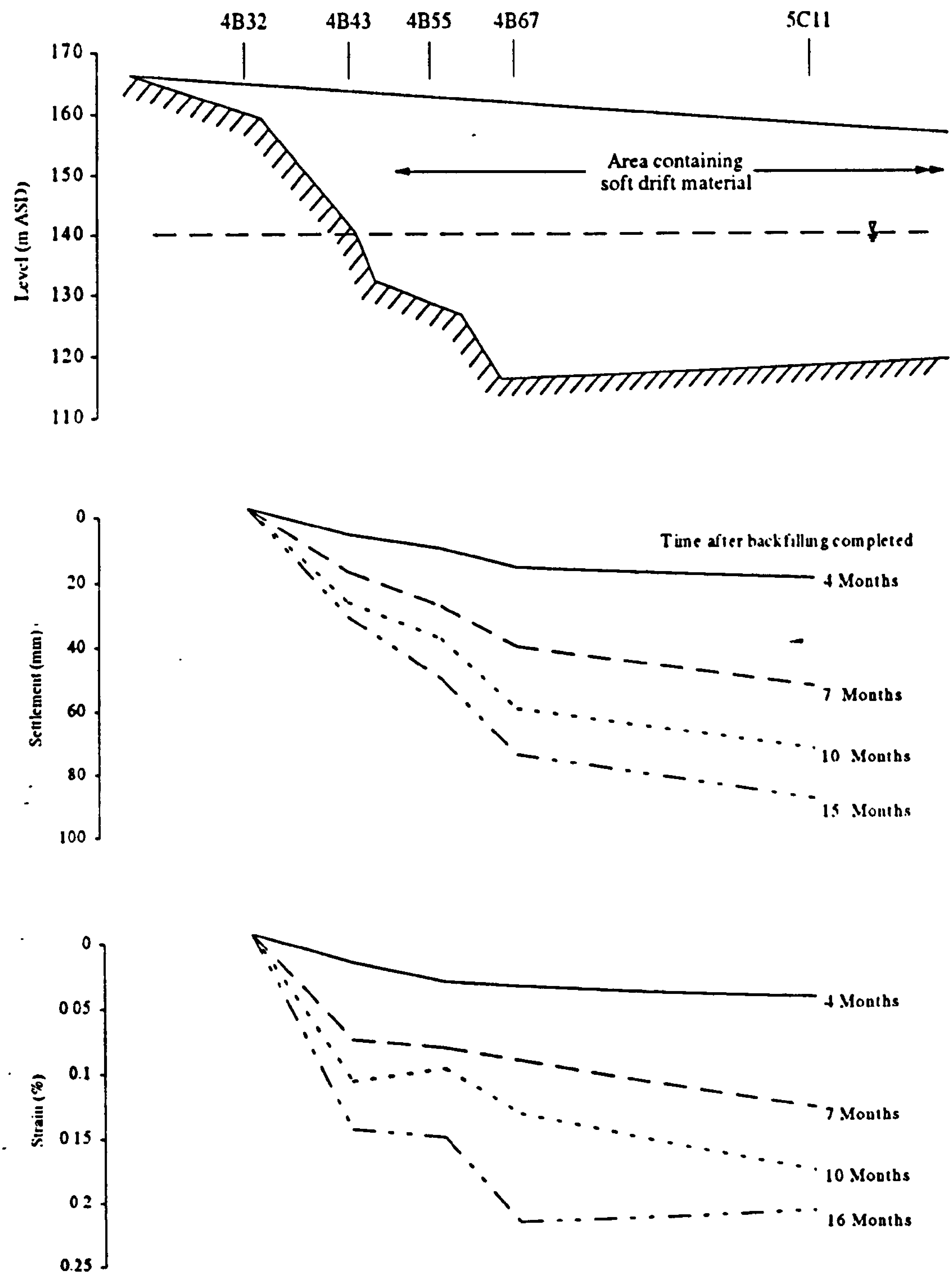


Figure 3.5. Ketley Brook differential settlements, markers 4B32 to 5C11

3.7.2.5 Differential Settlements

Differential settlements are measured by two traverses of settlement markers crossing the side wall of the excavation. Figures 3.4 and 3.5 summarise the movements observed. The magnitude of the differential settlements measured between 4B32 and 4B67 will be accentuated by the fact that not only does fill depth increase along the traverse but the fill beneath the traverse changes to that which contains an unacceptable clay layer.

3.8 Lounge Opencast Coal Site

Lounge was worked in two separate areas, A and B, totalling some 136 ha reaching depths of up to 46 metres. Areas A and B were worked from August 1986 and April 1987, with compaction operations being completed by October 1987 and June 1990 respectively. Material excavated consisted mainly of Lower Coal Measures strata although a significant volume of Permo-Triassic (sand to moderately strong sandstone) and Glacial (very stiff silty sandy clay with gravel) deposits were encountered. The Measures strata consisted of highly weathered, very weak mudstone near surface to a fresh, moderately strong mudstone with interbedded strong siltstone and sandstone horizons, typically up to 1 metre thick, at depth.

Two distinct zones of backfilling can be identified at the site, backfill placed beneath the proposed highway corridor which was compacted on placement, and backfill placed over the remainder of the site which was simply placed in lifts no greater than 5 metres by layer tipping. The majority of the backfill material (98% volume) placed in the compaction zone was compacted to an end product specification with a target dry density of 2.00 Mg/m^3. The remainder, material with a natural moisture content greater than optimum, was compacted to a method specification base on the Department of Transport Specification for Road and Bridge Works (1976).

3.8.1 Compaction Monitoring

Compaction monitoring was carried out on a regular basis to ensure target dry densities were being achieved for materials compacted according to the performance specification. In situ dry densities were calculated using the in situ bulk densities obtained from a nuclear density gauge and moisture contents from samples taken from each test site. The accuracy of the nuclear density gauge was periodically checked by comparisons with values obtained from sand replacement density tests (BS1377:1975 Test 15) and large scale water replacement tests. Laboratory tests, also carried out on a regular basis, included liquid limits, particle size distribution and specific gravity, the latter being used in the calculation of air voids.

Area of Site	In situ Testing				BS1377:1975 Test 13	
	Moisture Content (%)	Dry Density (Mg/m³)	Air Voids (%)	Relative Compaction (%)	Optimum Moisture Content	Maximum Dry Density (Mg/m³)
Area A	8.5 [6.5-11.6]	2.03 [1.95-2.10]	8.4 [4-12]	110	11.0	1.85
Area B	8.9 [6.0-14.8]	2.04 [1.95-2.12]	6.3 [0-10]	110		

Table 3.11. Summary of the compaction monitoring data carried out at Lounge.

The results of the compaction monitoring are summarised in Table 3.11, from which it can be seen that the target density of 2.00 Mg/m³ was achieved by the majority of the material within the compaction zone. Any areas where compaction was deemed inadequate from the monitoring results, further work was carried out such that the requirements of the specification were met.

3.8.2 Instrumentation

Surface settlement markers, extensometers, piezometers and standpipes were installed along the length of the proposed highway corridor so that ground movements could be monitored at surface and at depth. The location of the instrumentation can be seen in Figure B7. A period of between 1 and 6 months elapsed between completion of backfilling and installation of instruments, with a typical interval of 2 months. Monitoring results made available to date, cover a period of up to 1,000 days, with an average period of 500 days.

3.8.2.1 Groundwater Recovery

The monitored water levels obtained across the site indicate that generally the pre-excavation water levels have not been attained and that groundwater recharge has not fully occurred. It must be noted though that the site is in the vicinity of a local watershed and that the base of the excavation dips towards worked coal seams east of the site which may be acting as drains to the whole area. The slight depression of the water table may therefore be a permanent feature.

3.8.2.2 Creep Strain

The average alpha value obtained for the whole site from the surface settlement marker results is 0.30 (0.14 s.d.). Considerable collapse settlement was recorded at the majority of settlement markers which in some cases dominated the settlement results. This led to some difficulty in calculating alpha as the collapse component must be removed from the calculation. This may have lead to an overestimate of alpha in cases where the collapse component could not be completely removed. The value of 0.3 is however only slightly higher than the range expected for compacted backfill.

3.8.2.3 Collapse Strain

Periods of considerably accelerated settlement, indicative of collapse, have been recorded across the site at both the settlement marker and extensometer installations. As the groundwater table remains constant during the monitoring period it is considered that these settlements are as a result of surface water infiltration. Figure 3.6 shows the periods over which accelerated settlements have commenced, at various locations across the site, against rainfall data, averaged over the preceding 3 weeks. The precise time at which accelerated settlements commence cannot be determined due to the frequency with which settlement readings have been taken. It can be seen that some correlation exists between the commencement of increased settlement and increases in rainfall. Examination of extensometer results revealed that increased settlements occurred throughout the full depth of the fill or within near surface layers which is as would be expected if surface water draining down through the fill were responsible for these settlements.

The correlation between rainfall and settlement is by no means complete, but bearing in mind the variable permeability and air voids of the fill and hence variable flow of water through it, this is not to be unexpected. Also, perched water tables found within the Upper Lount Coal and overlying jointed sandstone and within the Sherwood Sandstone will provide another source of infiltration into the backfill from the surrounding undisturbed ground.

A summary of the collapse strains as measured from the extensometer installations are summarised in Table 3.12. It is of note that the absolute collapse settlements measured at adjacent settlement markers is in some cases considerably larger than that measured at depth by the extensometers. This may be indicative of the fact that the majority of the collapse is occurring very close to the surface above the top magnet and as such is not measured by the extensometer or that a considerable variation in collapse is

Figure 3.6. Comparison of the Commencement of Periods of Accelerated Settlement with Rainfall Data

Extensometer	Collapse Strain (% of layer thickness)	Layer Depth (m)	Ground Water (g) or Surface Water Infiltration (s)	Surface Response to Collapse at Depth (marker)[1]
E1	0.70	1.8 - 4.6	s	40 mm (SM4)
	0.70	1.8 - 4.6	s	80 mm (SM4)
	0.30	4.6 - 7.5	s	
	0.25	7.5 - 10.5	s	
E2	0.30	2.5 - 5.5	s	70 mm (SM18)
	0.15	5.5 -9.0	s	
	0.20	11.8 - 14.6	s	
	0.95	14.6 - 17.7	s	
	0.20	17.7 - 20.6	s	
	0.55	23.8 - 26.3	s	
E3h	0.60	4.0 - 7.0	s	130mm (SM25) 180mm (SM27)
	0.40	7.0 - 10.0	s	
	0.25	10.0 - 13.0	s	
	0.40	13.0 - 16.0	s	
	0.55	16.0 - 18.5	s	
	0.60	18.5 - 22.0	s	
	0.25	22.0 - 24.5	s	
E4	0.30	16.0 - 18.5	s	30mm (SM40)
	1.00	8.0 -12.0	s	60mm (SM36)
	0.25	12.0 - 16.0	s	
	1.50	16.0 - 18.5	s	
	0.75	25.0 - 27.5	s	
Average (s.d.)	0.52 (0.34)	-	s	-

Table 3.12. Summary of collapse strains as measured at Lounge

[1] The nearest settlement markers to the extensometer locations are between 25 and 50 metres distant.

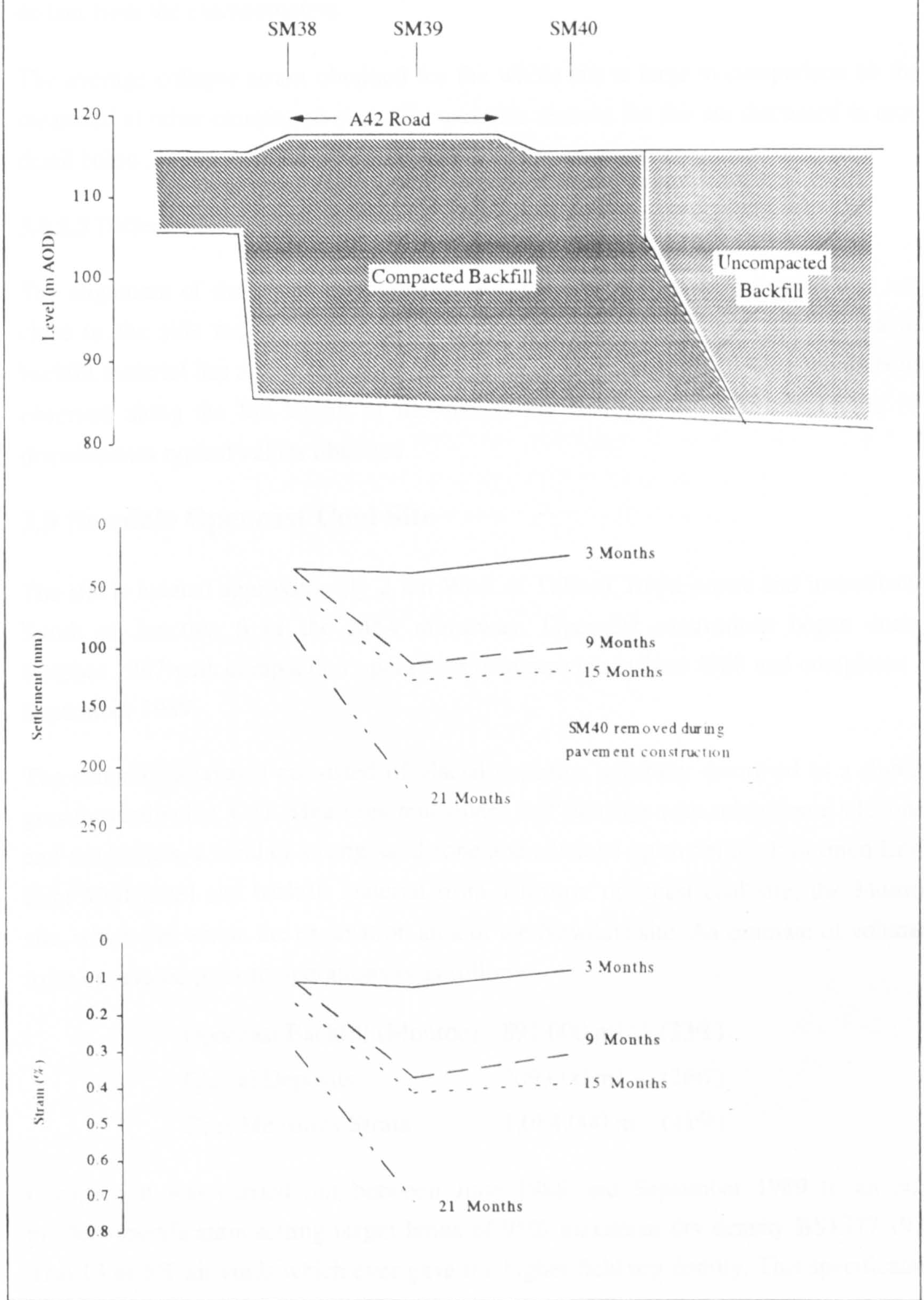


Figure 3.7. Lounge. Typical differential settlements across side walls

occurring laterally across the site as the settlement markers are some 25 to 50 metres distant from the extensometers.

The average collapse strain obtained for the whole site is large in comparison to that measured at other compacted sites. The possible reasons for this are discussed in more detail below.

3.8.2.5 Differential Settlements

The alignment of the proposed highway (A42) is such that it runs parallel and very close to the side wall of the excavation thus a large proportion of the compacted backfill material lies above this wall. This has resulted in differential settlements being observed along the full length of the site as the side wall is traversed. Figure 3.7 demonstrates typical values obtained.

3.9 Newdale Opencast Coal Site

The site is located approximately 2 km West of Telford Town centre and immediately South of Junction 6 of the M54 motorway. Opencast excavations began during October 1987 with compaction operations commencing in June 1988 and completed in September 1989.

The material excavated consisted of glacial deposits, generally described as a slightly gravely sandy clay, Coal Measures mudstones and fireclays with subordinate siltstones and sandstones, a band of strong sandstone and siltstone up to 5m thick (termed Little Flint Mudstone) and backfill material from a former opencast coal site, the Monroe site, which lies within the excavation area of the Newdale site. An estimate of volumes to be excavated prior to operations is as follows:

Opencast Backfill (Monroe)	891,000 m^3	(33%)
Glacial Deposits	709,000 m^3	(26%)
Coal Measures Strata	1,084,000 m^3	(41%)

Compaction was carried out between June 1988 and September 1989 to an end-product specification setting target limits of 95% maximum dry density BS1377:1975 Test 13 or 5% air voids which ever gave the higher field dry density. This specification was relaxed slightly for the Little Flint Mudstone to 93% maximum dry density.

During October 1988 backfill was placed and compacted in layers greater than specified and included large boulders. Three areas were identified, one of which was re-excavated and re-compacted but the other two remain leaving a suspect area as shown in Figure B8. Additional extensometers and surface settlement markers were installed to provide extra monitoring for this area.

Due to a slip in the west highwall and the associated buttressing a steep slope was buried without being reduced to 3m in height or excavated back to a 1 in 3 gradient, as required by the specification. Two lines of additional settlement markers were installed across this buried slope to monitor for differential settlements

3.9.1 Compaction Monitoring

Testing of the fill was carried out several times daily to check on the adequacy of the compaction. Testing consisted of in situ density and moisture content determination using both sand replacement test (BS1377:1975 Test 15) and a nuclear density gauge. In addition to these in-situ tests, on-going laboratory tests were carried out to determine maximum dry density and optimum moisture content values of the various materials and blends being incorporated within the compaction scheme.

In situ Testing			
Moisture Content (%)	Dry Density (Mg/m^3)	Air Voids (%)	Relative Compaction (%)
-	2.01 [1.90 - 2.11]	5.0 [0 - 12]	96.2 [91 - 101]

Table 3.13. Summary of the compaction monitoring data carried out at Newdale.

Note: The above is based on 115 daily average results

Average in-situ compaction results can be seen in Table 3.13 which shows that the average compaction for the whole site was carried out to the standard required of 95% of maximum dry density (BS1377:1975 Test 13). Any areas where compaction was deemed inadequate from the monitoring results, further work was carried out such that the requirements of the specification were met.

3.9.2 Instrumentation

Ground movements and groundwater levels were monitored by the installation of 51 surface settlement stations, 14 magnetic extensometers and 12 piezometers. The locations of which are shown in Figure B8. Installation occurred, on average, some 300 days after completion of compaction operations. Monitoring data made available to date generally covers a 10 month period for the surface settlement stations and 23 months for the extensometers.

3.9.2.1 Groundwater Recovery

Groundwater monitoring indicates that the groundwater within the backfill had reached, or was close to, an equilibrium level by July 1990 some 10 months after completion of compaction operations. This level is believed to be close to the pre-excavation level. Notable rises in water level were recorded by a number of piezometers across the site during the period between February and April 1990 amounting to between 5 and 10 metres. This followed a period of heavy rainfall between the months of December and February 1990.

3.9.2.2 Creep Strain

When discussing the settlement behaviour of the site it is convenient to split it into two distinct areas, the whole site excluding the area of suspect backfill and the area of suspect backfill; average alpha values obtained for these two areas are 0.07 (0.04 s.d.) and 0.24 (0.06 s.d.) respectively. Thus confirming a poorer level of compaction was achieved within the area of suspect backfill. The alpha value for this area still however compares with those typical for compacted backfill.

3.9.2.3 Collapse Strain

Periods of accelerated settlement can be seen at the majority of surface settlement stations and extensometer installations. Two distinct periods can be identified one coincident with a period of heavy rainfall and another coincident with the general water table rise which occurs across the site as discussed above. The close correlation between the timing of these 'events' and the inundation of the fill indicates them to be as a result of collapse. The magnitude of the collapse across the site is summarised in Table 3.14.

In examining the extensometer data it was noted that in some cases collapse strain recorded at depth was not seen transmitted to higher magnets. An arching effect prevented the migration of this movement up through the backfill implying that the collapse is occurring in isolated pockets rather than uniformly in a given horizon across the site. This is probably largely as a result of the variability of the %-age air voids contained within the backfill. The compaction monitoring data indicated air voids values to range from between 0 and 12%. The generally accepted air voids value below which it is considered collapse strains are virtually absent is 5%. Therefore due to the

Extensometer	Collapse Strain (% of layer thickness)	Layer Depth (m)	Ground Water (g) or Surface Water Infiltration (s)	Surface Response to Collapse at Depth (marker)
E1[1]	0.20	1.0 - 2.5	s	Insufficient Data
	0.10	15.0 - 18.0	g	Non
E2[1]	0.15	26.0 - 27.0	g	Insufficient Data
E3[1]	0.05	2.0 - 3.5	s	"
	0.20	2.0 - 3.5	s	"
E4[1]	0.25	14.0 - 16.5	g	"
	0.65	25.0 - 27.0	g	"
E5[1]	0.25	1.0 - 3.5	s	"
E6[1]	0.30	16.0 - 19.0	g	30mm (113)
	0.90	19.0 - 22.0	g	
E9[1]	0.22	12.0 - 14.0	g	Non
	0.25	17.0 - 19.0	g	
E10[1]	0.30	1.0 - 4.0	s	10mm (119)
E11[1]	0.08	7.0 - 10.0	g	10 mm (121)
	0.25	10.0 - 13.0	g	
E13[2]	0.50	1.0 - 3.0	s	Insufficient Data
	0.25	8.5 - 12.5	g	20mm (130)
E14[2]	0.40	14.0 - 17.0	g	60mm (131)
	0.17	20.0 - 23.0	g	
	0.22	23.0 - 26.0	g	
	0.52	26.0 - 27.0	g	
Averages (s.d.)	0.20[1] (0.09)	-	s	
	0.32[1] (0.26)	-	g	
	0.50[2] (-)	-	s	
	0.31[2] (0.15)	-	g	

Table 3.14. Summary of collapse strains as measured at Newdale

[1] Outside of suspect backfill area [2] Suspect backfill area

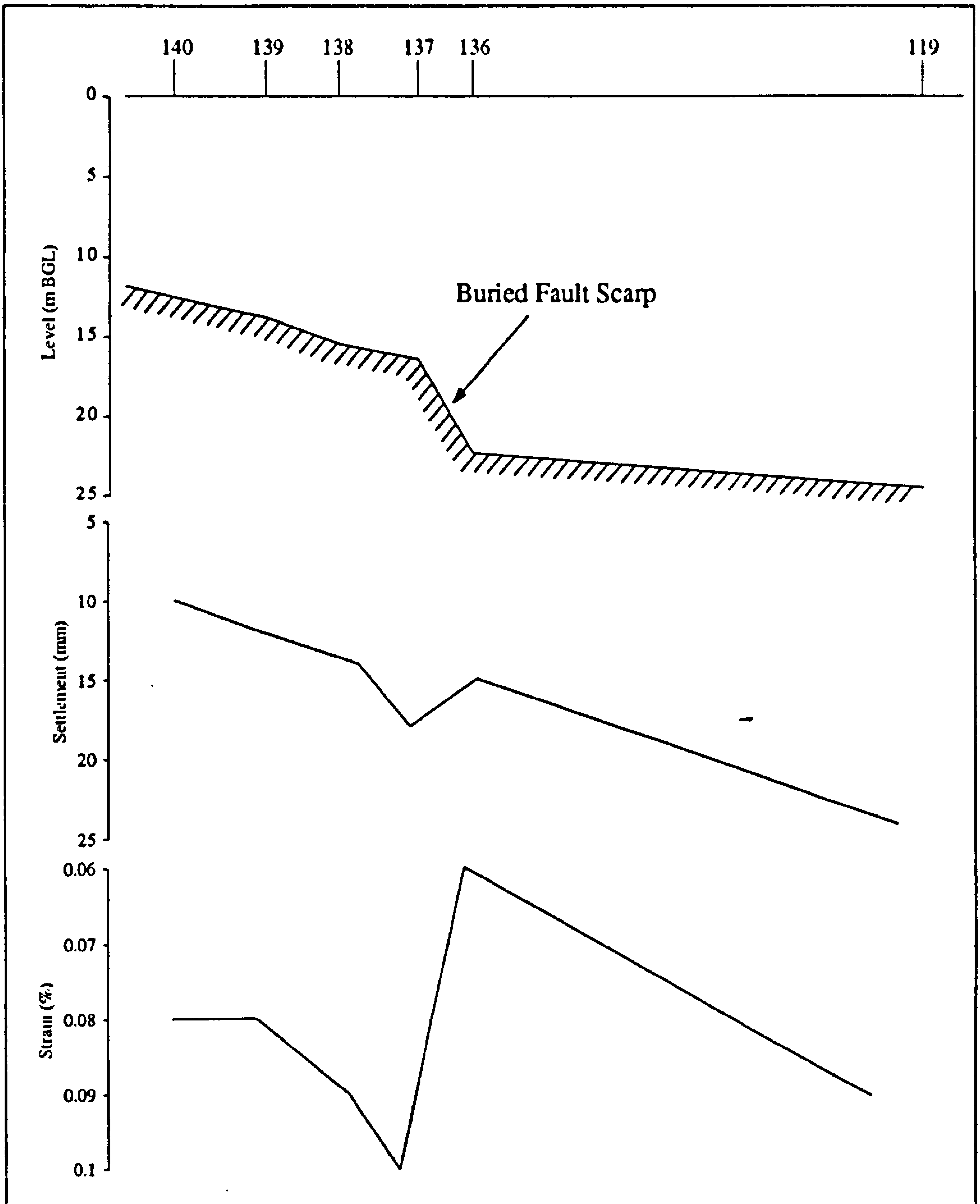


Figure 3.8. Newdale. Differential settlements across a buried fault scarp

variability of the air voids encountered the susceptibility of the backfill to collapse will also be variable thus it is likely to occur in isolated pockets.

3.9.2.5 Differential Settlements

Settlement markers were positioned such that differential settlements could not be measured across side walls. However a traverse of markers was positioned across a buried wall which was steeper than specified due to a slip as discussed above. The measured differential settlements that occurred over the monitoring period are summarised in Figure 3.8.

3.10 Patent Shaft Opencast Coal Site

The Patent Shaft Opencast Coal Site is located within the Metropolitan Borough of Sandwell and is situated some 1 km west of Wednesbury town centre. It covers an area of about 34 ha and was mined up to a depth of 40 metres. Opencast mining operations commenced in July 1988, compaction commenced in December 1988 and was completed by September 1990. Approximately 6 Mm^3 of overburden was excavated which predominantly consisted of made ground and Coal Measures strata. The Measures strata consisted mainly of mudstones ranging in strength from soft to hard.

The site is crossed by the Coseley-Wednesbury Fault with a throw of some 50m. This fault is oriented approximately east-west, resulting in some two thirds of the excavation area being to the south of it and one third to the north.

During excavation the existence of sub-artesian pressure at the base of the excavation resulted in water entering through old shafts and boreholes. Little water was observed to enter the excavation through high/side walls. Water entering the base of the excavation was dealt with by channelling it via a system of drainage trenches to two pumping sumps. These were subsequently brought up with the backfill, using precast concrete manhole rings, such that a control on the groundwater could be maintained.

Compaction was carried out in accordance with the Department of Transport Specification for Road and Bridge Works (1976). The method of compaction chosen was determined from compaction trials carried out on all materials encountered. Plant used during compaction trials consisted of a Bomag BW6 smooth vibrating roller, a CAT 825 tamping roller, a CAT 653 vibrating roller, an A.G.10 towed vibrating roller and a JV180 heavy vibrating roller.

Following completion of the backfilling to the south of the Cosely/Wednesbury Fault it became necessary to place overburden mounds on this area in order to store sandstone and concrete.

3.10.1 Compaction Monitoring

Field and laboratory tests were carried out almost every day on material compacted within the void. Field density tests took the form of both sand replacement and nuclear density tests, laboratory testing included particle size analysis, plasticity indices, compaction BS1377:1975 Test 13, California bearing ratio, specific gravity, and moisture content determination.

A summary of moisture content, dry density and air void results are presented in Table 3.15 for each material type encountered. Dry densities are generally within 95% of the maximum dry densities achieved during compaction trials for the relevant class of material. With the more variable tip mixture and made ground material it is difficult to determine a representative maximum dry density and as such a comparison of field densities with this value will not necessarily be indicative of the compactive state of the material. A better measure of the state of compaction achieved in this case is that of air voids. An average air voids value for all the materials compacted is approximately 9.1% which is consistent with that which would be expected from material compacted at appropriate moisture contents to a DTp method specification.

3.10.2 Instrumentation

Surface settlement stations, extensometers and piezometers were installed, on average, some 360 days after the completion of compaction operations the locations of which are shown in Figure B9. Monitoring data made available to date covers an average period of about 220 days.

3.10.2.1 Groundwater Recovery

Piezometers installed prior to excavation monitored an average piezometric head of 118.8 mAOD to the south of the Coseley-Wednesbury Fault and 120.6 mAOD to its north. Past records of the South Staffordshire Mines Drainage Commission indicate that this fault constitutes a partial barrier to groundwater flow in this area.

Monitoring following restoration indicate that a stable water level is attained some 5 months from the start of monitoring close to pre-excavation levels. Monitored rises were in the order of about 5 metres in 5 months with the maximum rise of 15 metres in less than 2 months being recorded at PX02.

3.10.2.2 Creep Strain

The ground movement measured at Patent Shaft is predominantly heave. Average heaves for the longest monitoring period of just under 2 years are in the order of 20mm which is not dissimilar to that measured over shorter time periods. Settlements are generally very small, in the region of 10mm, less than 0.1% strain, with the largest values being measured around the two pumping shafts. This is probably as a consequence of compaction difficulties up to and against the concrete rings used in the construction of the shafts.

Material	Site Class	No. of Results	Moisture Content (%)	Dry Density (Mg/m^3)	Air Voids (%)
Soft blue grey mudstone	1	62	9.0 [7.0-12.6]	1.96 [1.69-2.07]	8.1 [2.1-12.3]
Light grey mudstone/tip mixture	2(m)	67	11.3 [7.4-14.1]	1.87 [1.71-2.11]	8.9 [0-14.2]
Light grey mudstone	2	159	7.1 [4.9-9.5]	2.04 [1.79-2.21]	9.1 [3.1-13.7]
Tip mixtures (mudstone, with rubble, ash , clay)	4	244	11.9 [7.3-18.2]	1.80 [1.54-2.04]	8.9 [0-16.7]
Hard light grey mudstone mixture	2a(m)	15	7.2 [5.2-8.3]	2.03 [1.92-2.12]	10.3 [5.9-13.6]
Hard light grey mudstone	2a	23	7.3 [4.3-13.6]	1.95 [1.73-2.04]	13.2 [5.4-21.9]
Yellow brown clay	3	11	13.9 [10.3-21.0]	1.83 [1.66-1.96]	5.6 [0-9.3]
Rubble and clay matrix	5	28	11.1 [8.1-14.5]	1.88 [1.61-2.01]	8.2 [1.0-15.6]
Moorcroft Export	4	19	11.8 [7.3-17.1]	1.83 [1.65-1.98]	10.5 [5.6-14.8]
		Total	Average	Average	Average
		617	12.7	1.90	9.1

Table 3.15. Summary of the compaction monitoring data carried out at Patent Shaft.

The large amounts of heave measured are probably as a result of the overburden mounds, between 15 and 20 metres in height, placed south of the Coseley Wednesbury Fault. It can be seen that where settlements are occurring they are much reduced in this southern area when compared with average values north of the fault, thus indicating the beneficial effect, in terms of reduced settlement, the overburden mounds have had.

For a large proportion of the surface settlement stations it was not possible to determine a value for alpha. Where alpha values could be determined, predominantly north of the Coseley/Wednesbury Fault, an average value of 0.09 was obtained. These values have been calculated using a time for the 'mid point of construction' as either the start of monitoring if before September 1990 or September 1990, the date compaction operations were completed across the site. These values have been used because the true 'mid point of construction' times are unknown. This will result in an underestimate of the true alpha value, the scale of which will be proportional to the difference in the actual 'mid point to construction' time and the value used.

3.10.2.3 Collapse Strain

The influence of the ground water on surface movement at Patent Shaft appears to be minimal. There is no indication of any periods of accelerated settlement from the surface settlement stations and only at extensometer 2 is there any slight indication of collapse.

The reason for this apparent absence of collapse may be that any that did occur was missed by the monitoring scheme as the ground water table was virtually restored to its pre-excavation level at the start of settlement monitoring. The examination of extensometer 2 indicates a possible period of increased settlement during a 15 metre water level rise of about 0.06% strain.

3.10.2.4 Heave

Splitting the settlement data north and south of the fault it can be seen that average heaves south of the fault are much higher than those to the north. Strains are in the region of 0.25% which ties in well with strains measured by Knipe (1979) and Charles and Burford (1978) for the elastic expansion of backfill on the removal of overburden mounds. These strains are believed to decrease with time.

3.10.2.5 Differential Settlements

Lines of monitoring stations were installed across the subterranean batter associated with the Coseley/Wednesbury Fault and across high/side walls. The subterranean batter is some 30 metres high, the crest of which is only some 3 metres below restored ground level. Differential settlements measured across it average in the region of 1:1500 with a maximum of 1:400 being measured between P89 and P84. Measurements over the high/side walls, the heights of which vary from between 10 and

30 metres, average 1:1000 with a maximum of 1:300 being measured between P6 and P5. These values are in most cases a result of differences in heave.

3.10.2.5 Lateral Movements

Lateral movements measured at Patent Shaft average 6.6mm (0.024%) compression and 7.3mm (0.025%) expansion. The largest movements are generally measured in the areas of batter zones which is to be expected as lateral movements are considered to be closely related to large differential movements. The pattern of movement across the site is quite random but there is some indication that a greater proportion of the cases where expansive lateral movements have been measured this has been perpendicular to batters lying either directly or partially over them.

3.11 Pithouse West Opencast Coal Site

Pithouse West is located north if Beighton, South Yorkshire, and approximately 11km south east of Sheffield. The site comprised derelict land released after closure of the Brookhouse Pit and Coking works. Coaling was carried out over an area of 7 ha and the excavation was carried out to a depth of approximately 50m. Approximately 1.3 Mm^3 of backfill material, consisting largely of mudstones with some sandstones, was replaced by controlled compaction during March 1990 and January 1991. Some compaction was carried out prior to this but was not monitored for settlement. Within the compaction area the water table was recorded within 2m of site formation level prior to excavation.

A method compaction specification was adopted, confirmed by compaction trials. Compaction plant used consisted of a Cat 825 tamping roller and a single light vibratory roller. During trials the range of acceptable moisture contents was modified to optimum moisture content (omc) - 3% to omc + 3%. Maximum dry density and optimum moisture content were determined with representative material using BS1377:1975 Test 12.

3.11.1 Compaction Monitoring

Monitoring of the compaction operations was carried out by both field and laboratory testing. Field testing mainly consisted of in-situ density testing using the sand replacement method. A nuclear density gauge was used between March and October 1990 but only to demonstrate its versatility and not to assess the performance of the compaction.

The field density results from sand replacement testing are summarised in Table 3.16. This shows that acceptable levels of compaction were generally achieved with an average dry density of 2.08 Mg/m3 and an average relative density of 108%.

In situ Testing			BS1377:1975 Test 12	
Moisture Content (%)	Dry Density (Mg/m^3)	Relative Compaction (%)	Optimum Moisture Content	Maximum Dry Density (Mg/m^3)
8.6 [4.0-15.0] (220 results)	2.08 [1.60-2.40] (203 results)	108 [75-120]	11 (6 tests)	1.97 (6 tests)

Table 3.16. Summary of the compaction monitoring data carried out at Pithouse West.

3.11.2 Instrumentation

Settlement and groundwater monitoring was carried out by the installation of 4 extensometers and piezometers and 10 surface settlement markers, the locations of which are shown in Figure B10. Surface settlement marker monitoring commenced soon after completion of compaction operations and data available to date covers about 260 days. Monitoring at extensometer locations commenced some 3 months after completion of compaction operations with some 5 months of monitoring data available.

3.11.2.1 Groundwater Recovery

The results from groundwater monitoring show that the groundwater table was largely re-established to pre-mining levels prior to the start of monitoring. Monitored levels are constant with only very slight 'seasonal' variations (less than 1m). Approximately 80% of the fill is saturated by groundwater. The rate of rise was approximately 4 metres per month having occurred prior to monitoring.

3.11.2.2 Creep Strain

An average alpha value of 0.15 (0.05 s.d.) was determined from settlement monitoring data for all stations excluding marker 6. Marker 6 was installed in an area of uncompacted fill which the calculated alpha value of 0.73 reflects.

3.11.2.3 Collapse Strain

There is little evidence of any collapse strain from either surface infiltration or ground water recharge from the surface settlement markers. This is no doubt as a result of the considerable groundwater recharge that occurred prior to monitoring. There is however some evidence of collapse within a layer at extensometer E2 where a period of accelerated settlement occurred during June 1991 amounting to 0.25% layer thickness. The groundwater monitoring results indicate this layer to be saturated during and some months prior to this strain, which may imply that the strain is as a result of material softening occurring some time after saturation.

3.11.2.4 Heave

At extensometer E1 a degree of heaving has occurred within the upper layers which may be as a result of elastic rebound to the compactive effort of the compaction operation and surface water ingress. Values are however very small, being less than 6mm.

3.12 Summary

3.12.5 Groundwater Recovery

An equilibrium groundwater level appears to have been attained at the majority of the sites examined close to the pre-excavation level, generally, within a year of completion of backfilling operations. Table 3.17 indicates the rates of recovery which have been observed at the studied sites.

The rapid nature of the recoveries observed would suggest that highly permeable conditions exist within the backfills examined. This is further borne out by the rapid response to seasonal variations and to pumping operations recorded at Bilston and Dixon respectively.

At only two of the sites examined, Flagstaff and Lounge, did the measured post-excavation water levels fall considerably short of the pre-excavation levels. At Flagstaff, however, there is some doubt as to whether the pre-excavation measurements were indicative of the true water table as only small seepage's were noted during excavation which were more indicative of perched water. At Lounge, pre-excavation level measurements, at either end of the excavation, indicate that there is a depression in the post excavation water table in the centre of the excavation. It is considered that external influences, mainly past and present mining activities, are responsible.

Site	Recovery	Notable Monitored Water Table Rises	Average Draw Down (m)
Barnabas	Stable groundwater table measured from the commencement of monitoring, some 1 to 10 months from completion of compaction.	-	-
Bilston	Stable groundwater table measured, approximately 3 metres below pre-excavation level, from the commencement of monitoring, some 15 to 24 months from completion of compaction.	Seasonal variation amounts to approximately 6 metres over a 6 month period	20
Dixon	Fluctuating water table measured as a direct response to pumping operations.	5 m rise in 4½ mths (R2) 10 m rise in 6 mths (R2) 3 m rise in 2 mths (R3) 6 m rise in 7 mths (R3) (as a direct result of the cessation of pumping)	-
Ketley Brook	Relatively stable groundwater table measured from the commencement of monitoring, some 8 to 10 months from completion of compaction.	-	-
Flagstaff	Relatively stable groundwater table measured from the commencement of monitoring, soon after the completion of compaction. Measured level some 5 to 10 metres below the pre-excavation level.	-	10
Lounge	Stable groundwater table measured from the commencement of monitoring, soon after the completion of compaction. Measured levels below the pre-excavation levels.	-	-
Newdale	Stable groundwater table measured, roughly equal to pre-excavation level, some 7 to 13 months after the completion of compaction.	5 m rise in 3 mths (P1) 5 m rise in 4 mths (P4) 10 m rise in 2½ mths (P9) 10 metre rise in 20 mths (P2) 7 metre rise in 3½ mths (P6) 8 metre rise in 3½ mths (P11)	15
Patent Shaft	Gradually rising water table measured on completion of compaction operations. Measured levels 3 metres below pre-excavation levels in the North after 6 months of monitoring and between 4 and 8 metres below in the South.	12 m rise in 4½ mths (PP06) 5 m rise in 2 mths (PX01) 16 m rise in 2 mths (PX02) 7 metre rise in 6 mths (PP06) 6 metre rise in 6 mths (PX05)	20
Pithouse West	Stable groundwater table measured, approximately 5 metres below pre-excavation level, from the commencement of monitoring, some 3 months after the completion of compaction.	-	-

Table 3.17. Groundwater recovery rates at study sites

It is therefore considered that compacted opencast backfill has a permeability such that the rebound of the natural groundwater table will generally occur within 1 year from the completion of compaction operations and the cessation of any pumping operations effecting the local groundwater table. Such a rebound rate may be restricted though by the influence of adjacent mining activities, past and present, as shown at Lounge were it is believed that adjacent past deep mining activity acts as a drain to the area thus depressing the water table.

Perched water tables may develop within opencast backfills due to layers of less permeable material preventing percolation of surface water or water draining from natural perched water within the surrounding strata into the fill; at both Ketley Brook and Patent Shaft there is some evidence of this. At Ketley Brook piezometer tips located within the backfill measured higher water levels than those located within the base of the backfill. There is slight evidence that the greatest differences between water levels measured from piezometer tips within the fill and those within the base are observed at installations close to the periphery of the excavation. Thus suggesting that this difference is as a result of drainage from perched water tables in the natural strata into the backfill.

At Patent Shaft a difference in water level is measured between piezometer tips located at different depths within the backfill. Generally though, as monitoring continues, the difference between the two levels decreases, thus indicating slightly less permeable layers rather than impermeable layers within the backfill.

3.12.1 Creep Strain

Creep strain has been recorded at all the sites studied, the magnitude of which expressed in terms of alpha is summarised in Table 3.18. This shows that for an opencast backfill compacted in accordance to a specification based upon the Department of Transport Specification for Road and Bridge Works or Highway Engineering, subsequent creep will occur at a rate defined by an average alpha value of 0.20 with a typical variation of ± 40%.

Cases where alpha is at the upper end of the range or greater indicates compaction to lower standards. At Barnabas an alpha value of 0.32 was obtained with a large variation of ± 70%. The high alpha value can be attributed to the relatively low level of compaction achieved (only 93% of the maximum dry density as determined from BS1377 Test 12) whilst the large variability is probably due an area of poorly compacted backfill. At Bilston in the final cut area where the inclusion of unsuitable material produced a less homogenous backfill a variation in alpha of ± 80% was

Site		Dry Density (Mg/m^3)	Air Voids (%)	Relative Compaction (%)	Monitoring Delay (days)	Creep Strain (%)		Collapse Strain as measured at depth (%)			
						Average	S.D.	Surface Water Infiltration		Groundwater Inundation	
								Average	S.D.	Average	S.D.
Barnabas	Whole Site	1.82	-	93 (Test 12)	120	0.32	0.23	0.29	0.06	-	-
	Poorly Compacted Area	-	-	-	-	-	-	1.15	0.93	-	-
Bilston	Whole Site	1.88	9.92	95 (Test 13)	550	-	-	Non	-	Non	-
	Central Area	-	-	-	-	0.24	0.08	"	-	"	-
	Final Cut	-	-	-	-	0.24	0.20	"	-	"	-
Blindwells		1.57	27.5	-	700	1.00	0.56	0.15	-	0.10	-
Dixon	Area A	1.97	-	98 (Test 12)	50	0.28	0.08	-	-	-	-
	Area C	1.86	-	94 (Test 12)	40	0.17	0.08	-	-	0.36	0.15
Flagstaff	Whole Site	1.93	5.0	100 (Test 13)	180	-	-	-	-	-	-
	Compacted Zone	-	-	-	-	0.19	0.18	0.23	0.12	-	-
	1 metre Zone	-	-	-	-	0.22	0.11	0.57	0.40	0.52	0.24

Table 3.18. Study Site Summaries

Site		Dry Density (Mg/m^3)	Air Voids (%)	Relative Compaction (%)	Monitoring Delay (days)	Creep Strain (%)		Collapse Strain as measured at depth (%)			
						Average	S.D.	Surface Water Infiltration		Groundwater Inundation	
								Average	S.D.	Average	S.D.
Ketley	Whole Site	1.87	6.0	93 (Test 13)	90	-	-	-	-	-	-
Brook	Excluding Suspect Area	-	-	-	-	0.12	0.06	Minimal	-	Minimal	-
	Suspect Area	-	-	-	-	0.23	0.07	Minimal	-	Minimal	-
Lounge	Area A	2.03	8.4	110 (Test 13)	-	-	-	-	-	-	-
	Area B	2.04	6.3	110 (Test 13)	-	-	-	-	-	-	-
	Whole Site	-	-	-	180	0.30	0.14	0.52	0.34	-	-
Newdale	Whole Site	2.01	5.0	96 (Test 13)	300	-	-	-	-	-	-
	Excluding Suspect Area	-	-	-	-	0.07	0.04	0.20	0.09	0.32	0.26
	Suspect Area	-	-	-	-	0.24	0.06	0.50	-	0.31	0.15
Patent Shaft		1.90	9.1	-	360	0.09	-	Minimal	-	Minimal	-
Pithouse	Compacted	2.08	-	108 (Test 12)	70	0.15	0.05	-	-	0.25	-
West	Uncompacted	-	-	-	-	0.73	-	-	-	-	-

Table 3.18. Study Site Summaries (cont.)

observed. At Ketley Brook a change in working practices and inclusion of unsuitable material and at Newdale the placement of layers greater than specified considerably increased the resultant alpha value, compared to other areas at the sites, placing it at the upper end of the range.

At Lounge an alpha value of 0.30 was recorded which is relatively large considering the high levels of compaction achieved at this site. It is thought that this is as a result of the large amount of collapse settlement that occurred at Lounge which obscured the creep behaviour leading to an over estimate of alpha.

The study site data gives only limited information on the creep behaviour of uncompacted backfill. At Blindwells and Pithouse West where uncompacted material was encountered alpha values of 1.4 and 0.73 were recorded respectively. Information obtained from the literature indicates that alpha values for uncompacted backfill are in excess of 0.70 and it is considered that due to the heterogeneous nature of the backfill that the variation will be in the region of ± 70%.

3.12.1 Collapse Strain

Collapse strains at the study sites are summarised in Table 3.18 for both groundwater inundation and surface water infiltration. The average value of collapse as a result of groundwater inundation measured for opencast backfill compacted in accordance to a specification based upon the Department of Transport Specification for Road and Bridge Works or Highway Engineering was found to be 0.30% of the saturated layer with a variation of ± 40%. It is considered that collapse as a result of groundwater inundation will represent the maximum that can occur and that as a result of partial saturation due to capillary effects in advance of the measured groundwater level and that due to surface water infiltration will be some percentage of this maximum.

Collapse as a result of surface water infiltration is generally seen to be negligible at sites were monitoring commenced some time after the completion of compaction operations. This is to be expected as once collapse occurs due to a surface water infiltration it is unlikely to happen again within the same material given similar levels of saturation. Thus providing yearly rainfall levels remain relatively constant it can be considered that after 2 years from the completion of compaction operations collapse as a result of rainfall infiltration will become negligible.

A considerable increase in collapse occurs when opencast backfill is compacted to a lesser standard. At Barnabas and Newdale in regions of poorly compacted backfill average collapse of 1.15% and 0.50% were recorded respectively with a variation well in excess of ± 40%. At Newdale this increase in collapse strain is thought to be due to isolated pockets of poorly compacted backfill as collapse recorded at depth within material becoming saturated is not recorded in material above. This is thought to be due to the bridging of well compacted material over these collapsing pockets of poorly compacted material.

There is little information about the collapse of uncompacted backfill from the study sites with the only example being that from Flagstaff where collapse of 0.57% was recorded within the 1 metre zone with a variation of ± 70%. However there are a number of examples from the literature as discussed in Chapter 2 from which an average of 2.5% is obtained and as with creep settlement, due to the heterogeneous nature of the resultant backfill a considerable variation is to be expected.

In the case of Lounge where high standards of compaction were achieved, considerable collapse settlements where recorded due to surface water infiltration. This apparently anomalous result is thought to be due to the influence of the side walls of the excavation as a large proportion of the compacted backfill lies above/close to these walls. A build up of shear stress between the settling backfill and the static walls is believed to have prevented the fill from settling at a rate typical of compacted backfill. The effect of surface water infiltration on softening the backfill material will, to a certain degree, have released these shear stresses leading to the considerable accelerated settlements observed.

It is of interest to note that in some instances complete saturation of a given layer at depth within a backfill occurred with no resultant collapse. It is therefore considered that collapse can be eliminated given high enough levels of compaction.

3.12.2 Heave

The heave observed at the study sites can be identified as that being due to elastic rebound upon removal of a surcharge and that as a result of the combined effects of near surface water ingress and elastic rebound after completion of compaction operations. This latter heave is believed to be typical of all compacted sites and as shown at Dixon, where monitoring commenced soon after compaction operations finished, can be in the region of 0.2% strain. This type of heave is however short lived and has been missed by the majority of the study site settlement monitoring results as monitoring generally commenced in excess of 6 months after completion of compaction operations.

Heave as a result of surcharge removal has been observed at both Bilston and Patent Shaft. At Patent Shaft this is in the order of 0.25% however at Bilston the value is much lower but monitoring here commenced after a greater period from surcharge removal than at Patent Shaft. In both cases the overburden mounds were of a similar size being in the region of 15 to 20 metres high but information about the period the mounds were in place is unknown. Both sites monitored heave for a period in excess of 3 years after the removal of the overburden mounds with any settlements being negligible. The rate at which heaving movement occurs is seen to decrease with time.

3.12.3 Differential Settlement

Differential settlements have been predominantly recorded over buried internal walls and high/side walls. This is to be expected as increasing fill depths will lead to increasing absolute settlements as measured at the surface. The magnitude of differential settlements will be determined from the combined effects of creep and collapse settlements and any heave that may occur due to surcharging.

In the cases where settlements have been measured over internal or high/side walls it is of note that the settlement profile does not match that of the pit floor profile exactly. The settlement profile is displaced from the pit floor profile towards the centre of the pit. This is believed to be as a result of the build up of shear stresses between backfill material and adjacent walls restricting settlement. This restriction is passed between adjacent backfill particles due to the shear stresses, in-turn, being developed between individual particles as the wall is traversed down slope. It is such a mechanism that is thought to be responsible for the large collapse settlements noted at Lounge. Shear stresses developed due to the close proximity of side walls to the majority of the fill are to some degree released due to softening of particles on surface water ingress resulting in increased settlement.

Differential settlements will occur not only due to changes in fill depth but also as a result of uneven compaction and changes in fill material laterally within the backfill. Such variations are inevitable but the tighter the control on the compaction operation the smaller these variations will be. As discussed above, typical variation in alpha for a well controlled compacted site is ± 40%.

3.12.4 Lateral Movements

Lateral movements were only monitored at Patent Shaft of all the study sites. Lateral movements measured were minimal with an average 6.6mm (0.024%) compression and 7.3mm (0.025%) expansion recorded. As discussed in Chapter 2 lateral movement is generally considered to be only significant in uncompacted backfills where quite large movements have been measured. Lateral movements are associated with large differential settlements and are therefore generally restricted to areas of fill overlying buried/side walls. Generally expansive movement is observed above the top of the wall and compressive over its base.

3.13 Conclusion

The analysis of the data obtained from the study sites combined with information obtained from the literature gives us a better understanding of the behaviour of opencast backfills thus enabling more informed planning of post restoration structural development to be carried out. It can be considered that two approaches exist, engineer the backfill to accommodate the structural development or design the structural development such that it can accommodate the anticipated backfill movements. In practice it will tend to be a combination of these two approaches but as the intended structural development is generally known at the planning stage of the mining operation it is the backfill that is initially engineered to accommodate the development.

In the following chapter the different types of structural development that are likely to be constructed upon opencast backfill and the means with which opencast backfill can be engineered to accommodate these developments are discussed.

BACKFILL PLACEMENT, IMPROVEMENT METHODS AND FORMS OF DEVELOPMENT

4.1 Introduction

In this chapter methods of backfill placement and ground improvement techniques will be discussed together with the influence they have on the behaviour of the resultant backfill. The behaviour of interest to this work is that of settlement. In the next chapter a method will be proposed for predicting backfill settlement which fundamentally relies upon a value for the creep compression rate parameter (alpha) and the magnitude of collapse that will occur upon saturation expressed as a percentage of the saturated layer thickness, the collapse strain parameter. It is therefore in terms of these two parameters that backfill settlement behaviour will be defined. Controlled and uncontrolled backfill placement will be examined together with the ground improvement techniques of surcharging, inundation and dynamic compaction. How these methods influence the backfill in terms of the creep compression rate and collapse strain parameter is examined. Finally, forms of development typical to opencast backfill restoration projects will be examined, to give an indication of acceptable settlement characteristics that the backfilled land must comply with for development to proceed.

4.2 Backfill Placement Methods

4.2.1 Controlled Backfill Placement

Opencast coal mining operations, where the backfilled land is to be developed upon completion of mining, typically incorporate some form of control during backfill placement. This is generally in the form of controlled compaction. The basis of controlled compaction is the tracking back and forth of compaction plant over each layer of fill as it is placed. To ensure the desired level of compaction is achieved the operation must comply with some form of specification. The specification dictates, amongst other things, the type of material which is acceptable for incorporation in the works and the degree of compaction which the material is to receive.

In recent years highway earthworks have been carried out either in accordance with The Department of Transport Specification for Road and Bridgeworks 1976 ('Blue Book') or the DTp Specification for Highway Works 1986 ('Brown Book'). In the 'construction' of opencast backfill, similar post-reclamation fill properties are required to those of soil/rock embankments, hence the specifications adopted for controlled opencast backfill works are generally based upon the above DTp Specifications. Difficulties can arise however when applying these specifications to opencast backfilling operations. These difficulties have been largely addressed in the report, SARCOB (1993) which gives the details of 3 standard specifications suitable for the controlled placement of opencast backfill. It is these 3 methods that will be examined together with how each method influences the settlement behaviour of the resultant backfill.

4.2.1.1 Performance Specification

This specification imposes the greatest control over the backfilling operation of any of the three methods. It can be summarised in terms of specifications for material classification and acceptability, compaction requirements and form of excavation slopes.

Material classification and acceptability

Backfill material is initially classified as either acceptable or unacceptable. Unacceptable material consists of organic material, material in a frozen condition, material susceptible to spontaneous combustion, material having hazardous chemical or physical properties and material not complying with the permitted constituents and material properties of acceptable material. Acceptable materials are further categorised according to a range of material properties including moisture content, grading, 10% fines and CBR values into the Classes 1A - well graded granular, 1B - uniformly graded granular, 1C - coarse granular, 2A - wet cohesive, 2B - dry cohesive and 2C - stony cohesive (acceptable material limits are also given for a further Class, 2D - silty cohesive, however such material is unlikely to be encountered in opencast reinstatement projects).

The importance of moisture content for categorisation can be demonstrated by the typical dry density/moisture content relationship for compacted material as shown in Figure 4.1. Thus demonstrating the necessity to compact material close to its optimum moisture content (omc) to achieve adequate levels of compaction. Acceptable moisture contents are therefore quoted in terms of a range either side of the omc, typically ± 2%. In the case of Class 2A however moisture content limits are set in respect of the plastic limit (PL) of the material. This is a better criterion than omc for 2A material, as the moisture content for this type of material will be in excess of the omc and a criteria with respect to the PL ensures adequate trafficability and shear strength. 2A material will typically consist of glacial drift in opencast reinstatement projects and as such will represent only a small proportion of the total backfill.

Class 1C material has a broad grading envelope and can contain particles up to 500mm in size, thus upon placement and compaction, this material will have relatively large voids between constituent particles. To ensure both strains on inundation and creep settlements are minimised the material needs to be durable in the long term. This is ensured by specifying a minimum 10% fines value for Class 1C material (modified so that testing is carried out on samples that have been soaked for a period of 5 days). Material falling into this class, with opencast reinstatement projects, will typically be fresh and slightly weathered sandstones and some siltstones.

Classes 2A and 2C have a minimum CBR value stipulated in addition to the moisture content requirements to ensure these materials have adequate strength.

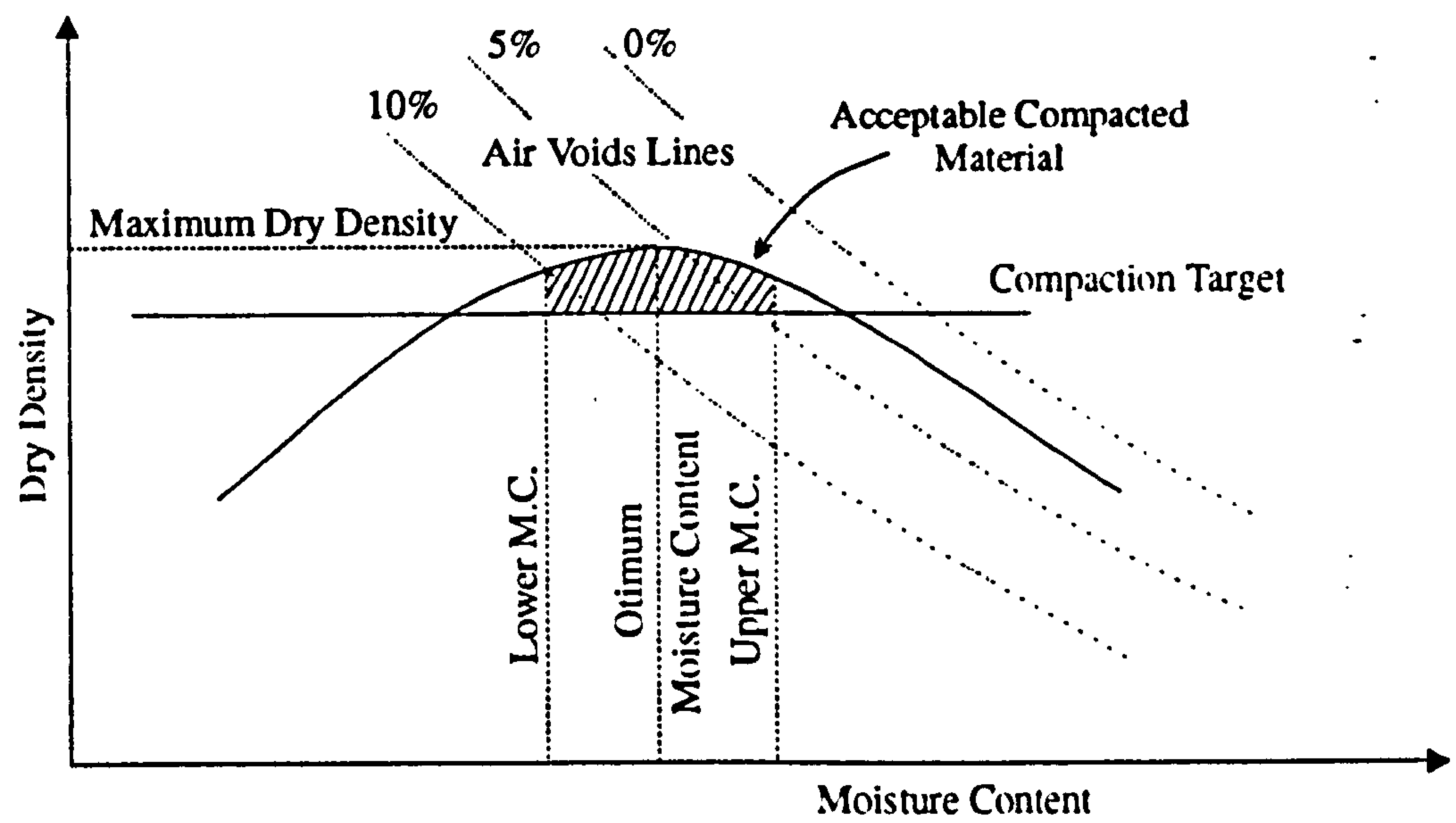


Figure 4.1. Typical dry density moisture content relationship.

The majority of the testing methods for determining material properties can be found within BS1377:1990. Modifications necessary for the application of these tests to materials encountered and compaction procedures adopted at opencast reinstatement projects can be found within SARCOB (1993).

Compaction Requirements

The compaction requirements for the performance specification are that fill Classes 1A, 1B, 2B and 2C are to be compacted to an end product, whilst Classes 1C and 2A require method compaction.

End product compaction specifies a criteria that has to met by the compaction operation thus leaving the method of compaction open to the Contractor. In this case the criteria to be met by all material compacted to end product compaction is 95% of the maximum dry density measured by the 4.5kg rammer compaction test (as modified for opencast applicability) for that Class or sub-Class of material. Compliance with the specification is assessed by regular in-situ testing. The maximum compacted layer thickness for materials requiring end product compaction is 300mm, a value principally stipulated because the state of compaction of layers greater than this thickness cannot be measured accurately by a standard nuclear density gauge.

Method compaction relies on a specific method of compaction being carried out on each of the different Classes of material that will be encountered during backfilling. Methods specify the compaction plant, the number of passes and the layer thickness and can be found for a range of compaction plant and all the Classes mentioned above within SARCOB (1993). These methods are based upon the DTp Specification for Highway Works (1986) with modifications and additions appropriate to the compaction of opencast backfill materials.

It is considered that greater control and more compact backfill is achieved with end product compaction as opposed to method compaction. However in the case of Classes 1C and 2A, end product compaction is not considered necessary. In the case of Class 1C, the long term strength and durability of the constituent particles will ensure that subsequent strains within the compacted layer are small provided the required method is complied with and as such this material does not require the greater control of end product compaction. Class 2A material is relatively 'wet' having a moisture content in the range of PL to PL - 4%. Consequently, a good state of compaction in terms of air voids is easily and consistently achievable by method compaction. Class 2A materials often comprise made ground which has highly variable compaction characteristics and for this reason compliance with performance criteria is difficult to monitor.

Form of excavation slopes

This is concerned with the gradients of the excavation slopes which are important in respect of the differential movements of subsequent developments over these slopes. Requirements are given for slopes within both the superficial deposits and below rockhead, maximum slope gradients are given together with benching requirements.

A backfilling operation carried out with the control imposed by the performance specification will produce a backfill that is both dense and relatively homogenous in terms of its settlement characteristics. In the previous chapter the settlement behaviour of a range of opencast backfill have been examined, none of which have been placed to the performance specification. However, in a number of cases the method of compaction carried out is comparable and as such gives an indication of the likely alpha and collapse strain values that can be attributed to backfill compacted to a performance specification; these are as summarised in Table 4.1.

Placement Method		Alpha (%)	Collapse (%)
Controlled	Performance	0.15 ± 0.05	0.25 ± 0.07
	Method	0.25 ± 0.10	0.40 ± 0.15
	Thick Layer	0.50 ± 0.30	0.90 ± 0.55
Uncontrolled		0.80 ± 0.55	1.20 ± 0.80

Table 4.1. Values considered appropriate for alpha and collapse for backfill placed in both a controlled and uncontrolled manner.

4.2.1.2 Method Specification

A compaction operation carried out in accordance to the method specification is similar to that of the performance specification. Material classification and slope requirements are the same, the difference is in the compaction requirements. All Classes of material are compacted in accordance to standard methods as defined within SARCOB (1993) or from compaction trials if a method other than one of the standard methods is chosen. Where non standard methods are chosen they must produce compaction comparable to the standard methods for all Classes of material that are to be encountered during the backfilling operation.

Backfill material placed in accordance to a method specification will be comparable to that placed in accordance to a performance specification. However, with the method specification there is inevitably less control and it is estimated that the average compaction achieved from method compaction will be in the region of 90% maximum dry density as opposed to the target density of 95% with performance placement (SARCOB 1993). This will lead to slightly larger post-reclamation settlements. In the previous chapter a number of the sites examined carried out backfill compaction to a specification very similar to the method specification discussed here. Thus enabling the typical values for alpha and collapse strain, in Table 4.1 to be attributed to backfill compacted to a method specification.

4.2.1.3 Thick Layer Method Specification

In this case the method specification, as discussed above, is carried out on material within the upper 20 m of the site, material below this is placed in 1 m layers without specific compaction. The 20 m of compacted fill may be reduced where considered appropriate, to a thickness ideally of not less than 15 m.

Material within the compacted zone will have alpha and collapse strain values as above, material within the 1m layers will have considerably greater values as no specific compaction is applied. However the trafficking by construction plant over the 1m layers will impart a degree of compaction. Values considered typical for alpha and collapse strain for such material are given in Table 4.1. There are however few cases from the study sites comparable to backfill placed in 1 m layers without specific compaction; therefore the values within Table 4.1 are values considered appropriate by the author and as such may require modification in the light of further monitoring data specific to material placed in this manner.

4.2.2 Uncontrolled Backfill Placement

Where no control is imposed on the backfilling operation, overburden is simply placed into the mined void by the machinery that was used for its excavation, no classification of material is carried out, all material being considered acceptable and slope gradients are defined in terms of stability only. Such a backfill will be highly variable and loose and undergo large settlements due to both creep and collapse. Of the study sites examined in the previous chapter, only Blindwells gives an example of settlement over backfill placed with no control. However together with examples in the literature as discussed in Chapter 2, estimates of typical alpha and collapse strain values can be made, as summarised in Table 4.1.

4.3 Backfill Improvement Techniques

Backfill improvement techniques that will be examined here include surcharging, dynamic compaction and inundation. It is considered that the use of any of these techniques will in most cases be considered unnecessary where controlled compaction has been carried out to either a performance or method specification as this will generally negate the need for any further improvement. The techniques will therefore generally only be applicable to backfills placed by uncontrolled means or with limited control such as 1 metre layer placement with no compaction.

4.3.1 Surcharging

The effectiveness of a surcharge provided by spoil mounds, in reducing post-reclamation settlements has been shown at both the restored opencast ironstone workings at Snatchill, Corby and the opencast coal workings at Horsley. A description of the mining operation and method of backfill placement at both of these sites are given in Appendix A. In both cases the backfill is considered to be a non-engineered fill as no control was carried out during placement, the backfill was simply placed by the machinery used during excavation.

Ground improvement technique	Settlement during construction				Total settlement to end of 1990			
	Mean	Max.	Min.	Max. Diff'l	Mean	Max.	Min.	Max. Diff'l
Preloading with 9m surcharge	1.4	3.0	-0.4	2.3	9.7	22.3	4.5	12.6
Dynamic consolidation	7.0	9.2	3.2	5.9	45.0	58.5	22.2	18.1
Inundation	6.1	14.3	2.8	6.8	48.0	143.0	11.7	89.5
Untreated	2.7	6.8	1.4	2.8	25.1	39.5	11.7	27.8

Table 4.2. Settlement (mm) of experimental houses at Snatchill, Corby (Burford and Charles 1990)

At Corby a 50m square was preloaded with a 9m high surcharge of fill which was placed over a 3 week period and left in position for one month before being removed. Settlements of up to 550mm were induced at ground level and 40mm at 10m depth. Table 4.2 shows the settlements after its removal as compared to the other ground improvement techniques carried out at the site. Settlement has been smallest in the preloaded area and the performance of the houses built here has been very satisfactory (Burford and Charles 1990). During the opencast extraction of the iron ore the groundwater table was kept below the base of the excavation by the installation of a drainage system which was left in place during backfilling. This has resulted in the water table remaining at about rockhead level and as such the monitored settlements have not been effected by collapse strains as a result of inundation due to a rising ground water table.

At Horsley a 30m high overburden heap was placed upon backfilled spoil during mining operations. A comparison made between settlements measured within fill beneath the overburden heap and fill elsewhere are summarised in Table 4.3. Settlements are given for three distinct periods which are primarily delineated by the period April 1974 to April 1981 during which a considerable rise in the groundwater table occurred saturating some 35m thickness of backfill. This water table rise occurred during the first three years of this period but it is considered that it affected the monitored settlements until April 1981 (Charles *et al* 1993).

Backfill (Gauge)	Monitoring Period		
	Dec.73 - Apr.74	Apr.74 - Apr.81	Apr.81 - Dec.92
Preloaded with 30m high surcharge (D1)	10 (heave)	104	1
Not surcharged (B2)	2	464	33

Table 4.3. Surface settlement (mm) as measured at Horsley between 1973 and 1993. (after Charles *et al* 1993)

From the analysis of the study site data only two examples of surcharging are found, at Bilston and Patent Shaft. At both these sites a scheme of controlled compaction was carried out based upon the Department of Transport Specification for Road and Bridge Works (1976) method specification. Surcharging was placed out of necessity to stockpile backfill during the backfilling operation and not specifically as a means of improving the backfill's settlement characteristics. Backfill improvement did however occur as discussed in the previous chapter which may indicate a benefit from surcharging backfills even when compacted to a method specification. However a relaxation of the method specification did occur at Bilston which would emphasise the benefit of surcharging and at Patent Shaft the monitoring data available to data indicates only relatively minor improvement due to surcharging.

The effectiveness of a surcharge can be examined by comparison of the vertical effective stress on the fill before and following placement of the surcharge. The increase in the effective vertical stress can be expressed in terms of the ratio of effective vertical stress following surcharging to that prior to surcharging. Figure 4.2 shows the increase in effective stress at the Corby site and settlement obtained. The data suggests that the surcharge was only effective down to a depth of 10m where the effective vertical stress ratio equalled 1.7. In the case of Horsley virtually all the settlement following removal of the spoil mound was within fill at depths greater than 34 metres below ground level. Thus indicating the depth to which the surcharging was effective was a little greater than the height of the surcharge and corresponded to an effective vertical stress ratio of 1.9.

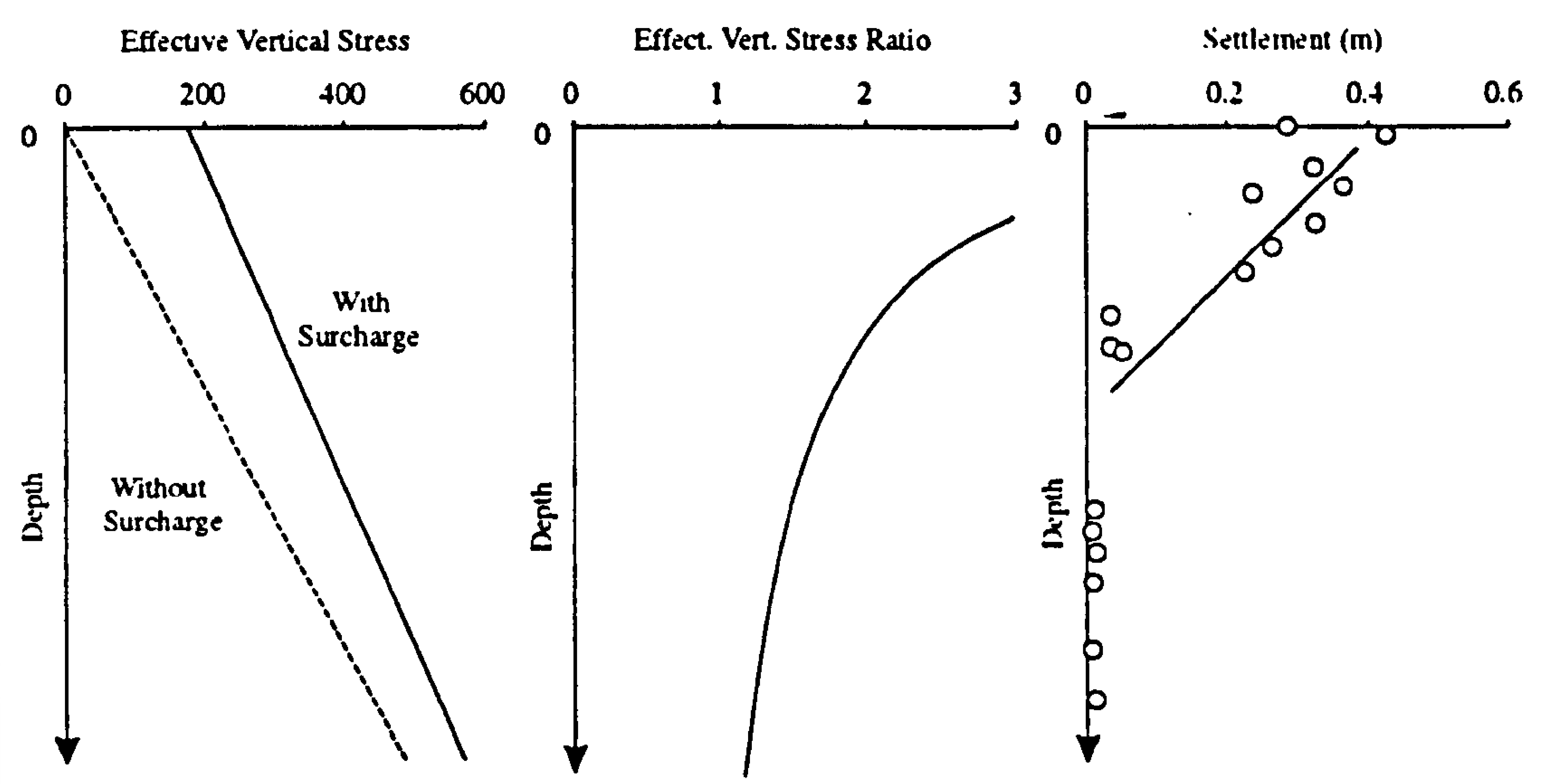


Figure 4.2. Variation of stress and settlement with depth in opencast backfill at Corby (Charles *et al* 1986b)

It is therefore proposed that to determine the influence surcharging has on settlement behaviour, backfill at a depth such that the effective vertical stress ratio is less than 1.8 will be unaffected and backfill at a depth such that the effective vertical stress ratio is greater than 1.8 will be improved. The improvement will be inversely proportional to depth below the base of the surcharge. This improvement can be expressed in terms of the alpha and collapse strain values which will both decrease as the settlement characteristics of the backfill improve. The relationship in Figure 4.3 is proposed for

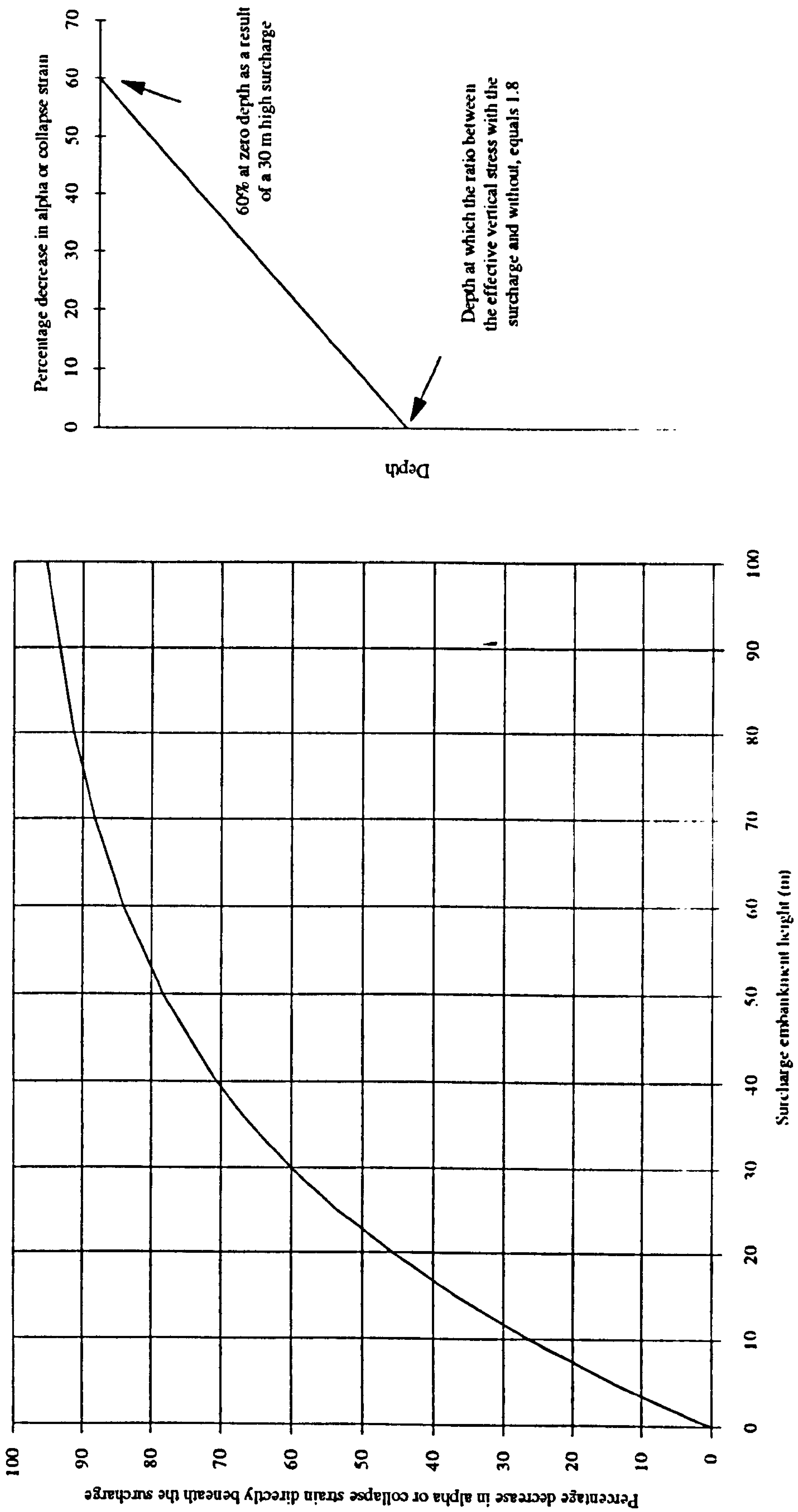


Figure 4.3. Relationship between the percentage decrease in alpha and collapse with the surcharge embankment height and the depth beneath the base of the surcharge

determining the decrease in both alpha and collapse dependant upon the height of the surcharge embankment. This is defined as follows:

$$P = \left[1 - \left(\frac{1}{C^H}\right)\right].100 \tag{4.1}$$

where P is the percentage decrease in both the alpha and collapse strain parameters. H is the surcharge height and C is the constant defining the relationship, known here as the surcharge constant. The value of P obtained from this relationship will apply to backfill material immediately beneath the surcharge. Its value will decrease linearly with depth until its value is zero at the point at which the effective vertical stress ratio is 1.8 (Figure 4.3). The effect this relationship has on the settlement prediction of surcharged backfill is examined in the Chapter 6 in terms of the value of C.

It must be noted though that this is a purely hypothetical relationship and factors not considered here will be of influence such as the ability of the backfill to compress under load. Backfill comprising largely of sandstone or fragments of fresh strong dry mudstone will show little improvement as a result of surcharging. The proposed relationship will therefore require modification dependant upon factors such as the material properties of the surcharged backfill. Further monitoring is therefore necessary to identify the factors that influence a backfill's suitability to surcharging and how these affect the proposed relationship.

4.3.2 Dynamic Compaction

Developed in the 1970s, this technique for the bulk in-situ stabilisation of loose materials involves the use of high energy impacts to produce immediate settlement, resulting in a reduction in voids and hence compressibility. To assess the likely depth of effectiveness of dynamic consolidation, Menard and Broise (1975) suggested the relationship:

$$WH > Z^2 \tag{4.2}$$

where W is the weight of the pounder in tonnes, H the height of fall in metres and Z is the thickness of the compacted layer in metres. Where the technique has been employed on fill materials, and other areas of marginal land, the depth of the treated material has usually been restricted to 10m (Condon 1986). Treatment usually involves dropping the pounder on primary and secondary grids, followed by tamping passes in between initial points. Its suitability for the stabilisation of opencast mine backfill was also examined at the Snatchill experimental housing site and settlement results are shown in Table 4.2. A 15 tonne weight of area $4m^2$ was dropped from a height of up to 20m on a 5m grid. Menard Pressuremeter tests carried out before and after treatment showed an improvement in pressuremeter modulus and limit pressures to a depth of 5 to 6 m. This agrees well with the measured settlements which also suggest that the treatment process had little effect below about 6m depth. At this site the groundwater level was below the base of the fill and hence the depth of treatment was not influenced by the presence of groundwater. No guidelines have apparently been published on the use of dynamic compaction on opencast backfills.

To assess the suitability of dynamic compaction as a backfill improvement technique, a means of calculating the improved settlement characteristics of the backfill is required. This could be achieved by a similar method to that proposed for surcharging whereby an adjustment is made to the alpha and collapse strain values related to both the scheme chosen and the fill depth below ground level down to the depth Z (see above equation). However, the limited data available concerning the dynamic compaction of opencast backfill make it difficult to propose such a method.

4.3.3 Inundation

A further potential backfill improvement technique is the inundation of newly placed fill by addition of water from the surface. Addition of water accelerates ground water recharge which gives rise to collapse settlements. Water is allowed to percolate into the backfill thus saturating the ground beneath. An example of its potential effectiveness can be seen at the Horsley site, the details of which can be found in Appendix A, where collapse strains due to a rise in the groundwater table were much reduced in an area which had been previously overlain by a lagoon (Charles *et al* 1984a).

An experiment to induce settlement by artificial inundation was conducted at the Snatchill experimental housing site, Corby, the results are summarised in Table 4.2 above. Water was allowed to percolate into the backfill from water filled trenches thus saturating the ground beneath. Settlements were induced rapidly on the filling of trenches and showed a degree of proportionality with the volume of water draining into the fill. The settlement caused by inundation seemed to be confined to the upper 5m of the fill.

The subsequent settlements of experimental houses built on the treated ground were seen to be the least successful of the three ground improvement techniques carried out at the site. The ineffectiveness of inundation as a method of ground treatment in this case was probably due to the difficulty of saturating the fill by the addition of water from the ground surface via trenches. The water tending to run away down the largest voids and fissures thus failing to provide a uniform treatment over the area. More settlements may have been produced if the trenches had been closer spaced and deeper (Charles *et al* 1978).

A more effective method of artificial inundation may be through the ponding of water on the backfill layers as restoration proceeds or by injecting water into the backfill via closely spaced boreholes. Such methods are examined in more detail within SARCOB (1993).

To assess the suitability of inundation as a backfill improvement technique, settlement predictions for inundated backfill could be made using a modified value for the collapse strain parameter. It must be considered though that such a technique would also improve the creep characteristics of the backfill and as such a modification to alpha would also be required. As in the case of dynamic compaction, without further case studies, it is difficult to determine how both the collapse strain parameter and alpha will be modified upon a scheme of artificial inundation.

4.4 Forms of Development

The specification chosen for the restoration of a given site should be tailored to the type and timing of the proposed development where known. Restoration of opencast sites by means of a controlled compaction operation will generally render the site suitable for development with lightly loaded structures on shallow foundations or roads and associated drainage as described below (taken from SARCOB 1993).

4.4.1 Building Structures

These may typically comprise of the following:

4.4.1.1 Light frame structures for industrial retail or warehouse use

These typically comprise a steel framework with cladding which may include a considerable amount of glazing. The lower portion of the sides of the structure are typically formed by brick walls. The structural load of frame and cladding is carried via steel columns to pad foundations which may typically be 2 to 3 m in size. Typically, on fills the low brickwork walls are supported by reinforced concrete strip foundations and the reinforcement is continued through the pad foundations so as to form a continuous ring beam. The floor of the unit will generally comprise reinforced concrete ground bearing slabs of a thickness appropriate to the proposed loadings. Alternatively, where higher post-construction differential movements are expected, the brickwork walls are supported on thickened edge beams which are integral with a thicker floor slab.

4.4.1.2 Brickwork and blockwork structures

Brickwork or blockwork structures may be, for example, offices associated with industrial units or housing. When constructed with shallow foundations on fill they should generally be of a simple rectangular shape in plan and limited to two storeys. Where post-construction settlements are likely to be small, foundations may be reinforced strip footings or a semi-raft foundation incorporating a lightly reinforced concrete ground floor slab with a thickened edge beam. In situations where expected movements preclude the use of this type of foundation fully reinforced raft foundations, possibly incorporating thickened edge beams must be adopted. In any case, such structures should ideally be single units of maximum plan dimension of the order of 15 m. Larger structures should be constructed by adoption of articulation joints between elements of the structure founded on separate raft foundations. Offices integral with warehouse units should have minimal connection with the steel framed structure so as to permit differential movement.

Services entering buildings will require flexible connections and drainage should be designed with appropriate falls to counter the effect of both total and differential movements.

4.4.1.3 Settlement Considerations

Differential settlements and distortions of structures should be considered in accordance with Figure 4.4. Table 4.4 summarises typical limiting values of differential settlement in terms of either relative rotation or deflection ratio. Limiting values are usually considered in terms of the visual appearance and serviceability of the structure rather than structural damage. Cracking of load-bearing brickwork, or walls forming part of a framed building, will be detrimental to their appearance and may also affect the weather-tightness and heat and sound insulation properties.

When considering differential movements of framed buildings and reinforced brickwork, relative rotation is the appropriate criterion. Table 4.4 indicates typically assumed limiting values of relative rotation for structural damage and cracking of walls.

Differential settlements in reinforced brickwork are considered in terms of the deflection ratio. Figure 4.4 shows that the deflection ratio is the differential settlements occurring over half of the length of the structure elevation. Although allowable movements in reinforced brickwork are greatly dependant on the layout of doors and windows and arrangement of brickwork panels, Table 4.4 nevertheless provides a useful guide to typical values.

Type of Structure	Type of Damage	Typical limiting values
		Values of relative rotation (angular distortion)
Framed buildings and reinforced load bearing walls	Structural Damage	1/150 - 1/250
	Cracking in walls and partitions	1/500 (1/1000 - 1/400 for end bays)
		Values of deflection ratio
Unreinforced load bearing walls	Cracking by sagging	At L/H = 1 1/2500
		At L/H = 5 1/1250
	Cracking by hogging	At L/H = 1 1/5000
		At L/H = 5 1/2500

Table 4.4. Limiting values of distortion and deflection of structures (after Tomlinson 1986) (Note: Refer to Figure 4.4 for definition of L and H.

Overall tilt, or rotation, of the structure is also a consideration and such movements mat occur over buried slopes. A tilt of greater than 1/250 is likely to be noticeable and a tilt of 1/100 will be clearly visible (Tomlinson 1986). A deflection ratio of greater than 1/250 will also be clearly visible. Allowable tilts for floors supporting sensitive machinery or high-bay stacking systems are likely to be smaller. Large structures should not be sites over buried slopes because of the risk of structural disruption due to differential movements in addition to overall tilt.

4.4.2 Roads and Associated Drainage

The design of major highways will be carried out in accordance with appropriate Department of Transport standards which stipulate requirements for changes of curvature both in vertical and horizontal alignment. These relate mainly to ensuring road safety in terms of both safe road gradients and sighting distances.

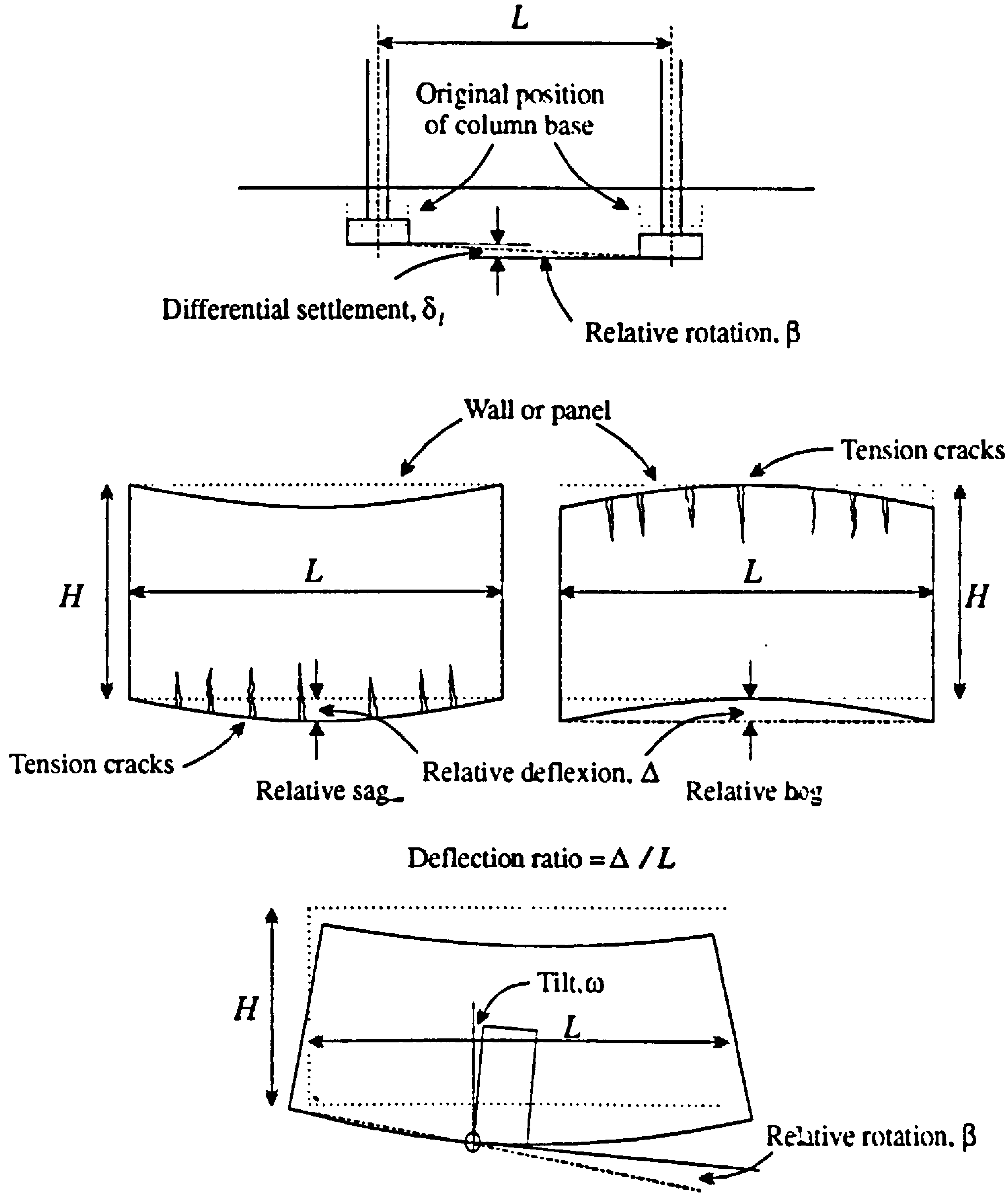


Figure 4.4. Definitions of differential settlement and distortion for framed and load-bearing wall structures (after Tomlinson 1986)

Road drainage will comprise highway drains collecting surface run-off and formation drains carrying groundwater from the pavement layers and sub-grade materials. Minimum acceptable gradients for highway drains, to ensure that they are self-cleaning are likely to be of the order of 1/150 to 1/225 depending on the pipe diameter. For the larger diameters an absolute minimum of 1/300 may be acceptable (SARCOB 1993). The gradient of formation drainage, comprising either filter or fin drains, will be governed by the geometry of the road. Slightly lesser minimum gradients than those quoted above may be acceptable. The normal cross-fall on the surface of major carriageways is 2.5%.

In all cases, as-constructed gradients and cross-falls must include appropriate margins for possible reduction due to differential movements and this is particularly critical along sections of reverse horizontal curvature where surface water ponding may arise. Differential movements are likely to be most pronounced over buried slopes and initial gradients must be adequate to ensure that minimum falls are maintained following settlement.

4.5 Conclusion

This chapter has outlined some of the more common methods of backfill placement and techniques used to improve the settlement characteristics of backfills placed with either limited or no control. How these methods and techniques influence the settlement characteristics of the resultant backfill have been assessed in terms of the creep compression rate parameter (alpha) and the collapse strain parameter. In the following chapter it can be seen how these parameters will be used in the prediction of backfill settlement.

Typical forms of development have been briefly discussed to give an indication of the range of settlements that can be accommodated by such development. For any given development scheme it is these settlement characteristics that the opencast backfill must comply with for development to proceed. It is therefore important when tailoring the chosen development to the chosen method of backfill placement and/or improvement technique that a means of predicting the settlement characteristics of the resultant backfill is available. In the following chapter a method proposed by the author for predicting backfill settlement is given together with the development a computer program to facilitate its use.

SETTLEMENT PREDICTION AND PROGRAM DEVELOPMENT

5.1 Introduction

The backfill movement that is to be predicted by this work is that of creep settlement and collapse settlement which consequently enables differential settlements to be determined. Heave movements and lateral movements do occur within opencast backfill but their prediction is out of the scope of this work and generally speaking of only minor consideration when compared to the magnitude of backfill settlements.

Methods of both creep and collapse settlement prediction found in the literature will first be reviewed followed by the methods proposed by this work.

5.2 Methods of Creep Settlement Prediction

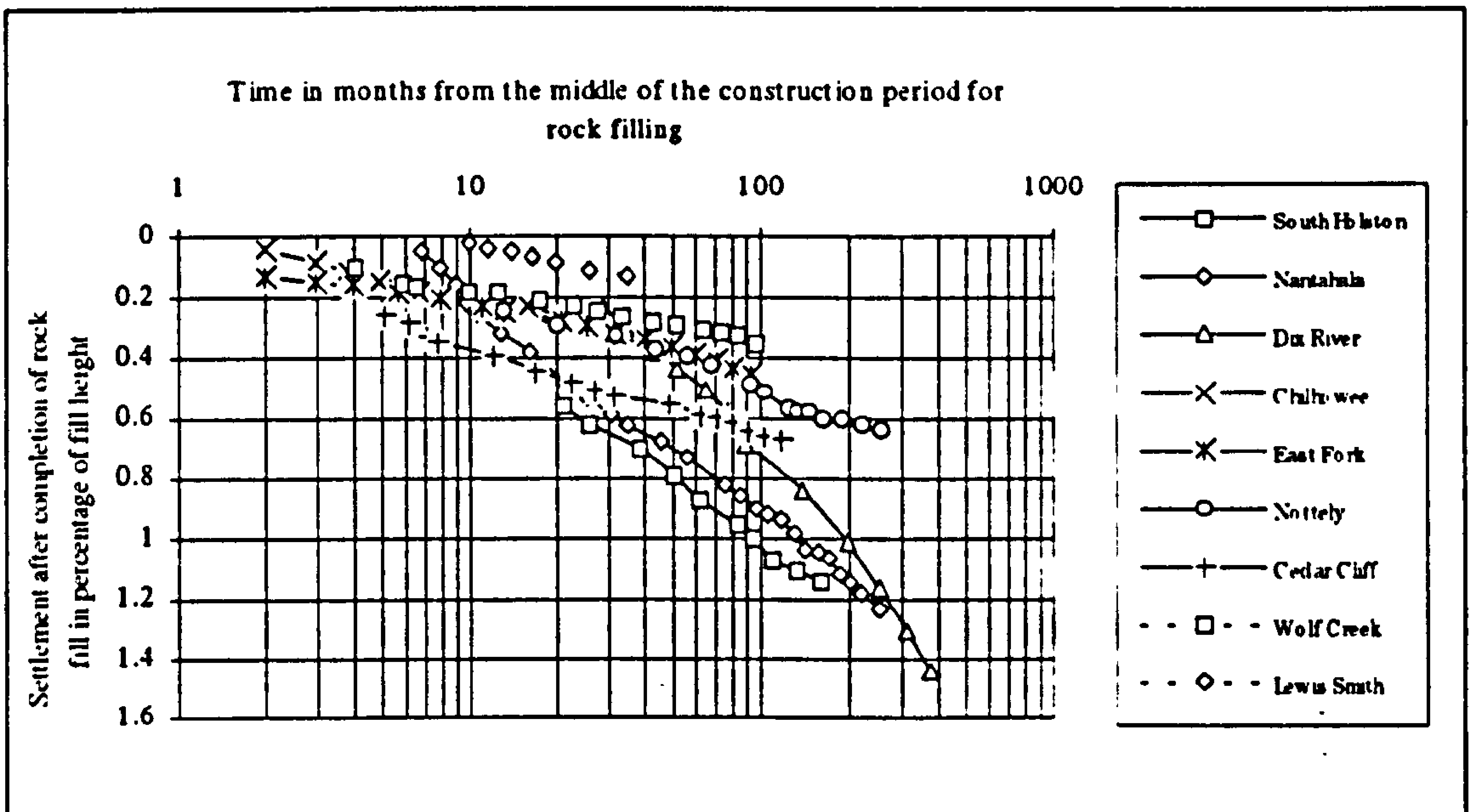
Creep settlement, as discussed earlier, is the long term process of the gradual re-arrangement of the opencast backfill fragments resulting in a reduction of voids under conditions of constant stress and moisture content.

Sowers *et al* (1965) examining the long term crest settlement of 14 rockfill dams demonstrated that the settlement, expressed as a percentage of the fill height, showed a linear relationship with log time from the middle of the dams construction period, as shown in Figure 5.1. It has been shown that a similar relationship is found when examining the long term settlement (creep settlement) of opencast backfills, Figure 5.2. Utilisation of this relationship has led to the widely accepted method of predicting opencast backfill settlement based upon the following equation:

$$s=\alpha\left(\log_{10} t_2-\log_{10} t_1\right) \qquad (5.1)$$

where s is the creep settlement expressed as a percentage of the fill height (creep strain), that occurred between times t_1 and t_2 from the beginning of the period of creep settlement and α is the creep compression rate parameter (alpha). Difficulty arises in determining the time at which creep settlement began (t_0) with layers placed first clearly beginning to settle before the placement of latter layers. Sowers *et al* (1965) took this time to be the date when half the fill was completed.

Parkin (1977) proposed an alternative method of interpretation by means of rate analysis based upon the theory of rate processes. In this, deformation is restricted by a series of discrete energy barriers. A constant, and random, energy exchange occurs between particles, and displacement occurs when any particle acquires sufficient energy to overcome an energy barrier. The rate of displacement is thus determined by the probability of any particle acquiring the necessary energy. There is a gradual distribution of energy through the system, so that the probability of particles acquiring critical energy decreases with displacement and therefore, time.

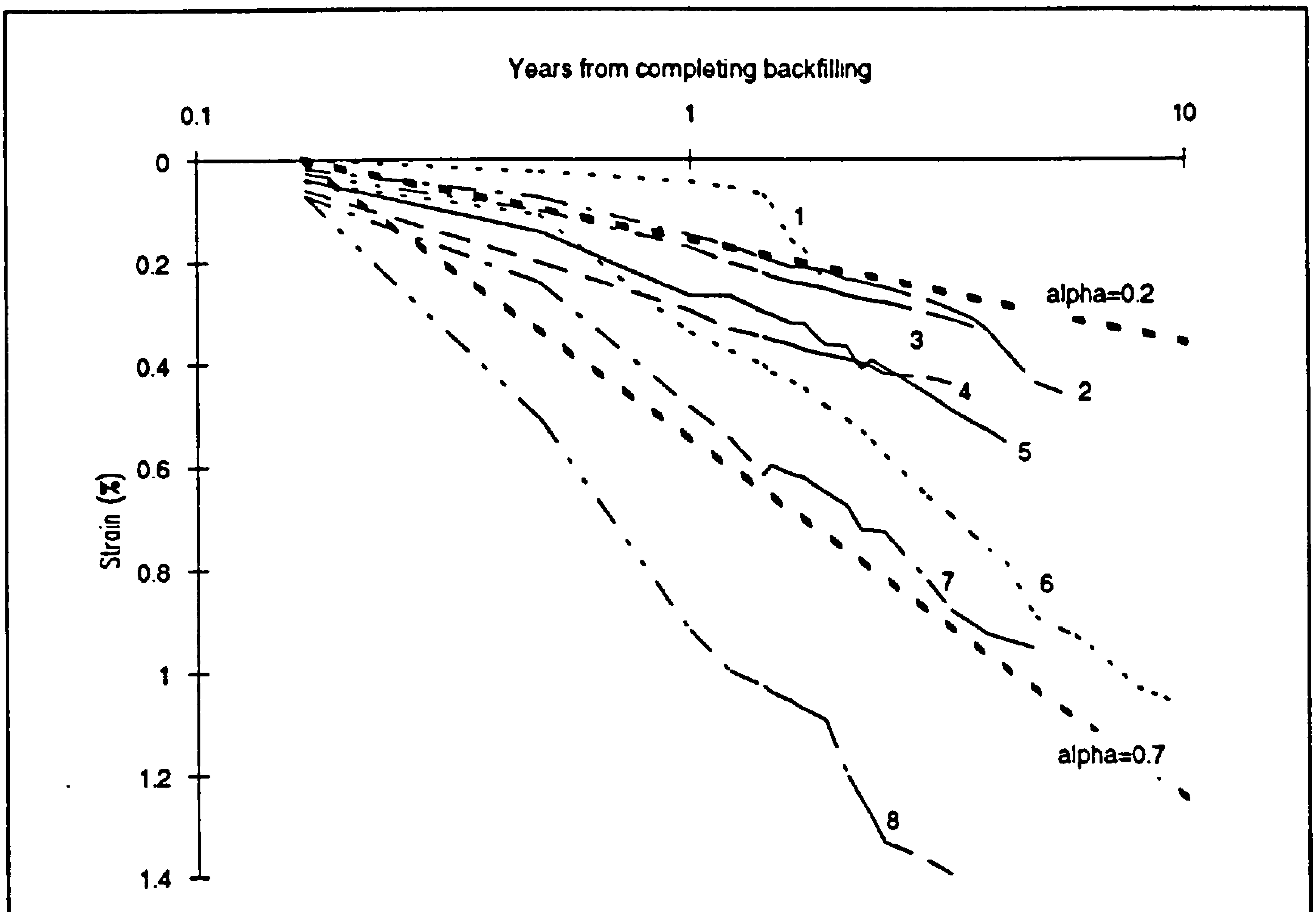


Dam	Height (m)	Type*	Rock
South Holsten	87	CC	Sandstone
Nantahala	78	SC	Greywacke
Dix River	84	SD	Limestone
Chilhowee	23	SC	Schist
East Fork	41	SC	Schist
Nottely	56	CC	Quartzite
Cedar Cliff	50	SC	Schist
Wolf Creek	50	SC	Schist
Lewis Smith	97	SC	Sandstone

* SC - Sloping Core, CC - Central Core, SD - Sloping Deck Types of Dams

Figure 5.1. Observed settlement of rockfill dams after completion of construction (after Sowers *et al*)

If the above described rate process character of creep in rockfill is accepted, then the creep strain rate $\dot{s}$ can be expected to follow the exponential decay function as follows:



Site	Degree of Compaction
Clockfields (8)	Uncompacted
Chibburn (6) Horsley (1)	Relatively Uncompacted
Dibdale West (2) Clockfields (4) Amblecote House (5) Turners Lane (7)	Spread in layers using box scrapers
Amblecote North (3)	Spread in layers and compacted with vibrating rollers

Figure 5.2. Settlement against log time for various opencast coal sites (Knipe 1979)

$$\dot{s}=a(t-t_0)^{-m} \tag{5.2}$$

where a and m are constants and t_0 is the point of initiation of creep.

Expressing in logarithmic form i.e.

$$\log\dot{s}=\log a-\log(t-t_0)\,.\,m \tag{5.3}$$

enables double logarithmic plotting to give the values of a and m, provided the initial time t_0 is known. In most cases, it is found that m is close to 1 with some fluctuations. In the case where m is equal to 1 the above creep strain rate relationship integrates to the logarithmic relationship of Sowers *et al* (equation 5.1) i.e.

$$\dot{s}=\frac{ds}{dt}=a(t-t_0)^{-1} \tag{5.4}$$

integrating between the limits of time T_2 to T_1 gives:

$$s=\int_{T_1}^{T_2} a(t-t_0)^{-1}.dt \tag{5.5}$$

$$s=a\left[\ln(t-t_0)\right]_{T_1}^{T_2} \tag{5.6}$$

(T_2-t_0) and (T_1-t_0) become the times t_2 and t_1, respectively as in Sowers's equation i.e. times from the beginning of the period of creep settlement. Thus we get:

$$s=\frac{a}{\log_{10} e}(\log_{10} t_2-\log_{10} t_1) \tag{5.7}$$

$$s=\alpha(\log_{10} t_2-\log_{10} t_1) \tag{5.8}$$

For Parkin's method to be used as a means of settlement prediction the constants a, m and t_0 have to be identified, then equation 5.2 can be integrated over the limits of the period of prediction as referenced from t_0, to give the magnitude of the creep strain over that period. As with Sower's method problems arise in determining the point of initiation of creep settlement t_0. Parkin (1971) proposed a method for estimating t_0 by writing equation 5.2 as:

$$(t-t_0)^m=a/\dot{s} \tag{5.9}$$

From which it can be seen that since m > 0, a graph of t against $1/\dot{s}$ must then indicate t_0 when $1/\dot{s}$ becomes zero, i.e. as $1/\dot{s}$ tends to zero t tends to t_0 (Parkin 1971).

Such a method of estimating t_0 does however require a considerable degree of interpretation as the graph of t vs $1/\dot{s}$, for which the relationship is not known, must be extrapolated back until $1/\dot{s}$ becomes zero.

As an example of the comparison between these two methods of settlement prediction settlement data obtained from the Cedar Cliff Dam (Sowers *et al*) has been used. Figure 5.3 shows the implementation of equation 5.1, with alpha being found equal to 0.289, thus over the period 100 to 400 months from the middle of the construction period, a strain 0.17% will occur.

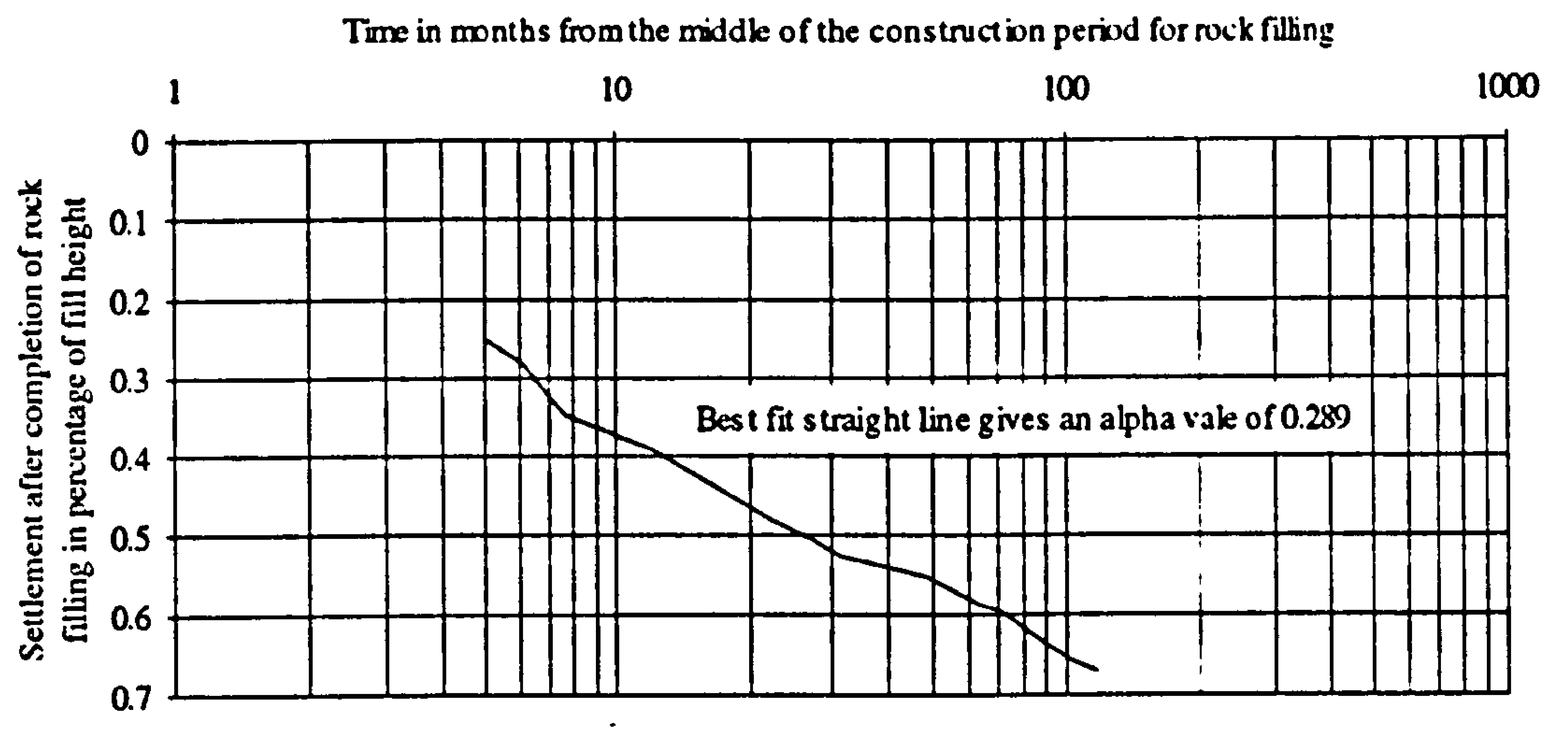


Figure 5.3. Implementation of Sowers creep strain analysis.

Implementation of Parkin's method is somewhat more complicated. The first stage is to establish t_0 as shown in Figure 5.4, a method of least squares was used to establish a straight line relationship which was then extrapolated to give t_0. This then enabled $\log \dot{s}$ to be plotted against $\log(t-t_0)$ from which a and m were found to be 0.039 and 0.762 respectively; again a method of least squares was used to determine the straight line relationship. Equation 5.2 therefore becomes:

$$\dot{s}=0.039(t-8.9)^{-0.762}$$

from which integration over the limits 100 to 400 gives:

$$s=0.039\left[\frac{(t-8.9)^{0.238}}{0.238}\right]_{100}^{400}=0.20\%$$

A slightly higher, but similar, value to that obtained by using Sowers *et al*'s method.

Parkin argued that the value of a rate analysis approach was in its ability to identify erratic behaviour within a set of settlement data thus enabling the creep component to be more easily considered in isolation thus leading to more accurate predictions. In practice however, the amplification of imperfections in the data, inherent to a rate analysis approach, make it difficult to distinguish the basic creep pattern from the irregularities (Clements R.P. 1984) thus a considerable degree of interpretation of the data is required to establish the parameters t_0, a and m.

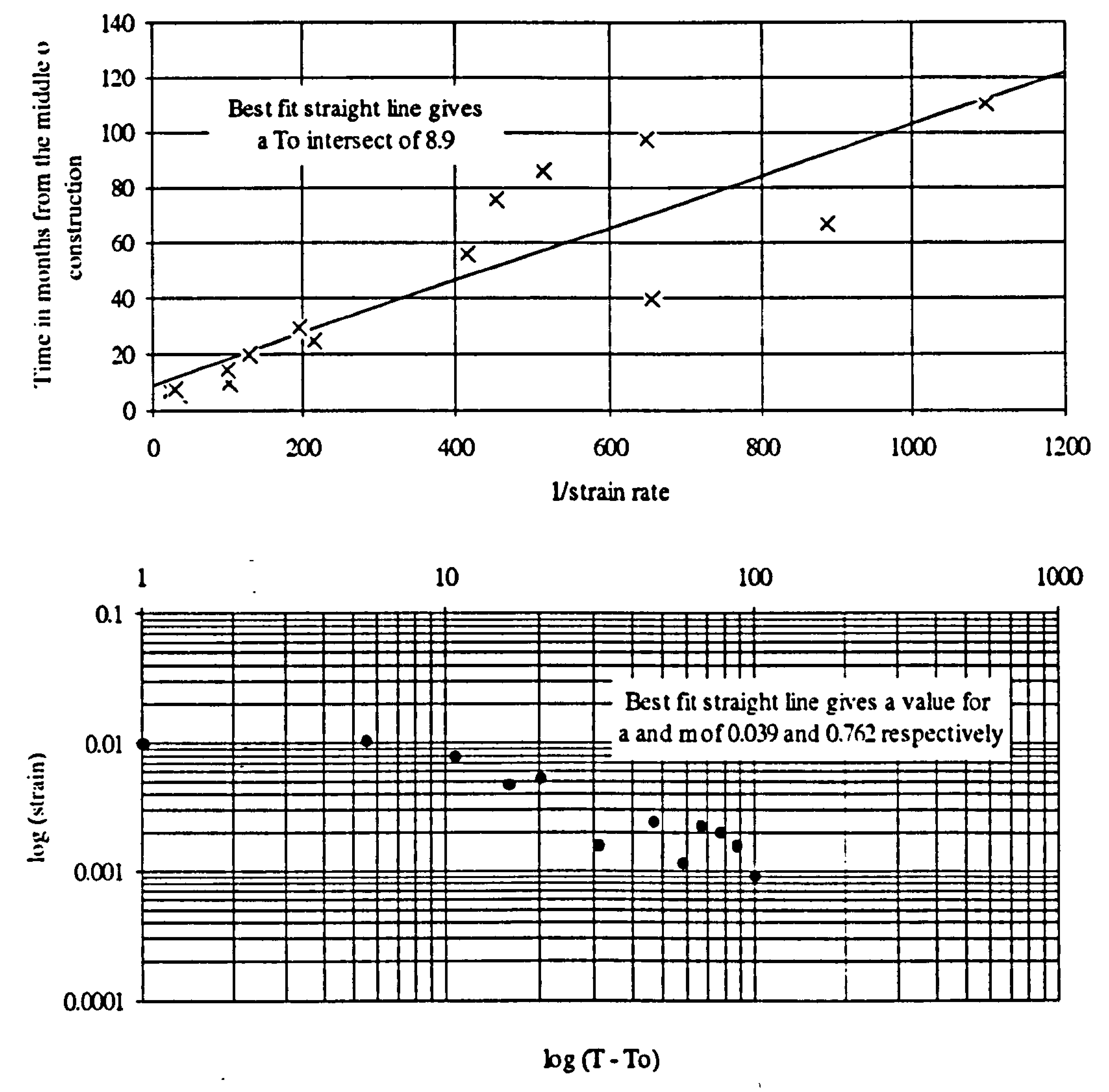


Figure 5.4. Implementation of Parkins' creep strain analysis.

Other author's examining the behaviour of creep settlement in rockfills include Soydemir C. and Kjaernsli B. (1979) and Clements R.P. (1984). Soydemir *et al* having examined the post construction performance data for 23 rockfill dams observed both displacement/time and displacement/dam height relationships. In an attempt to combine these relationships equations were produced for the displacement/dam height relationship for different time periods of the form:

$$S=\beta H^{\delta} \tag{5.10}$$

where S is crest settlement, H dam height and β and δ constants dependant upon the time period for which settlement is to be estimated and the method of dam construction, values of which are given in Table 5.1.

	Membrane-Faced (Dumped Rockfill) and Sloping Core Dams		Membrane-Faced (Compacted Rockfill) Dams	
	After initial impounding (2yrs)	10 years service	After initial impounding (2yrs)	10 years service
β	5.0×10^{-4}	1.0×10^{-3}	1.0×10^{-4}	3.0×10^{-4}
δ	1.5	1.5	1.5	1.5

Table 5.1. Settlement/dam height relationships after Soydemir C. and Kjaernsli B. (1979)

Clements R.P., again examining the post construction settlement performance for rockfill dams, concluded that existing methods of settlement prediction, as discussed above, based on empirical relationships of displacement with time or height, led to significant errors in the predicted value when compared to actual readings. An alternative approach was proposed based on comparison with dams having similar characteristics. Data was published giving the settlement characteristics and dam type and construction method for some 68 dams for comparison purposes.

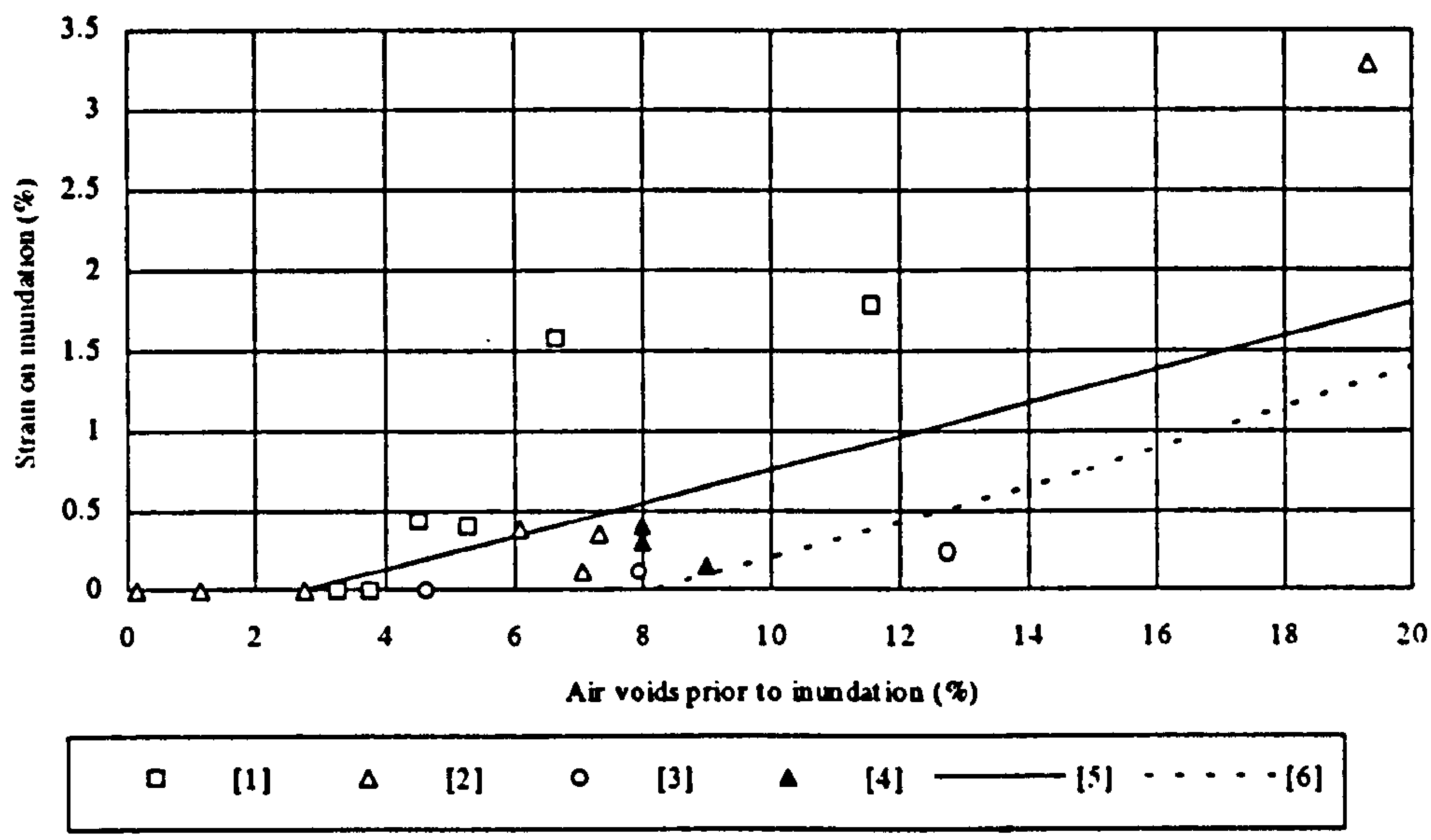
In the case of predicting settlement behaviour for opencast backfills, Sower's method is the most widely accepted with the majority of authors describing opencast backfill creep behaviour in terms of the creep compression rate parameter (alpha).

5.3 Methods of Collapse Settlement Prediction

The prediction of collapse settlement involves two stages, firstly the amount of backfill that is to become saturated must be determined, followed by the magnitude of collapse settlement that will occur within the saturated backfill.

The magnitude of collapse settlement will depend on the type of backfill, its density and moisture content, the stress level and the stress history (Charles 1984b). Laboratory tests carried out to determine its magnitude are presented in Figure 5.5.

Collapse settlement has been plotted against the air voids in the sample prior to inundation. It is considered that even though these plots do not take into account all the variables, they give a reasonable indication of likely collapse settlements. It is from such plots that the magnitude of collapse settlement is generally determined.



Graph	Material	Overburden Pressure (kN/m2)	Reference
[1]	Low plasticity clay	100	Charles (1984b)
[2]	Shale	100-260	"
[3]	Mudstone and sandstone fragments	400	"
[4]	Firm to stiff clay with shale fragments	-	Smyth-Osbourne and Mizon (1984)
[5]	Sandstone	-	Egretli and Singh (1988)
[6]	Mudstone	-	"

Figure 5.5. Relationship between air voids prior to inundation and collapse settlement

The mechanism by which backfill most typically becomes saturated is that of the re-establishment of the groundwater table upon completion of backfilling operations. It is generally considered that the groundwater table establishes itself to levels equal to those prior to mining relatively quickly upon completion of operations. Typical time periods are in the region of one year after mining and backfilling operations are completed. This has been confirmed in the majority of cases by the analysis of available data carried out by this work as described in chapter 3.

Collapse settlement can also occur as a result of partial saturation. most typically due to surface water infiltration. The source of surface water infiltration is most commonly rainfall, but other sources can exist such as the percolation of water into the backfill from water ways, ponds, lakes, or from leaking services or drainage trenches. The prediction of collapse as a result of surface water infiltration is complex as the amount of infiltration and the level of saturation have to be determined together with the resultant collapse which is considered to be less than that for complete saturation.

Observations carried out by this work generally indicate that collapse as a result of surface water infiltration from rainfall will be negligible some 2 years from the completion of compaction operations. As development of an opencast backfill site generally does not start for at least one year after backfilling, it is considered that there is little necessity in predicting collapse as a result of rainfall infiltration as it will in most cases be negligible during the design life of the development. Collapse as a result of infiltration from other sources of surface water will largely be negated provided adequate drainage measures are taken to prevent infiltration upon development of the site.

Collapse settlement as a result of surface water infiltration will therefore not be considered by this work, due, not only to the difficulties of its determination but the fact that given adequate drainage its magnitude will be negligible when compared to creep settlements and collapse as a result of complete saturation.

5.4 Proposed Method of Creep Settlement Prediction

Of the methods outlined above it is the analysis carried out by Sowers that will be used as a basis for the proposed method of settlement prediction. The reasons for this are as follows:

- Sowers analysis has been widely accepted as a means of predicting opencast backfill creep settlement and thus published data generally discusses the creep settlement behaviour of opencast backfill in terms of alpha values.

- The analysis proposed by Parkin requires a considerable amount of interpretation to establish the parameters t_0, a and m. This could potentially lead to different values being determined for the same backfill material due to different interpretations of the data. Also, as this has not been widely accepted as a means of opencast backfill creep settlement prediction, values for the parameters a and m have not been established in the literature.

- The approach taken by Soydemir and Kjaernsli, again, has not been adopted for opencast backfills and as such little or no information is available for the parameters β and δ in the case of opencast backfill. This method is also limited to pre-defined settlement prediction periods and as such is of limited use. Clements (1984) examining this method also found a considerable deviation from observed settlements to the relationship defined by $S=\beta H^{\delta}$, thus doubted the validity of such a relationship.

- The validity of the approach proposed by Clements (1984) may well be the best approach to be taken in the prediction of creep settlements within rockfill dams as a considerable amount of data has already been published on their behaviour enabling good comparisons to be made. It is felt though, that due to the greater variation of opencast backfill material and the limited amount of published data on the settlement characteristics of opencast backfill, a method of comparison would be of limited use

Having chosen Sowers' method it is fairly clear that the determination of the time at which creep strain commences and the creep compression rate parameter are critical to this analysis. For an accurate determination to be made values must be representative of the column of backfill material beneath the given point for which a settlement prediction is required.

5.4.1 Determination of the Commencement of Creep Strain

Inaccurate estimates of t_0 can lead to considerable errors in settlement prediction when estimates are base upon the relationship proposed by Sowers. In the situation for which the analysis was devised i.e. in the examination of the behaviour of earth/rockfill dams, t_0 was taken to be the time at which half the dam construction was completed. As material is generally placed lift upon lift such an approximation for t_0 can be made as the over estimate of settlement in the lowest layers will, to some degree, be compensated by the under estimate of settlement in the upper layers. The construction of an opencast backfill is generally very much different, the construction follows the advance of mining operations and as such does not create a structure consisting of layer upon layer across the whole site. Therefore, to take t_0 to be the time half the backfill construction was completed would introduce an error into the predicted settlement proportional to the distance from the point at which settlement is being calculated, to the point at which half the backfilling operations where completed (generally, the mid-point along the direction of mining operations). At an opencast mine taking 2 years to excavate and backfill, material could have been in place in the region of 300 days prior/after the assumed t_0 value leading, in some instances, to a considerable over/under estimate of settlement, respectively. The predicted settlement would simply mirror the depth of the backfill, taking no account of the schedule of mining/backfilling operations.

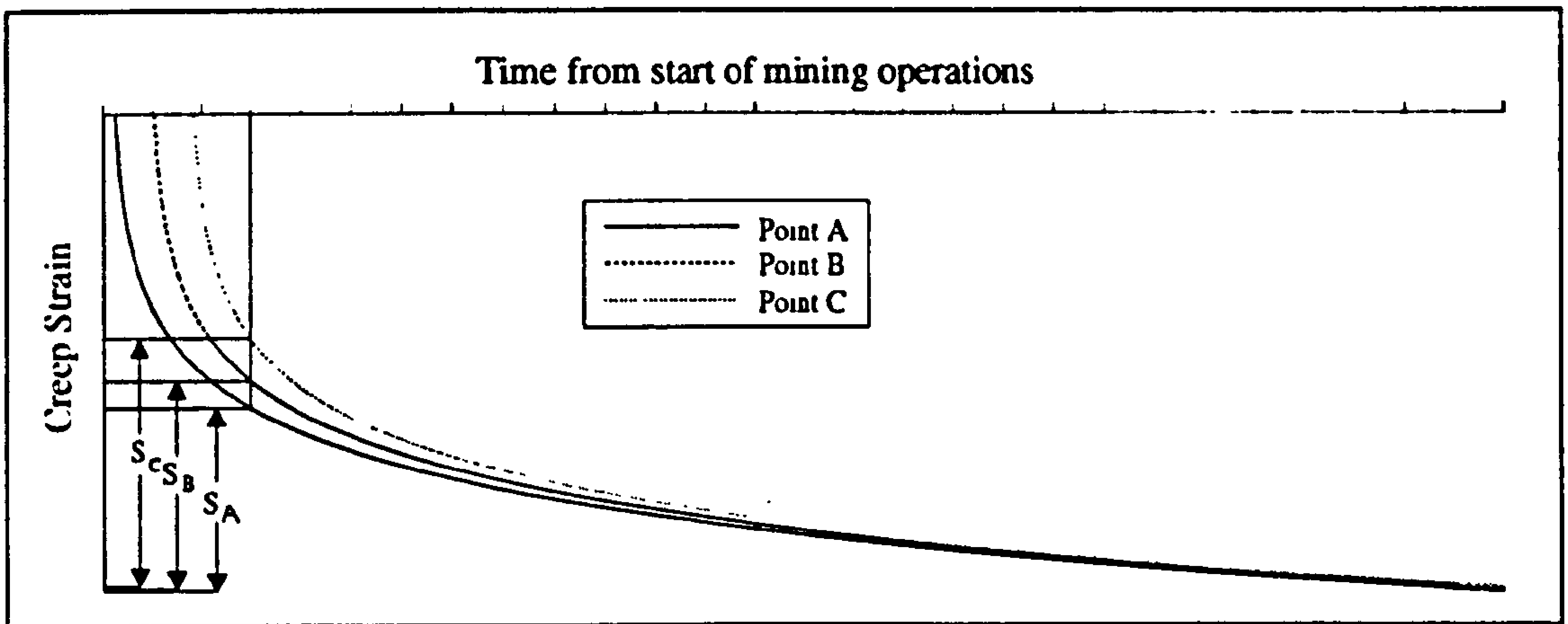


Figure 5.6. An example of the possible errors in the predicted strain when using t_0 as the mid-point of construction, for a mining operation taking 2 years to complete.

This point can be illustrated by considering an opencast mining and backfilling operation lasting 2 years and a settlement prediction period of 1 to 26 years after the completion of backfilling. Settlement predictions are to be made at three points along a line parallel with the direction of working, points A, B and C. Points A and C lie at either end of site, point A over the oldest fill, point C over the newest fill, point B lies in the middle of the site. Figure 5.6 shows the strain that will occur at the three locations demonstrating that if t_0 is taken as the mid-point of construction for all three points, an over estimate at point A and an under estimate at point C occurs. The magnitude of these over/under estimates amount to 12% and 20% respectively. If a construction period of 3 years is considered, as shown in Figure 5.7, an over estimate of 15% and an under estimate of 30% occurs.

We therefore require a better method of determining t_0 than simply taking it to be the mid point of construction. The method proposed by this work requires good knowledge of the backfilling operation. Information required is that of backfill placement rates and scheduling, methods of backfill placement and material properties. This then enables the backfill to be modelled, as a whole, as distinct blocks of material de-lineated by different periods of placement, compactive state and material properties. From such a model a column of material can be built up beneath any point for which a settlement prediction is required, consisting of a series of distinct blocks of backfill material. The time period over which each block is placed together with their settlement characteristics can then be determined enabling the settlement of the column of material to be estimated.

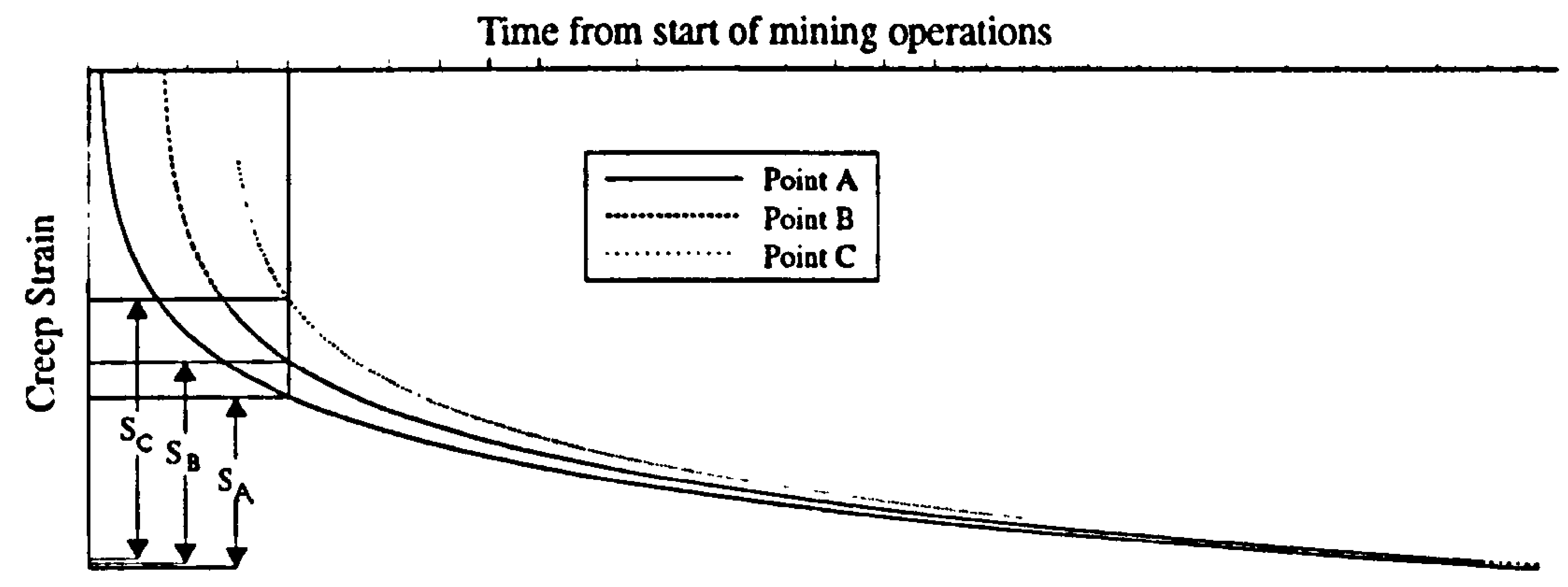


Figure 5.7. An example of the possible error in the predicted strain when using the mid-point of construction for t_0, for a mining operation taking 3 years to complete.

To determine the settlement of a given column of material t_0 has to be determined for the backfill material throughout the column. For any layer within the column, t_0 will lie between the time at which that layer was placed and the time at which the whole column was completed. The problem can be represented in Figure 5.8 together with the influence t_0 has on predicted settlement, as represented by the value of $\log_{10}t_2 - \log_{10}t_1$, over a 25 year prediction period 1 year after completion of mining operations. It could be argued that t_0 should be the time at which the whole column of material is completed as this is the time at which the effective vertical stress level becomes constant. Creep settlement occurs under conditions of constant stress as discussed above. It is however felt that within a column of material a point will be reached when the placement of material near the surface will have little influence on the behaviour of material at depth. It is considered that for the material at depth it is this point that marks the start of the creep settlement that is to be predicted. The time at which this point is reached will be determined from the time at which the percentage increase in effective vertical stress due to placement of material near the top of the column is less than a given percentage value. The problem is shown in Figure 5.9 which shows the effective vertical stress vs time relationships, within a column of backfill material consisting of four distinct blocks, at point A, the base of the column and at points B and C within the column. The point at which the creep strain (for which a prediction is to be made) initiates (t_0) will therefore be calculated, for any point within the a column of backfill, as the time at which the %-age effective vertical stress increase due to the

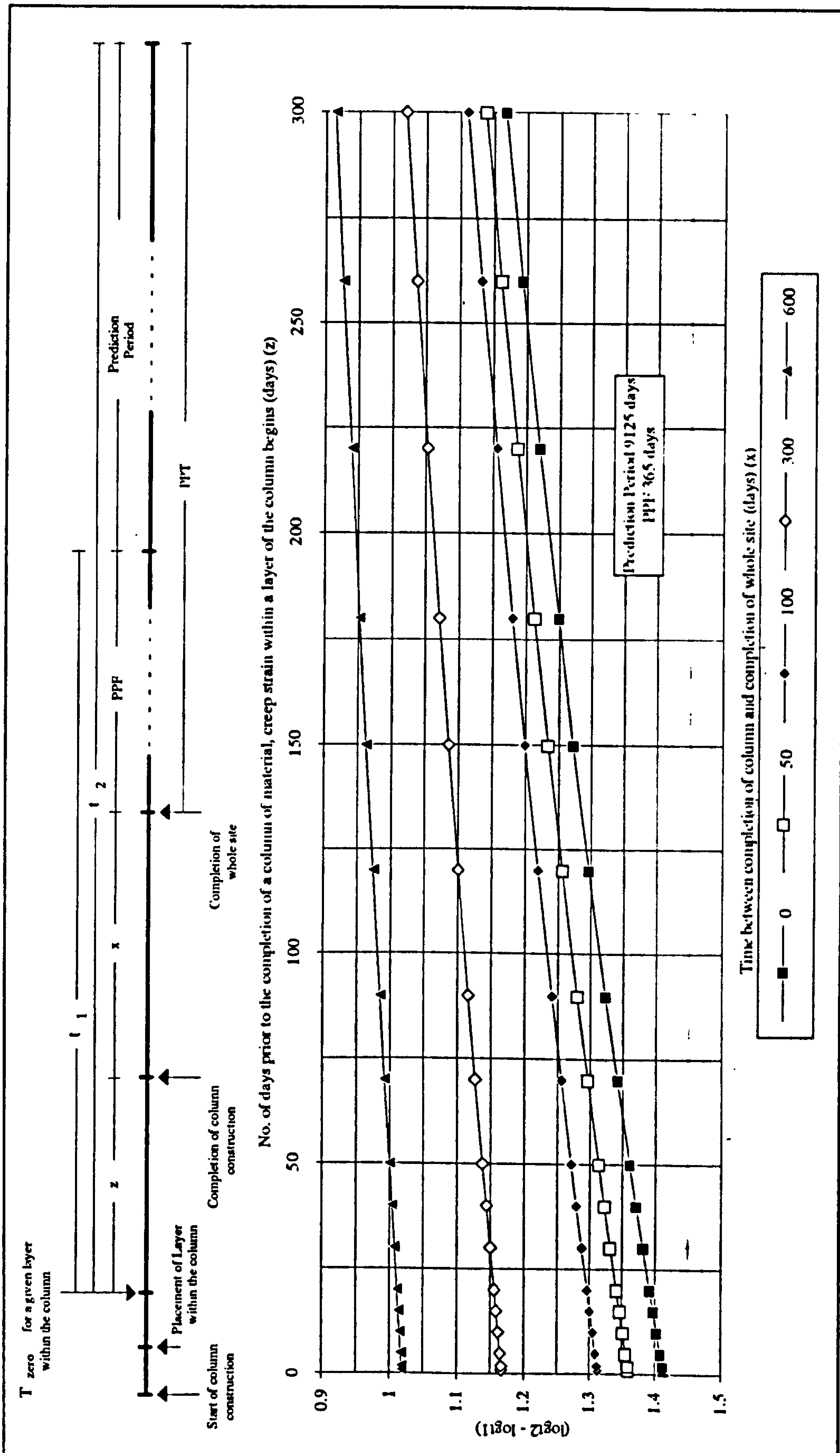
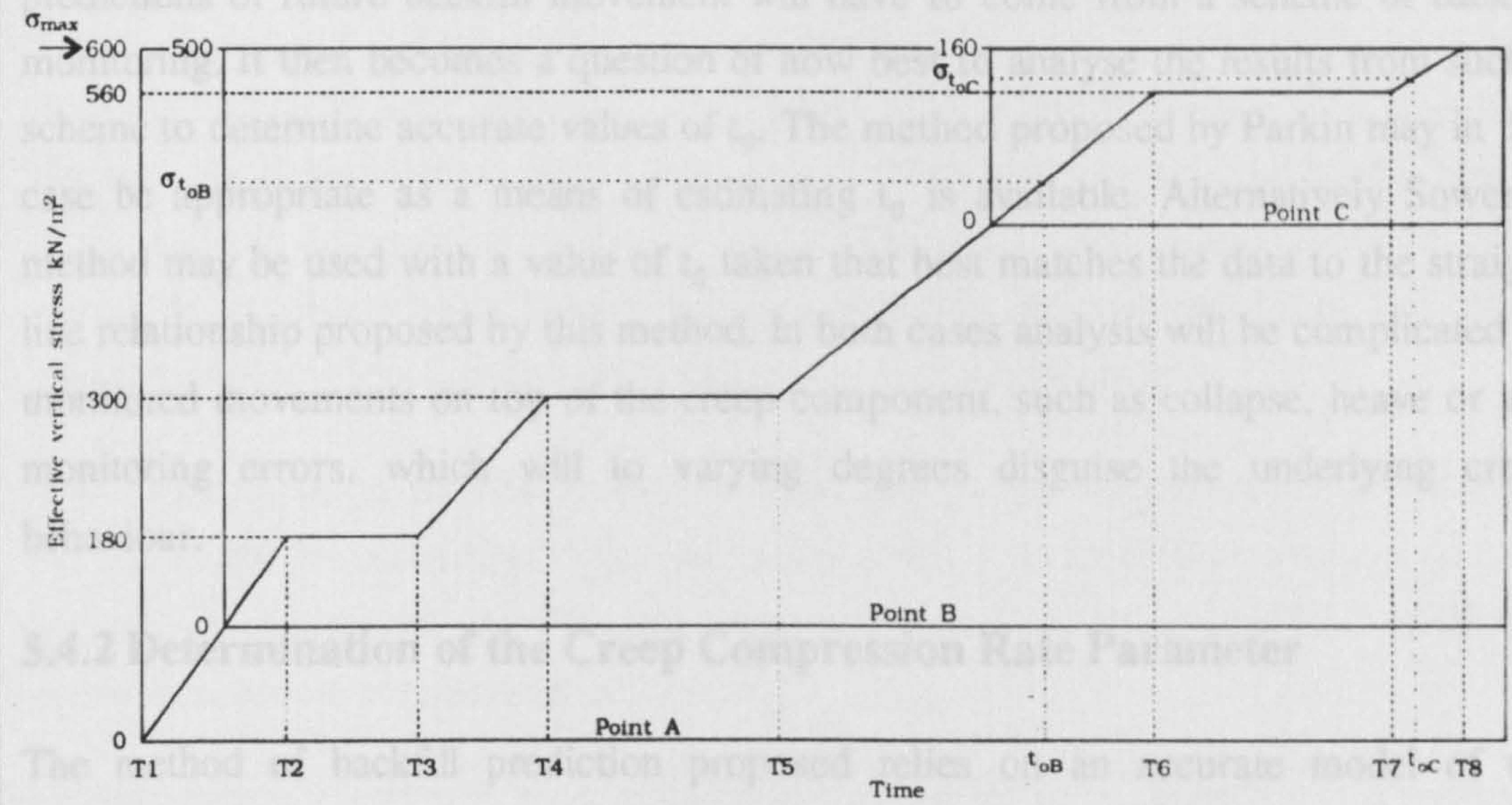
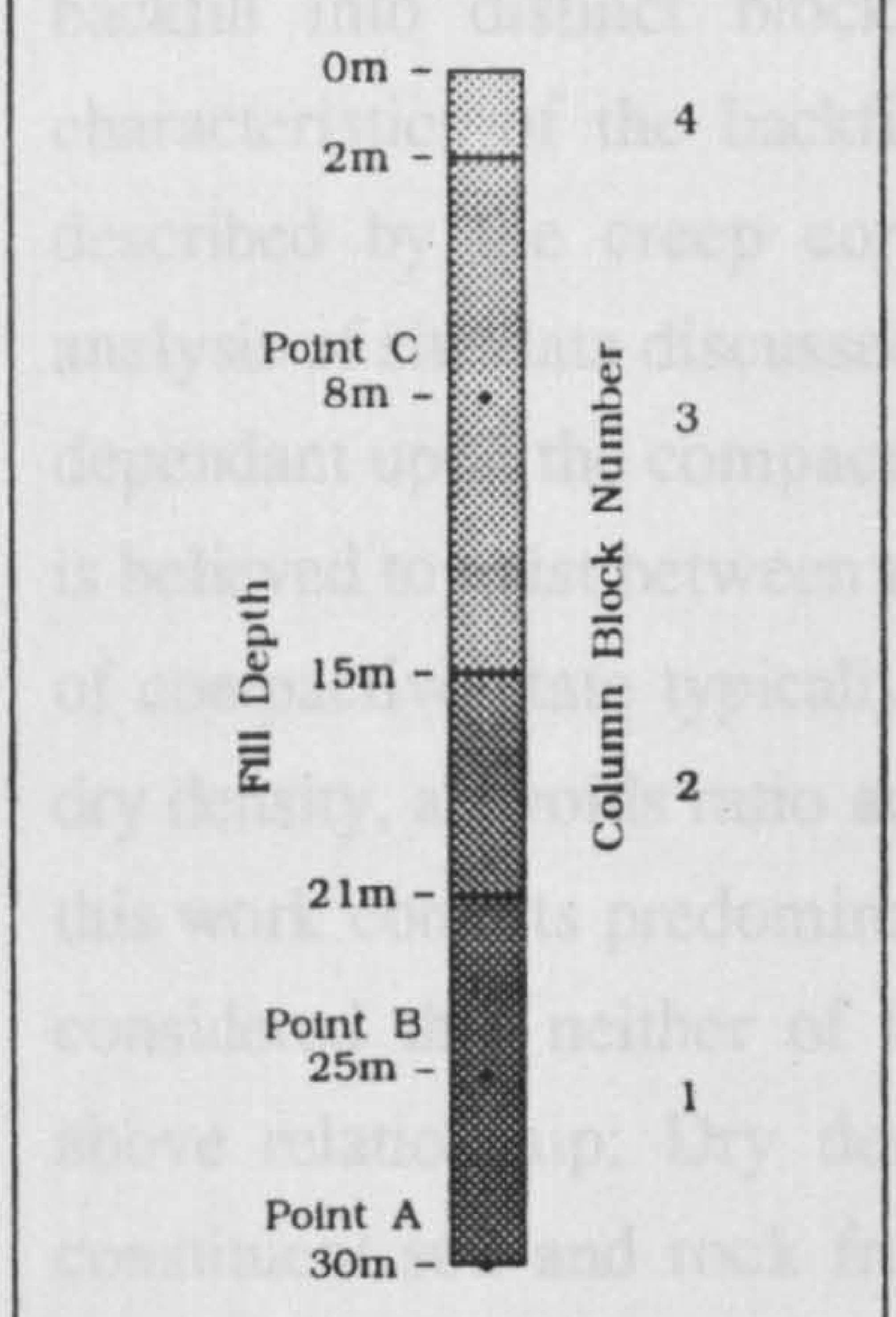


Figure 5.8. The influence of the zero time on the predicted settlement as represented by (logt2 - logt1)

placement of material above, reaches some "cut-off" value ($x\%$). x will be a user defined value during program execution and can be shown to be equal to:

$$\frac{\sigma_{max} - \sigma_{to}}{\sigma_{to}} \cdot 100 \quad \text{(Figure 5.9)} \qquad (5.11)$$





Block	Placement Time	
	Start	Finish
1	T1	T2
2	T3	T4
3	T5	T6
4	T7	T8

$$\sigma_{to} = \frac{\sigma_{max}}{\left(\frac{x}{100} + 1\right)}$$

In this example x (The Percentage Stress Increase 'Cut-Off' Value) is equal to 30%.

Figure 5.9. Effective vertical stress against time for points A, B and C within a column of backfill material

Thus knowing the value of both σ_{max} and x, σ_{to} can be determined which in turn gives t_0 (Figure 5.9).

It is assumed that during the construction of any given column of backfill, the groundwater table remains below the base of the column therefore effective vertical stress values will be determined simply from the unit weight of the overlying material.

In the case where the mining operation has already been carried out and insufficient data is available to construct an accurate model of the backfilling operation, predictions of future backfill movement will have to come from a scheme of backfill monitoring. It then becomes a question of how best to analyse the results from such a scheme to determine accurate values of t_0. The method proposed by Parkin may in this case be appropriate as a means of estimating t_0 is available. Alternatively Sowers's method may be used with a value of t_0 taken that best matches the data to the straight line relationship proposed by this method. In both cases analysis will be complicated by monitored movements on top of the creep component, such as collapse, heave or any monitoring errors, which will to varying degrees disguise the underlying creep behaviour.

5.4.2 Determination of the Creep Compression Rate Parameter

The method of backfill prediction proposed relies on an accurate model of the backfilling operation to be constructed as discussed above. The model splits the backfill into distinct blocks de-lineated by, among other things, the settlement characteristics of the backfill material. In the case of creep settlement this will be described by the creep compression rate parameter (alpha). The results from the analysis of site data discussed in chapter 3, indicate that the value of alpha is very much dependant upon the compactive state of opencast mine backfill. An inverse relationship is believed to exist between alpha and some measure of the compactive state. Measures of compactive state typically used to monitor the placement of opencast backfills are dry density, air voids ratio and relative density. Site data made available for analysis by this work consists predominantly of dry density and air voids ratio measurements. It is considered that neither of these measures would be appropriate in establishing the above relationship; Dry density is very much dependant upon the density of the constituent soil and rock fragments, thus backfills having the same compactive state could exhibit a wide range of dry densities due to differences in the constituent material type. Air voids ratio as a measure of compactive state is very much dependant upon moisture content, backfills having the same air voids ratio could have very different compactive states if there is a large difference in moisture content. It is considered that relative density measurements would be most appropriate being the percentage of the ratio between the in-situ dry density and the maximum dry density as determined from laboratory results. Thus the measure is independent of both the density of the constituent material and the moisture content of the backfill as a whole. It is however important that for such a relationship to be established, a standard laboratory test is used for the determination of the maximum dry density that is appropriate to the compaction of opencast backfill material.

Insufficient relative density measurements were made available for a relationship between alpha and relative density to be made. The establishment of this relationship would have to come from further density monitoring during the placement of opencast backfills combined with laboratory testing and settlement monitoring. The analysis of the data available enables only a very broad relationship to be established between alpha and the compactive state as given in Table 4.1. The typical variation values mirror the increased heterogeneity that occurs within opencast backfill as less control and subsequently less compaction is carried out during placement.

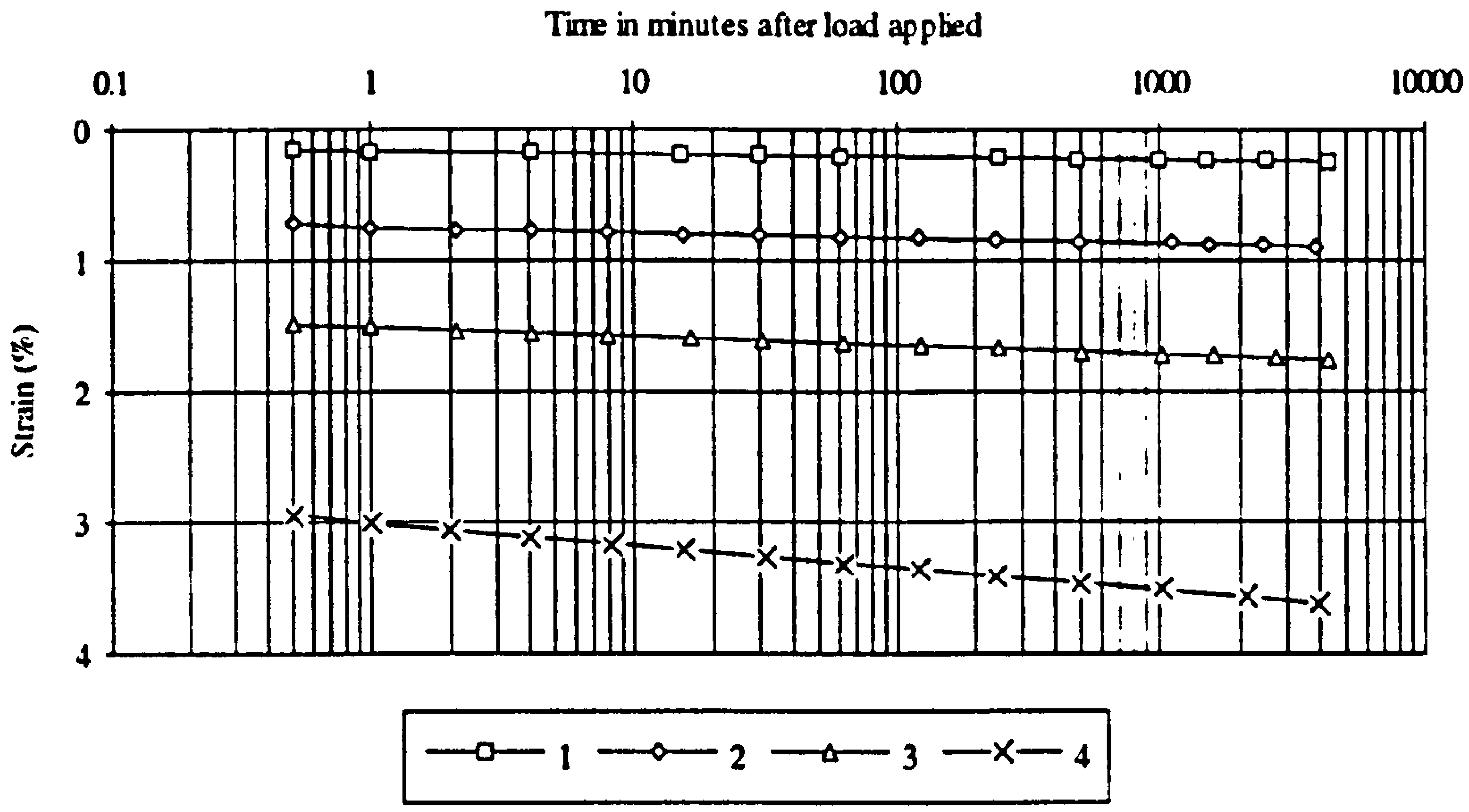
The value of alpha may be further complicated by the magnitude of the effective vertical stress the material is subject to during creep. Charles (1993) demonstrated that for heavily compacted rockfills alpha was stress-dependant; results from BRE field measurements are summarised in Table 5.2.

Fill Type	Compaction	Alpha	Reference
Sandy gravel fill	Heavy vibrating roller	$0.04\sigma'_v$*	P7.2
Mudstone fill	Heavy vibrating roller	$0.12\sigma'_v$*	P7.1
Sandstone/ Mudstone rockfill	Heavy vibrating roller	$0.13\sigma'_v$*	P9

Table 5.2. Relationship between alpha and effective stress (after Charles J.A. 1993)

* σ'_v - effective vertical stress in MPa

Sowers *et al* (1965) examining the influence of the effective vertical stress upon the creep compression rate parameter produced the results as shown in Figure 5.10. These results give a reasonable fit to the straight line relationship proposed by Charles (1993) giving a gradient of 0.10 i.e. alpha = $0.10\sigma'_v$. There is however a need for more published case histories to confirm the general applicability of this type of relationship. The data available for analysis by this work was not sufficiently detailed to either further establish or disprove such a relationship. As such the value of alpha to be taken by this work will be based upon the analysis of the available data as shown in Table 4.1. The influence of the vertical effective stress will therefore not be taken into consideration. It is felt that this is an acceptable approximation as an over estimate of alpha in the lower stress ranges will be compensated by an under estimate of alpha in the higher stress ranges.



Data Series	Effective Stress (KN/m^2)	Alpha
1	78	0.02
2	312	0.04
3	781	0.07
4	1561	0.17

Figure 5.10. The influence of the effective vertical stress on the value of the creep compression rate parameter, alpha. (after Sowers *et al* 1965)

5.5 Proposed Method of Collapse Settlement Prediction

The method used by this work will be similar to that used by previous authors, whereby the magnitude of collapse upon saturation will be determined from the air voids value of the backfill. Insufficient data was available from the study sites to more accurately identify the relationship between collapse and air voids than that shown above in Figure 5.5, but from the data available the relationship as shown in Figure 5.11 is proposed. The possible range of the collapse for a given air voids value is shown by upper and lower bounds which are thought to deviate further from the best fit line with increasing air voids. This is considered, as increasing air voids can be associated with backfill placed with progressively decreasing compaction and placement control which results in an increasingly heterogeneous backfill with a resultant increase in scatter. Three regions have been identified in Figure 5.11 indicating degrees of backfill placement. These regions are based upon typical air void measurements carried out at the study sites. The values for the collapse strain parameter given in Table 4.1 are based upon Figure 5.11.

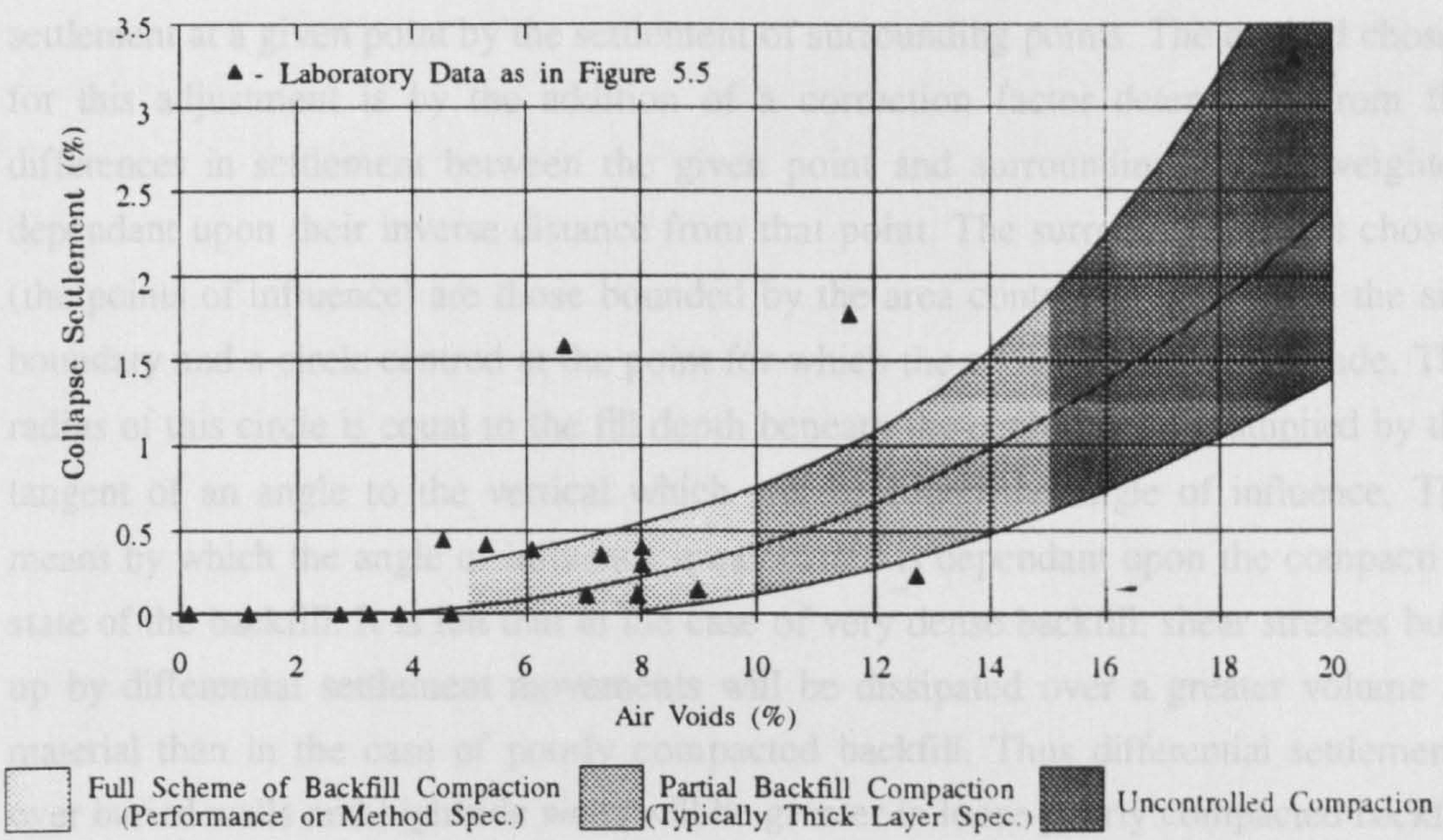


Figure 5.11. Proposed relationship between collapse settlement and air voids prior to inundation.

As pointed out above, such a plot does not take into account all the variables that effect collapse strain, but it is considered that it gives a good indication of the collapse that can be expected when a layer of opencast backfill, consisting predominantly of coal measures from which the results are based, becomes inundated.

The determination of the volume of backfill that will become inundated will be based upon the groundwater table prior to mining with an anticipated recovery within 1 year from the completion of mining operations provided no external influences effecting the water table.

5.6 Influence of Surrounding Material

It must be considered that the settlement at a given point cannot be taken in isolation as it will be influenced by the settlement of surrounding points. A sudden increase in settlement due to a rapid increase in fill depth (as in the case of crossing a buried wall) will be dissipated by the surrounding backfill due to a build up of shear stresses between the material that is settling at a greater rate as the wall is traversed than that above the top of the wall. Such an effect is borne out by the results obtained from site monitoring where less than expected differential settlements have been observed over buried walls and high/side walls.

This is incorporated into the method of settlement prediction by adjusting the settlement at a given point by the settlement of surrounding points. The method chosen for this adjustment is by the addition of a correction factor determined from the differences in settlement between the given point and surrounding points, weighted dependant upon their inverse distance from that point. The surrounding points chosen (the points of influence) are those bounded by the area contained within both the site boundary and a circle centred at the point for which the adjustment is to be made. The radius of this circle is equal to the fill depth beneath the central point multiplied by the tangent of an angle to the vertical which will be called the angle of influence. The means by which the angle of influence is calculated is dependant upon the compactive state of the backfill. It is felt that in the case of very dense backfill, shear stresses built up by differential settlement movements will be dissipated over a greater volume of material than in the case of poorly compacted backfill. Thus differential settlements over buried walls and high/side walls will be greater in loose poorly compacted backfill than in dense well compacted backfill. To determine the angle of influence a linear relationship between it and alpha is proposed with a lower bound value of 10° for very loose material (alpha value of 1.0) and an upper bound value of 60° for very compact material (alpha value of 0.2). This is based upon the angle of draw associated with ground settlement as a result of longwall coal mining (Tomlinson 1986) and differential settlement observations.

The points selected as the points of influence are bounded by the area contained within both the site boundary and the circle defined by the angle of influence. The points within this area are positioned such that they lie on the perimeter of a set of circles radiating out from the central point with the final circle being that which is defined by the angle of influence. The number of circles is user defined during program execution. The positioning of the points on these circles is such that for a user defined value of n, n multiplied by 4 (the 4 quadrants of a circle) points are positioned equi-distant around each of the n circles. The affect of the number of points of influence on the predicted settlement is examined in the following chapter.

5.6 Differential Settlement

Differential settlements will occur across changes in fill depth, across boundaries delineating backfill of different compactive states and periods of placement and across zones of unequal saturation. Such differential settlements will be identified from backfill settlement determinations based upon the methods described above. Differential settlements will also occur however, as a result of the heterogeneous nature of opencast backfill, therefore within an area overlying backfill of the same depth, compactive state, period of placement and saturation, differential settlements will still occur. This backfill heterogeneity has been expressed above by the ± values for alpha and the upper and lower bound lines for the collapse strain value in Figure 5.11.

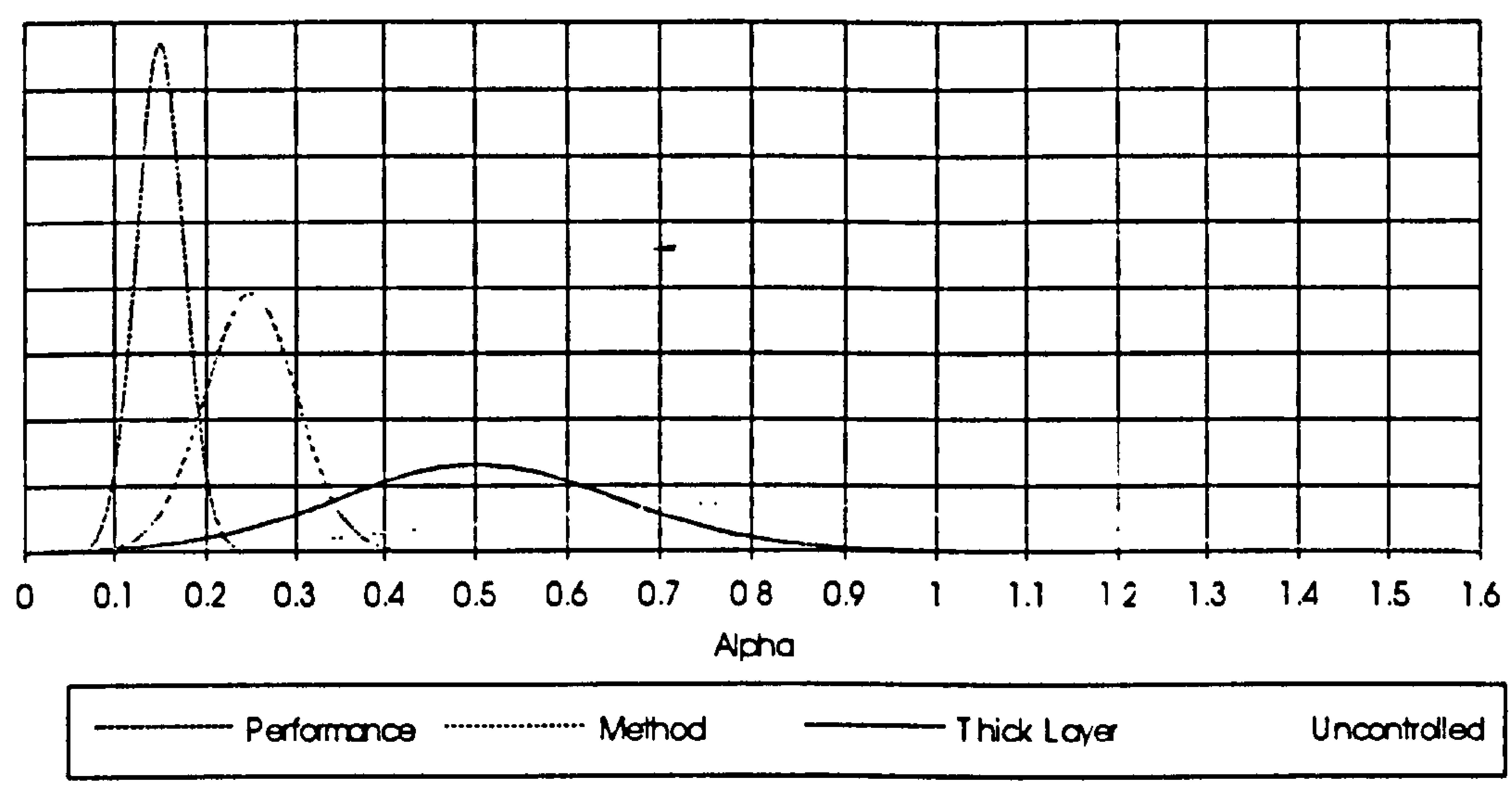


Figure 5.12. The normal distribution of alpha for different methods of backfill placement.

The approach taken to calculating differential settlement due to heterogeneity is to repeatedly calculate the settlement at the two points for which differential settlement is required, with values for both alpha and collapse strain being chosen randomly from populations of values having a normal distribution. The average value of these populations will be that taken from Table 4.1 for alpha and Figure 5.11 for collapse. The standard deviation (s.d.) is determined from the ± / upper and lower bound values such that 95% of the population lies within these values. This generates the populations as shown in Figure 5.12 and 5.13 for alpha and collapse respectively.

The method by which alpha and collapse values are chosen from their respective populations is by the Monte Carlo Simulation (Hamdy A. Taha 1982). This is a technique by which values are chosen randomly from a population such that the resultant sample population mirrors that of its source i.e. given enough sample points the average and variance of the sample population will be approximately equal to that of the source population. The number of times the simulation has to be repeated, to get a reasonably representative population, is dependant upon the variance of the source population, this value will however be user defined during program execution and is examined in the following chapter.

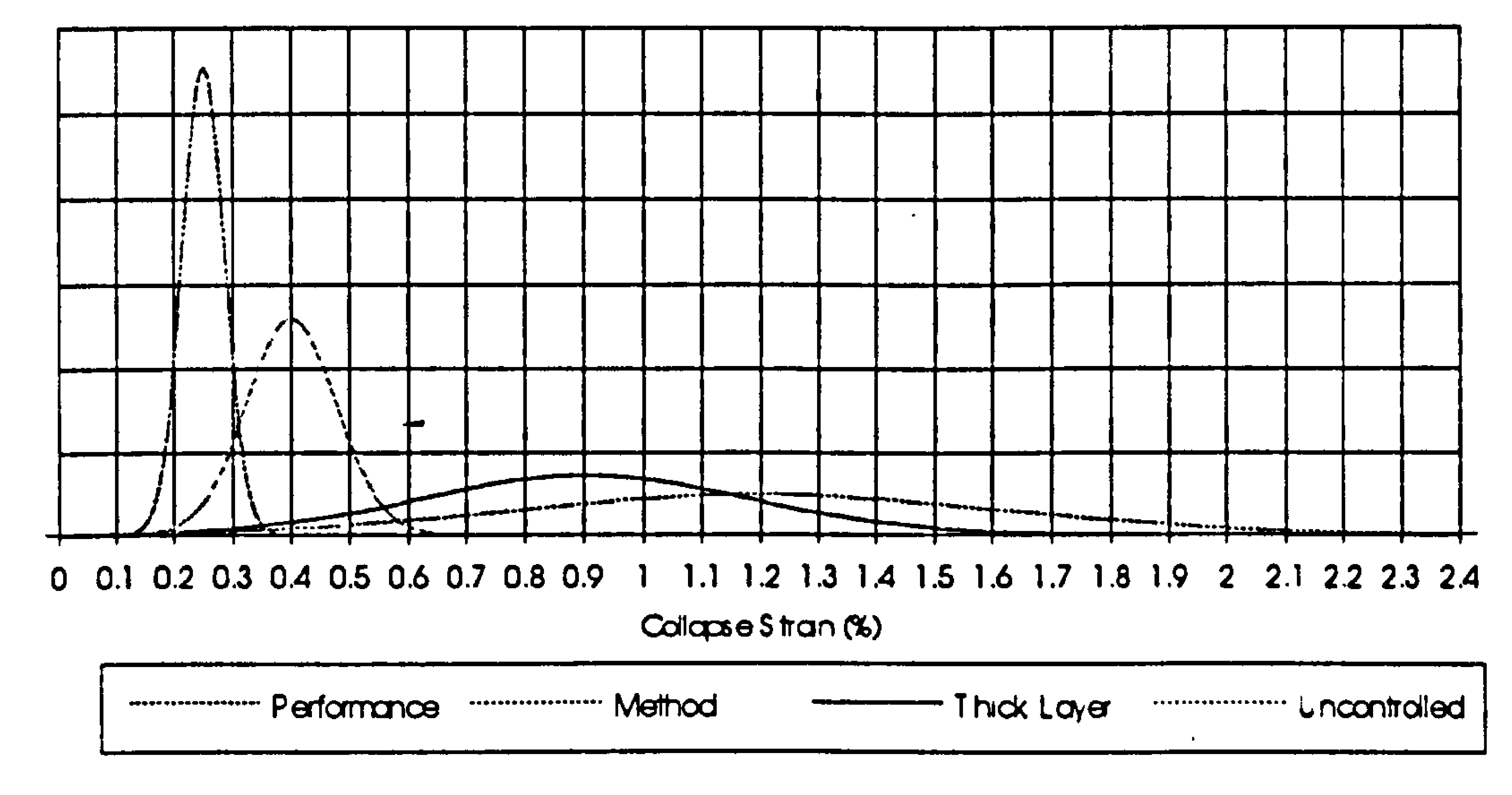


Figure 5.13. The normal distribution of collapse strain for different methods of backfill placement.

The result of this is two populations of settlement values, A and B, for the two points for which differential settlement is being measured from which due to the additive property of two independent normal distributions (Spencer *et al* 1977), a third population can be generated for the difference in settlement between the two points. This is defined by an average value equal to the difference of the averages of populations A and B and a variance equal to the sum of the variances of population A and B.

The generation of populations A and B does not however incorporate adjustment for surrounding material. This is because any adjustment made will have the effect of bringing the value of the settlement as determined from randomly chosen backfill properties towards the value as determined using average backfill properties. Thus the variance of the resultant settlement populations will be unrealistically low.

To accommodate for surrounding material the average of the population representing the difference in settlement is set equal to the difference in settlement between the two points as determined from a single calculation using average values for both alpha and collapse with adjustment for surrounding material carried out. Thus the Monte Carlo Simulation gives an indication of the variance of the difference in settlement between the two points whilst a one off calculation with adjustment carried out gives the absolute value of the difference. This is represented in Figure 5.14.

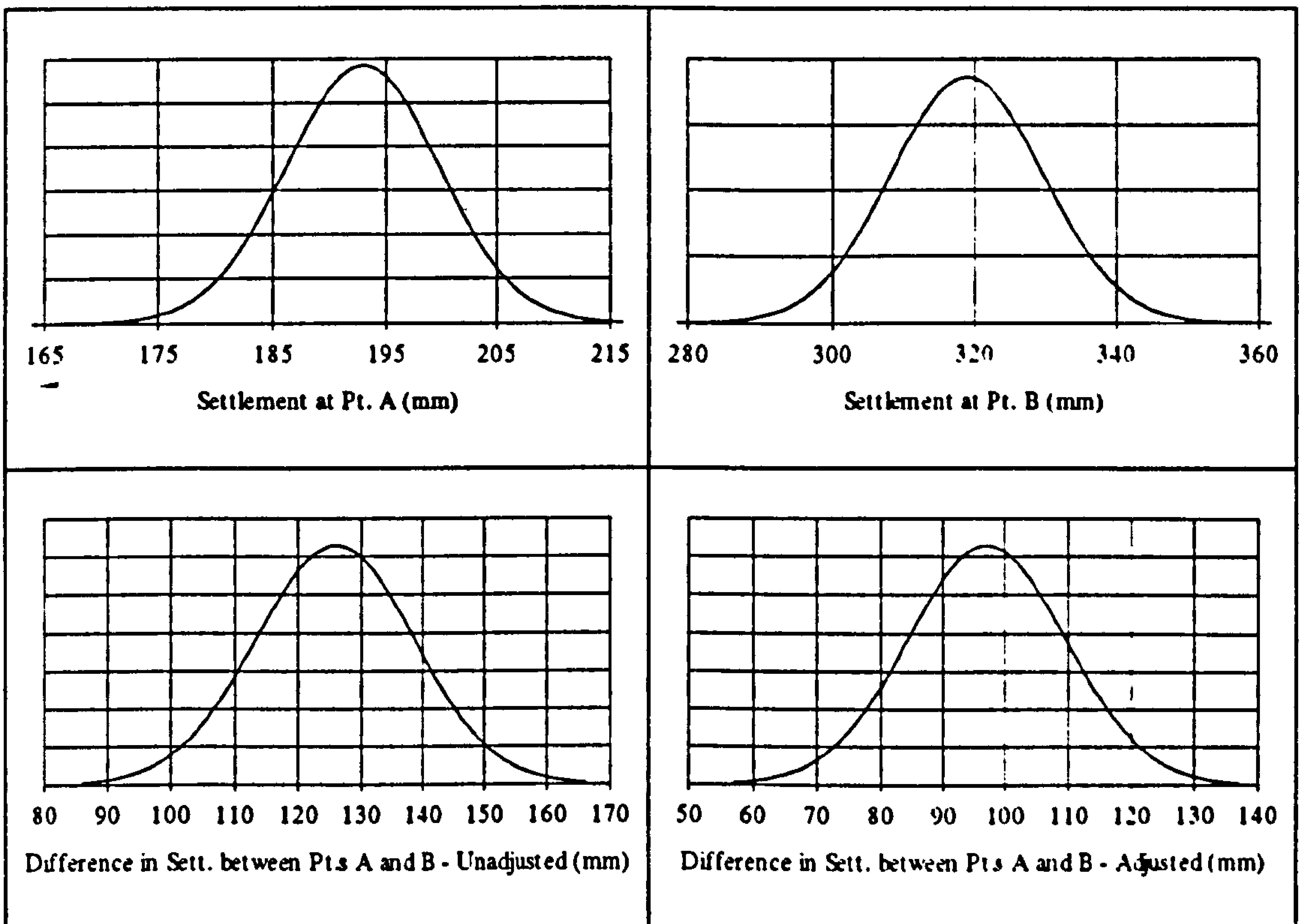


Figure 5.14. The method by which the difference in settlement, due to backfill heterogeneity, is calculated between two points.

5.7 Program Development

The above discussion outlines the methods by which opencast backfill settlement will be predicted. To implement these methods a computer program has been developed by the Author which will be referred to as OBSett (Opencast Backfill Settlement Prediction Package) in the following discussion. The development of OBSett involved two distinct stages; DTM interpretation and settlement calculation. Prior to this however backfill modelling is required providing the necessary input information for OBSett to enable settlement predictions to be carried out.

5.7.1 Backfill Modelling

In the majority of cases opencast backfills will have a complicated structure. They can be constructed by a number of methods and techniques are available after construction to modify them. They will be made up of material not only having different compactive states dependant upon methods of placement and post-constructional improvement techniques but also different material properties. They will be constructed over a considerable period of time and thus certain backfill properties which are time dependant will change over the construction period. They will be affected by changes in the groundwater table.

This therefore produces a backfill that can be envisaged as being made up of a series of blocks each of which has a different set of properties. The problem of predicting backfill settlement is in identifying these blocks and assigning to them appropriate settlement characteristics. The blocks can be identified by modelling the backfill as a group of blocks delineated by different periods of placement, compactive states, material properties and by groundwater table levels before and after the settlement prediction period. Groundwater levels apart, such blocks can be identified at the planning stage of the backfilling operation. Predictions for groundwater table recovery rates and levels are required to identify the blocks that will become saturated during the settlement prediction period. Settlement characteristics can then be assigned to each block by giving then appropriate creep compression rate and collapse settlement parameters.

The actual modelling of the backfill carried out by this work, has been done with the mine design package SURPAC. This enables a backfill to be built up as a series of distinct blocks as bound by triangulated surfaces. The triangulated surfaces generated by SURPAC are known as digital terrain maps (DTMs). The outline of the backfill mass as a whole is defined by the pit base and the final restored ground level. The pit base bounding surface is also represented by a DTM and the final restored ground level is made up of the last blocks placed. An example of the pit floor of the Newdale site together with one hypothetical backfill block is shown in Figure 5.15.

The position of the groundwater table at the start and end of the settlement prediction period will define the material that becomes saturated over that period. The two groundwater tables can be represented as DTMs which will act as delineators between backfill material that becomes saturated and that which is already saturated or that which remains unsaturated during the settlement prediction period.

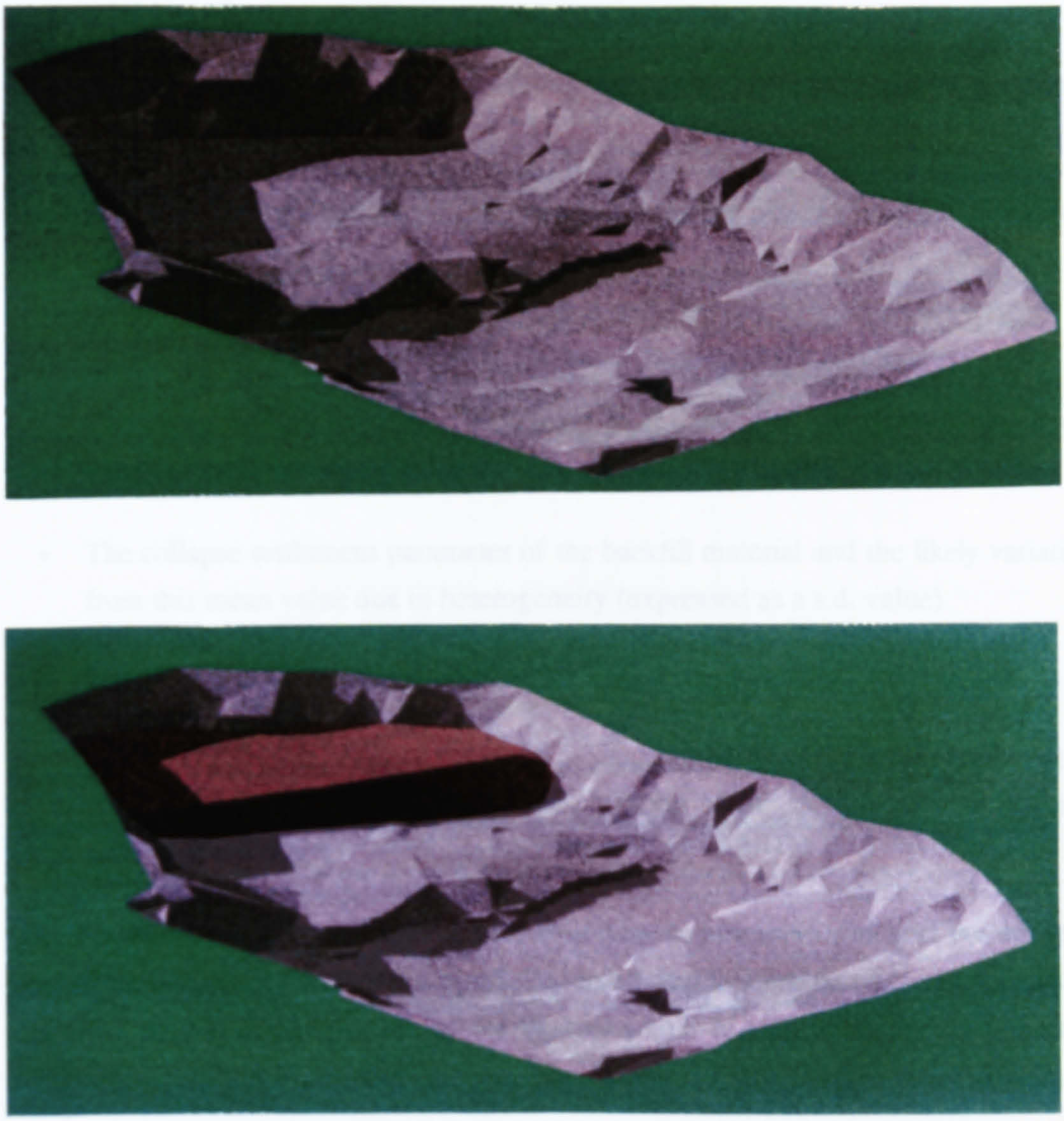


Figure 5.15. Newdale pit base plus hypothetical spoil block as represented by Digital Terrain Maps

This initial backfill modelling stage therefore produces a series of DTMs, one for the pit base, one each for the number of distinct blocks that the backfill consist of and pairs of DTMs representing groundwater levels at the start and end of a range of settlement prediction periods. The next stage is to assign to each backfill block values for the parameters that describe their settlement characteristics and fill properties. This is done by appending these values to the end of the SURPAC text files which describe the block DTMs. Parameters considered necessary to identify the blocks and adequately describe settlement characteristics and fill properties are as follows:

- The block number. Blocks are numbered sequentially in the order in which they were placed.

- The time from the start of backfilling operations the placement of the block commenced.

- The time from the start of backfilling operations the placement of the block was completed.

- The unit weight of the backfill material.

- The creep compression rate parameter of the backfill material and the likely variation from this mean value due to heterogeneity (expressed as a s.d. value).

- The collapse settlement parameter of the backfill material and the likely variation from this mean value due to heterogeneity (expressed as a s.d. value).

- The elevation of the lowest point on the base of the block.

- The maximum thickness of the block as determined from the difference between the base elevation (as above) and the elevation of the highest point on the top surface of the block.

The determination of suitable values for the creep compression rate and the collapse settlement parameters was as discussed above showing them to be primarily dependant upon the degree of compaction of the backfill which in turn is dependant upon the methods of backfill placement and any post-constructional improvement techniques used.

Lastly the series of points that describe the plan perimeter of all of the DTMs is appended and the files are converted into binary form to speed up subsequent processing. It is from this set of DTMs that settlement predictions are made. OBSett interprets these DTMs enabling absolute settlements to be predicted for a point, a section, an area, the whole site or differential settlement between any points for any chosen settlement prediction period.

5.7.2 DTM Interpretation

To predict the settlement beneath a given point the settlement characteristics of the column of backfill beneath that point have to be determined. This is achieved firstly by defining the column of backfill which will consist of a series of distinct column blocks and secondly as with the main backfill blocks, assigning appropriate settlement characteristics to these column blocks.

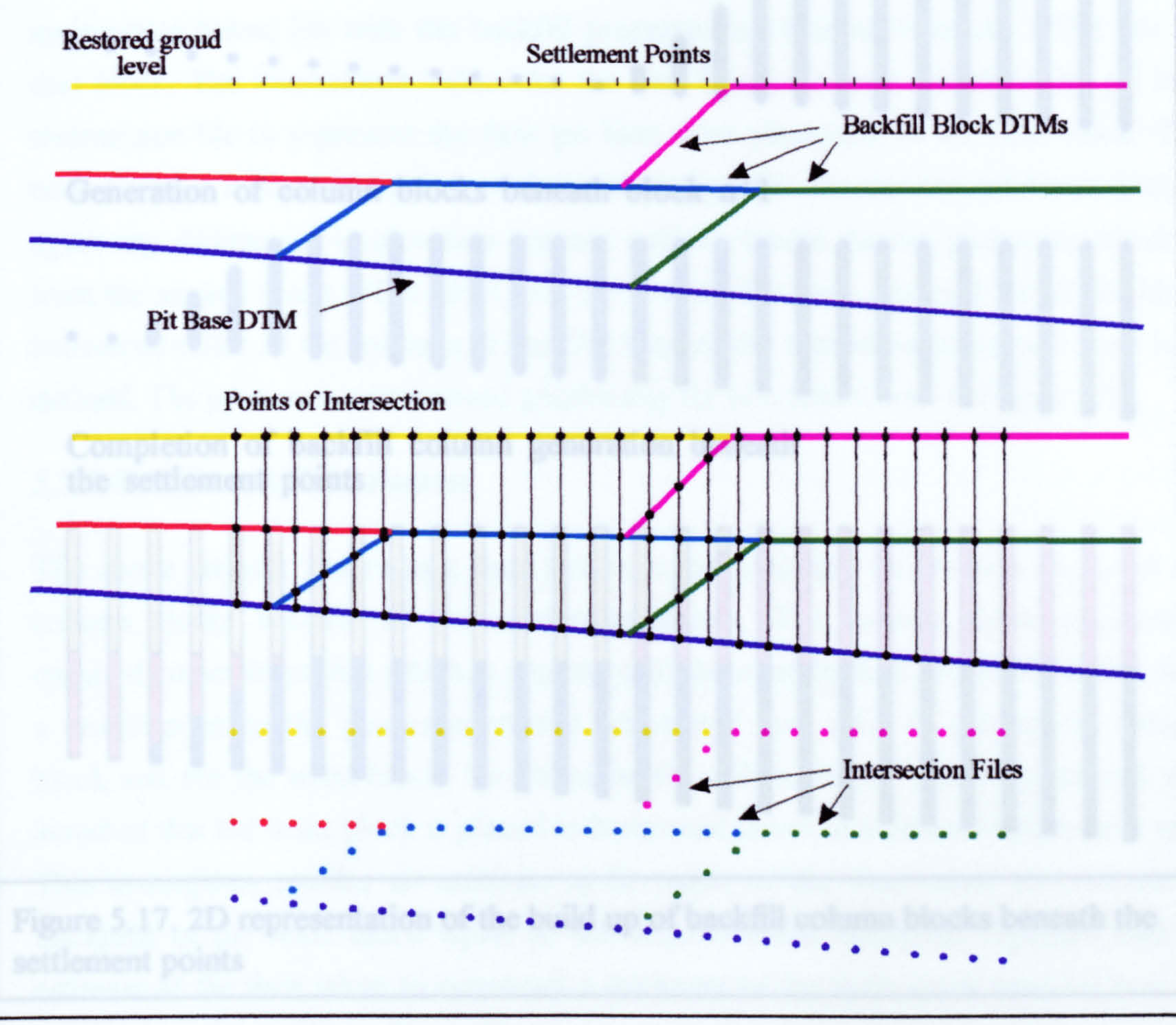


Figure 5.16. 2D representation of the lines of intersection between vertical lines beneath settlement points and the pit base and spoil block DTMs

The column of backfill beneath a settlement point can be defined by the points of intersection a vertical line drawn down through the backfill beneath the settlement point makes with the pit base DTM and any block DTMs underlying the point (Figure 5.16). As DTMs are made up of a series of triangles the problem of determining intersections beneath settlement points is one of determining which triangle within a given DTM the point directly lies above and then determining the point of intersection within that triangle. By comparing the easting and northing value of the settlement point in question with the easting and northing values for the vertices of each of the triangles for a given DTM the triangle that contains the settlement point in plan view can easily be determined. The elevation of the point of intersection can then be determined from a simple "point in a triangle algorithm".

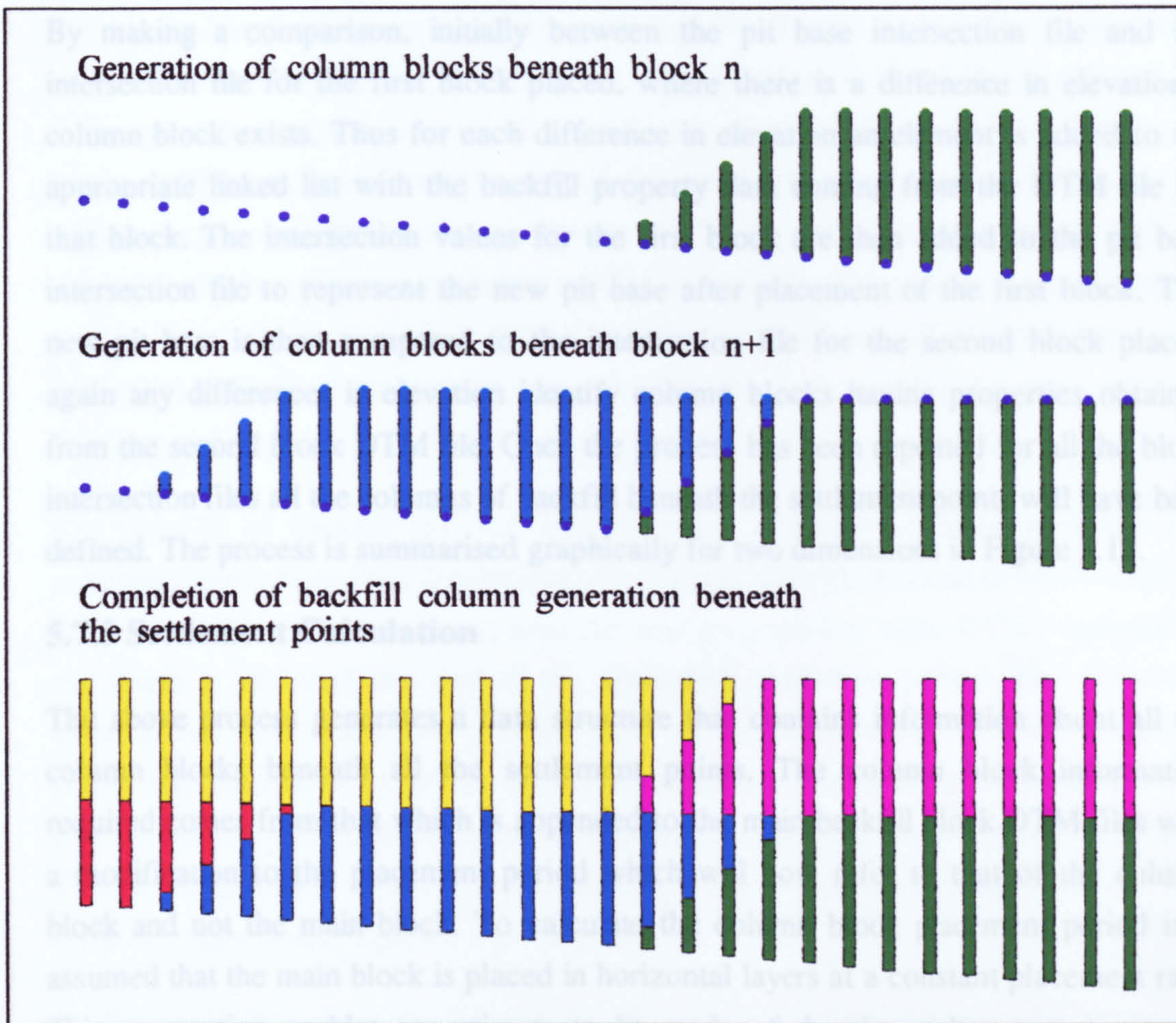


Figure 5.17. 2D representation of the build up of backfill column blocks beneath the settlement points

The above process of intersection determination produces a new set of binary files one for each of the block DTMs which lie beneath settlement points and one for the pit base DTM. Each file contains the easting, northing and elevation values of the intersections between the settlement points and the corresponding DTM. The process of generating these intersection files is represented in Figure 5.17 which shows the problem in two dimensions to assist visualisation.

This second set of binary files (the intersection files) then enable the columns of backfill material beneath the settlement points to be built up and defined. The programming technique used for this stage is that of generating an array of linked lists, whereby each element in the array represents a settlement point and each element in the linked lists represents one of the column blocks making up the column of backfill material beneath a given settlement point. The data stored in each element of the linked list is that which is necessary to calculate the settlement for that block.

By making a comparison, initially between the pit base intersection file and the intersection file for the first block placed, where there is a difference in elevation a column block exists. Thus for each difference in elevation an element is added to the appropriate linked list with the backfill property data coming from the DTM file for that block. The intersection values for the first block are then added to the pit base intersection file to represent the new pit base after placement of the first block. This new pit base is then compared to the intersection file for the second block placed; again any differences in elevation identify column blocks having properties obtained from the second block DTM file. Once the process has been repeated for all the block intersection files all the columns of backfill beneath the settlement points will have been defined. The process is summarised graphically for two dimensions in Figure 5.17.

5.7.3 Settlement Calculation

The above process generates a data structure that contains information about all the column blocks beneath all the settlement points. The column block information required comes from that which is appended to the main backfill block DTM files with a modification to the placement period which will now refer to that of the column block and not the main block. To calculate the column block placement period it is assumed that the main block is placed in horizontal layers at a constant placement rate. This assumption enables an estimate to be made of the time taken to construct a thickness of the main block equal to the column block thickness, together with an estimate of the time taken to construct a thickness of the main block from its base up to the base of the column block. These two estimates then enable the time period over which the column block was placed to be defined. Thus we have an estimate of the time from the start of the backfilling operation the placement of the column block commenced and the time from the start of backfilling operations the placement of the column block was completed.

All the information required to calculate the creep settlement beneath a given point is now available. Creep settlement is calculated by the methods discussed above for individual layers within the column of backfill with the creep settlement for the whole column being the accumulated settlement of all the layers. The column is split up into layers to provide a greater accuracy to the calculated settlement. The influence of the layer thickness on settlement predictions can be seen in the following Chapter, but can, as a rough estimate, be set equal to the approximate placement layer thickness.

To calculate any collapse settlement that is likely to occur we need to determine the amount of backfill that becomes saturated. This will be obtained from the prediction of the rate and level of the groundwater table rise. As discussed above the groundwater table will be represented in the form of a DTM, therefore to represent a predicted rise in the groundwater table a series of groundwater table DTMs are needed showing the levels at different times from the end of the compaction operation. To calculate the collapse settlement the chosen settlement prediction period must coincide with the times for which a groundwater table a DTM is available. If it can be assumed that the groundwater table will reach a stable level after a certain period of time which is the general case, the settlement prediction period can be open ended if it continues beyond this time and any chosen period if it starts after this time.

The amount of backfill material that becomes saturated is determined for each settlement point from the intersections of a vertical line extrapolated down through the fill beneath the settlement point, with the two groundwater table DTMs representing the levels at the start and end of the settlement prediction period. The process of determining these intersections is the same as for intersections between settlement points and backfill block DTMs as discussed above. The collapse settlement is then determined from the collapse settlement parameters for the saturated column blocks. This value is added to the creep settlement value to give an estimate of the total settlement that occurs beneath a given point.

Finally the influence of the settlement of surrounding material must be taken into consideration. This calculation requires the settlement values of points within the area of influence. In the case where the chosen settlement points cover the whole site in sufficient density, the settlement adjustment can be carried out using settlement values that have already been calculated for each of the settlement points, thus speeding up processing. In the case where settlement has been chosen for a point section, area or for the whole site in insufficient density, new settlement calculations will be required for the points of influence surrounding any one settlement point. In both cases the area of influence is determined from the fill thickness and the angle of influence. The density of the points of influence within this area used for the adjustment will be pre-defined by the settlement point density in the former case and by a user defined value in the later case.

An option is also available for calculating the differential settlement between two points due to the heterogeneous nature of the backfill as defined by the s.d. values for the creep compression rate and collapse strain parameters. This option generates a population of possible differential settlements as described above.

The influence of surcharging is incorporated into the program by making an adjustment to the creep compression rate and collapse strain parameter, as outlined in the previous chapter, backfill material lying beneath a surcharge mound. This adjustment will be significantly dependant upon the surcharge constant and the influence this constant has on the predicted settlement of surcharged material is examined in the following chapter. The surcharge mound will be defined by a DTM that encloses its surface to enable the height of surcharge above any point on the restored surface to be easily determined.

5.8 Conclusion

The prediction of opencast backfill settlement is an important consideration in the development of such sites as discussed above. This chapter has outlined the methods proposed by the author by which settlement predictions are to be made and how these have been implemented into a computer program.

The proposed method considers both the long term creep component of backfill settlement, taking into consideration the affects of time upon creep movements, together with the more rapid collapse settlement that occurs upon saturation. The settlement at a given point has not been taken in isolation and a method of accounting for the influence of surrounding backfill material has been devised. A means of assessing the affect backfill heterogeneity has on differential settlements has also been included.

The proposed method is largely empirical being based upon the observations made from an extensive literature search and the study sites as discussed in chapter 2 and 3. As a consequence of this a number of undefined parameters have been used within the settlement prediction process. To determine values for these parameters, their influence on predicted settlement is examined such that reasonable values can be chosen in the light of presently available literature and study site data. This is carried out in the first part of the following chapter. Also carried out in the following chapter, as a means of validation and program testing, is an examination of how the prediction method responds to changes in the backfilling scenario being examined.

PROGRAM CALIBRATION AND TESTING

6.1 Introduction

The process of settlement calculation described in the previous chapter requires values for a range of parameters which can be spilt into two categories. Those that are used for the internal workings of the program and those that describe the problem to be solved. This chapter examine the influence these parameters have on settlement predictions with the former category being examined under the heading Program Calibration and the latter under the heading Program Testing. In both cases the program will be tested on an example site based upon that shown in Figure 6.1.

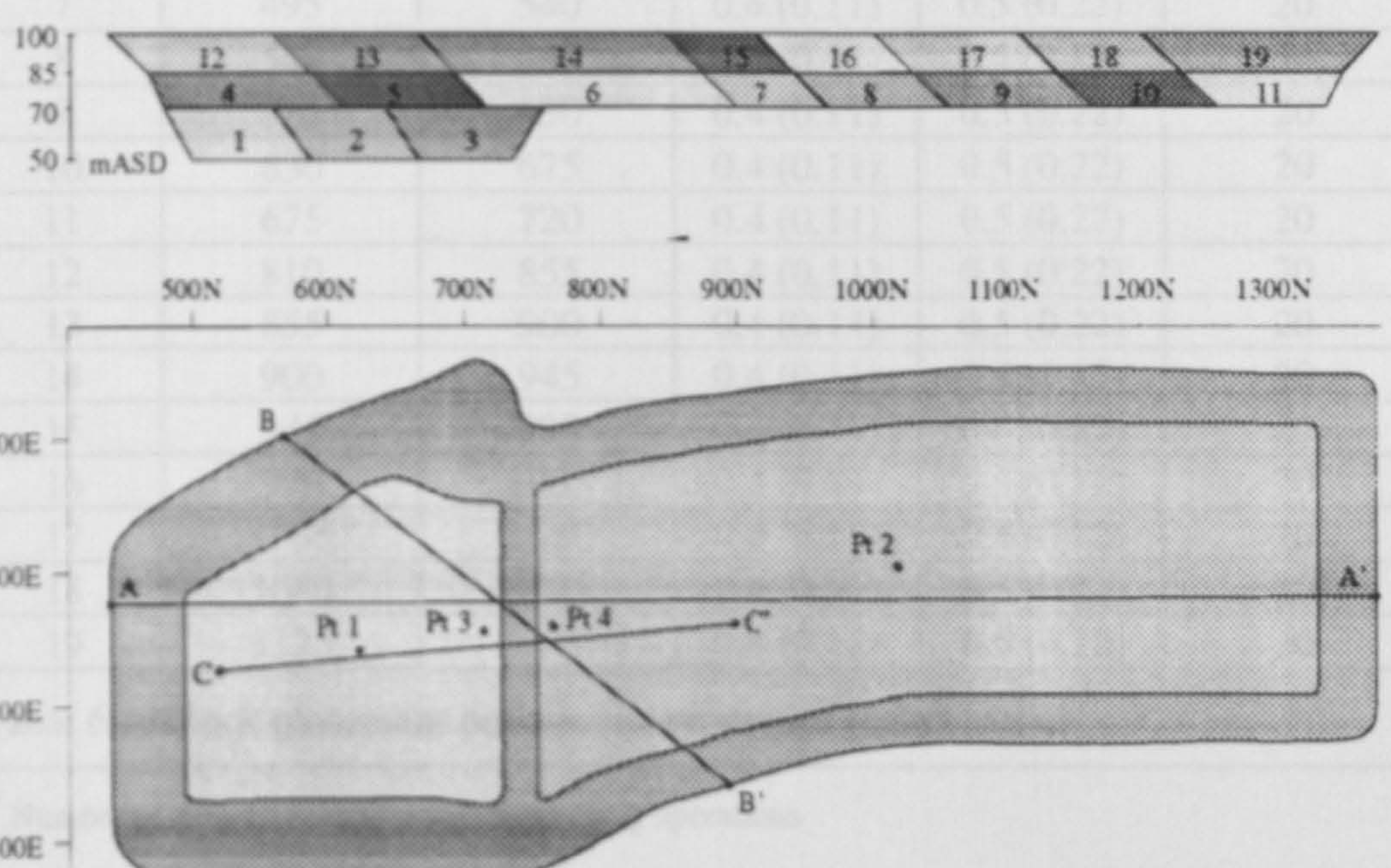


Figure 6.1. Example site.

6.2 Program Calibration

Parameters required for the internal workings of the program are as follows:

- The percentage stress increase 'Cut-Off' value.
- The thickness of the layers the backfill is split up into during the settlement calculation.

- The density of the points of influence (POI) used for the correction of settlement at a point dependant upon the settlement of surrounding material.

- The number of simulations carried out during the calculation of the differential settlement between two points, to get a representative sample population.

Block	Period of Placement (days) *		Alpha (S.D.)	Collapse (S.D.)	Unit Weight (kN/m³)
1	0	90	0.4 (0.11)	0.5 (0.22)	20
2	90	180	0.4 (0.11)	0.5 (0.22)	20
3	180	270	0.4 (0.11)	0.5 (0.22)	20
4	360	405	0.4 (0.11)	0.5 (0.22)	20
5	405	450	0.4 (0.11)	0.5 (0.22)	20
6	450	495	0.4 (0.11)	0.5 (0.22)	20
7	495	540	0.4 (0.11)	0.5 (0.22)	20
8	540	585	0.4 (0.11)	0.5 (0.22)	20
9	585	630	0.4 (0.11)	0.5 (0.22)	20
10	630	675	0.4 (0.11)	0.5 (0.22)	20
11	675	720	0.4 (0.11)	0.5 (0.22)	20
12	810	855	0.4 (0.11)	0.5 (0.22)	20
13	855	900	0.4 (0.11)	0.5 (0.22)	20
14	900	945	0.4 (0.11)	0.5 (0.22)	20
15	945	990	0.4 (0.11)	0.5 (0.22)	20
16	990	1035	0.4 (0.11)	0.5 (0.22)	20
17	1035	1080	0.4 (0.11)	0.5 (0.22)	20
18	1080	1125	0.4 (0.11)	0.5 (0.22)	20
19	1125	1170	0.4 (0.11)	0.5 (0.22)	20

Table 6.1. Block placement periods and properties for scenario 1.

* Number of days from the start of backfilling operations

6.2.1 The percentage stress increase 'Cut-Off' value

The significance of the percentage stress increase 'cut-off' value (x) is in determining the time origin at which creep settlement is considered to have commenced (t_0). It is felt that within backfill material placed at depth, creep settlement will have begun within it some time before the completion of the total thickness of backfill has been placed above it. Thus the time at which creep settlement commences (t_0) for any layer within the backfill will lie between the time at which the placement of the layer was completed and the time at which the top most material within the backfill above the layer was completed. This is discussed in the previous chapter and illustrated in Figure 5.9 which shows that it is the percentage stress 'cut-off' value (x) that defines t_0 together with the relationship between the effective vertical stress within the layer and time. As x increases t_0 approaches the time at which a given layer was placed, as x decreases t_0 approaches the time at which the total thickness of backfill above that layer was placed. The significance of x is therefore greatest in the situation where the backfilling operation is such that a considerable delay occurs between the placement of blocks in the vertical.

In this example the influence of x on predicted settlement will be demonstrated by two backfilling scenarios, one where the timing of block placement and block properties is as in Table 6.1 and the other where the timing and properties is as in Table 6.2. Test conditions are summarised in Table 6.3. Settlements will be calculated along the section A - A', Figure 6.1, at an interval of every 5 metres.

Block	Period of Placement (days) *		Alpha (S.D.)	Collapse (S.D.)	Unit Weight (kN/m³)
1	0	90	0.4 (0.11)	0.5 (0.22)	20
2	90	180	0.4 (0.11)	0.5 (0.22)	20
3	180	270	0.4 (0.11)	0.5 (0.22)	20
4	270	315	0.4 (0.11)	0.5 (0.22)	20
5	360	405	0.4 (0.11)	0.5 (0.22)	20
6	450	495	0.4 (0.11)	0.5 (0.22)	20
7	540	585	0.4 (0.11)	0.5 (0.22)	20
8	630	675	0.4 (0.11)	0.5 (0.22)	20
9	720	765	0.4 (0.11)	0.5 (0.22)	20
10	810	855	0.4 (0.11)	0.5 (0.22)	20
11	900	945	0.4 (0.11)	0.5 (0.22)	20
12	315	360	0.4 (0.11)	0.5 (0.22)	20
13	405	450	0.4 (0.11)	0.5 (0.22)	20
14	495	540	0.4 (0.11)	0.5 (0.22)	20
15	585	630	0.4 (0.11)	0.5 (0.22)	20
16	675	720	0.4 (0.11)	0.5 (0.22)	20
17	765	810	0.4 (0.11)	0.5 (0.22)	20
18	855	900	0.4 (0.11)	0.5 (0.22)	20
19	945	990	0.4 (0.11)	0.5 (0.22)	20

Table 6.2. Block placement periods and properties for scenario 2.

* Number of days from the start of backfilling operations

The results from the two scenarios for a range of x from 1 to 1000 are shown in Figures 6.2 and 6.3. This confirms that the significance of x is greater in the case where there is a considerable delay between the placement of blocks in the vertical as in scenario 1.

With these two scenarios it is possible to demonstrate how x influences the predicted settlement but without more detailed information on the timing of backfilling operations from the study sites it is difficult to set the value of x based upon observed settlements. A value that gives predicted settlements lying in the approximate middle of the range of possible settlements for the range of x examined will therefore be chosen. A value of 100 is considered appropriate and is set as the default; Thus the time origin at which creep settlement is considered to have commenced (t_0) will equal the time at which half the maximum effective vertical stress is attained due to the placement of material above a given point within the backfill. This value will be set as the default for subsequent calculations.

Percentage Stress 'Cut-Off' value(%)	Layer Thickness (mm)	POI Density	Number of Simulations
1 to 1000	50	4	N/A

Prediction Period*		Groundwater Table		Surcharge	
From	To	Rise (m)	Period (days)	Height (m)	Location
30	9155	Not considered		Not considered	

Table 6.3. Test conditions for testing the percentage stress 'cut-off' value.

* Number of days from the completion of backfilling operations

6.2.2 Layer Thickness

Creep settlement is calculated for individual layers making up the column of backfill beneath a given point for which settlement prediction is required. The layers have a thickness as defined by the value given to the layer thickness parameter. It is found that this value is inversely proportional to processing time. Thus to speed up processing, layer thickness can be increased. This does however have a detrimental effect in that anomalous settlements are predicted for values of 1000mm and greater. This is demonstrated by Figures 6.4 to 6.10 in which settlements have been calculated along the section A-A' at a 5m interval for a backfill as described above by scenario 1. Test conditions are as in Table 6.4.

The anomalous results obtained for values of layer thickness of 1000mm and greater are largely due to the value of t_0 assigned to a given layer. t_0 is calculated at the mid point of a given layer thus for very thick layers the transition of the value of t_0 between successive layers is much greater than for thin layers. Thus for thick layers a greater

Figure 6.2. The influence of the Percentage Stress 'Cut-Off' Value (1 to 1000%) on the predicted settlement along section A-A' for Scenario I

Figure 6.3. The influence of the Percentage Stress 'Cut-Off Value (1 to 1000%) on the predicted settlement along section A-A' for scenario 2

proportion of the backfill has an inappropriate t_0 value during creep settlement calculations.

<table>
<tr><td colspan="3">Percentage Stress 'Cut-Off' value(%)</td><td colspan="3">Layer Thickness (mm)</td><td colspan="3">POI Density</td><td colspan="3">Number of Simulations</td></tr>
<tr><td colspan="3">50, 100, 200, 500</td><td colspan="3">100 to 10,000</td><td colspan="3">Not considered</td><td colspan="3">N/A</td></tr>
<tr><td colspan="4">Prediction Period*</td><td colspan="4">Groundwater Table</td><td colspan="4">Surcharge</td></tr>
<tr><td colspan="2">From</td><td colspan="2">To</td><td colspan="2">Rise (m)</td><td colspan="2">Period (days)</td><td colspan="2">Height (m)</td><td colspan="2">Location</td></tr>
<tr><td colspan="2">30</td><td colspan="2">9155</td><td colspan="4">Not considered</td><td colspan="4">Not considered</td></tr>
<tr><td colspan="12">Table 6.4. Test conditions for testing the layer thickness value.</td></tr>
</table>

* Number of days from the completion of backfilling operations

The influence layer thickness has on the settlement calculation is also found to be dependant upon the way in which t_0 is calculated i.e. the value of the percentage stress increase 'cut-off' parameter. This is also shown in Figures 6.4 to 6.10 which show that for a value of the percentage stress increase 'cut-off' parameter of 100, the layer thickness has a minimal influence whereas for 50, 200 and 500, for layer thickness values of 1000 mm and greater, anomalous results are obtained.

The value of the layer thickness parameter has an influence on the accuracy of the settlement prediction method employed. The smaller the value chosen the greater the accuracy. The results from the tests carried out indicate however that for layer thicknesses of less than 500mm there is little improvement in accuracy and as layer thickness is inversely proportional to processing time a value of 500mm is considered an appropriate default for the layer thickness.

6.2.3 The density of the points of influence (POI)

In calculating the settlement at a given point, it is considered appropriate that the settlement of surrounding backfill is taken into consideration. This is achieved by adjusting the settlement at any given point by the settlement of surrounding points as discussed in the previous chapter. The surrounding points are known as the points of influence (POI) and the number chosen during settlement calculation is defined by the POI density value.

The influence of the POI density on the prediction of settlement is examined by calculating settlements along section B-B' at a 10m interval for three different

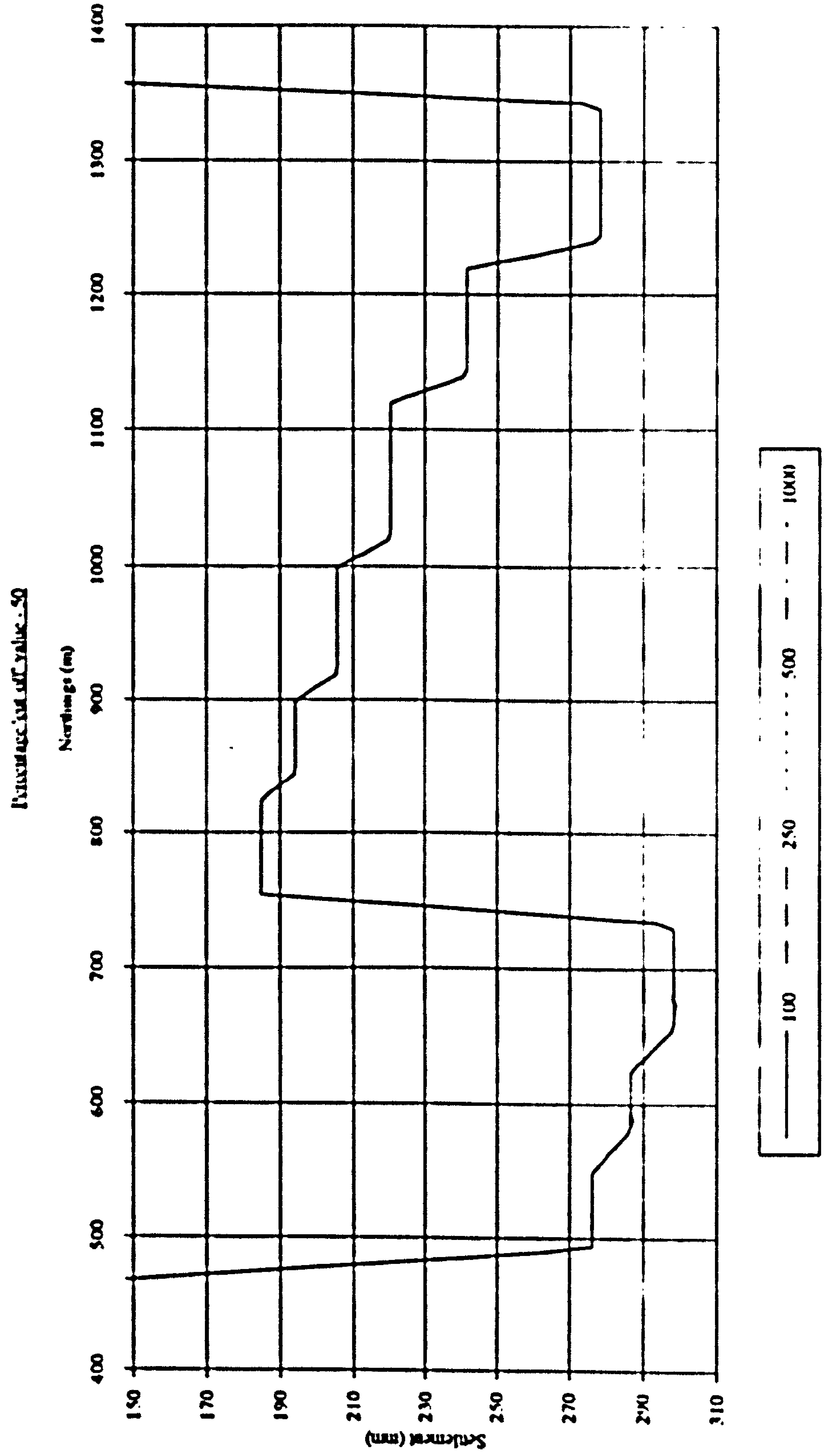


Figure 6.4. The influence of layer thickness (100 to 1000mm) on the predicted settlement along section A-A' for scenario 1.

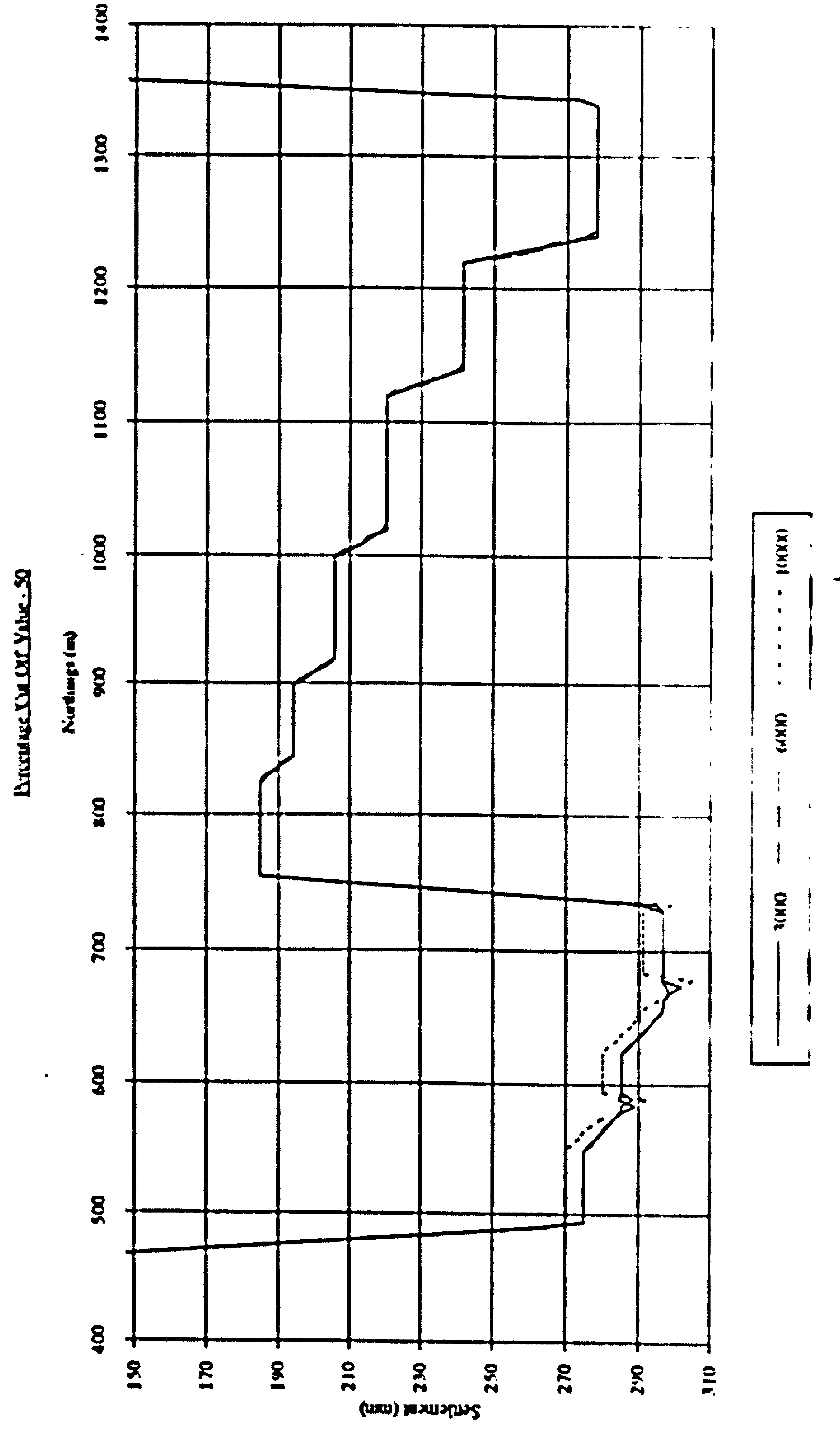


Figure 6.5. The influence of layer thickness (3,000 to 10,000mm) on the predicted settlement along section A-A' for scenario 1.

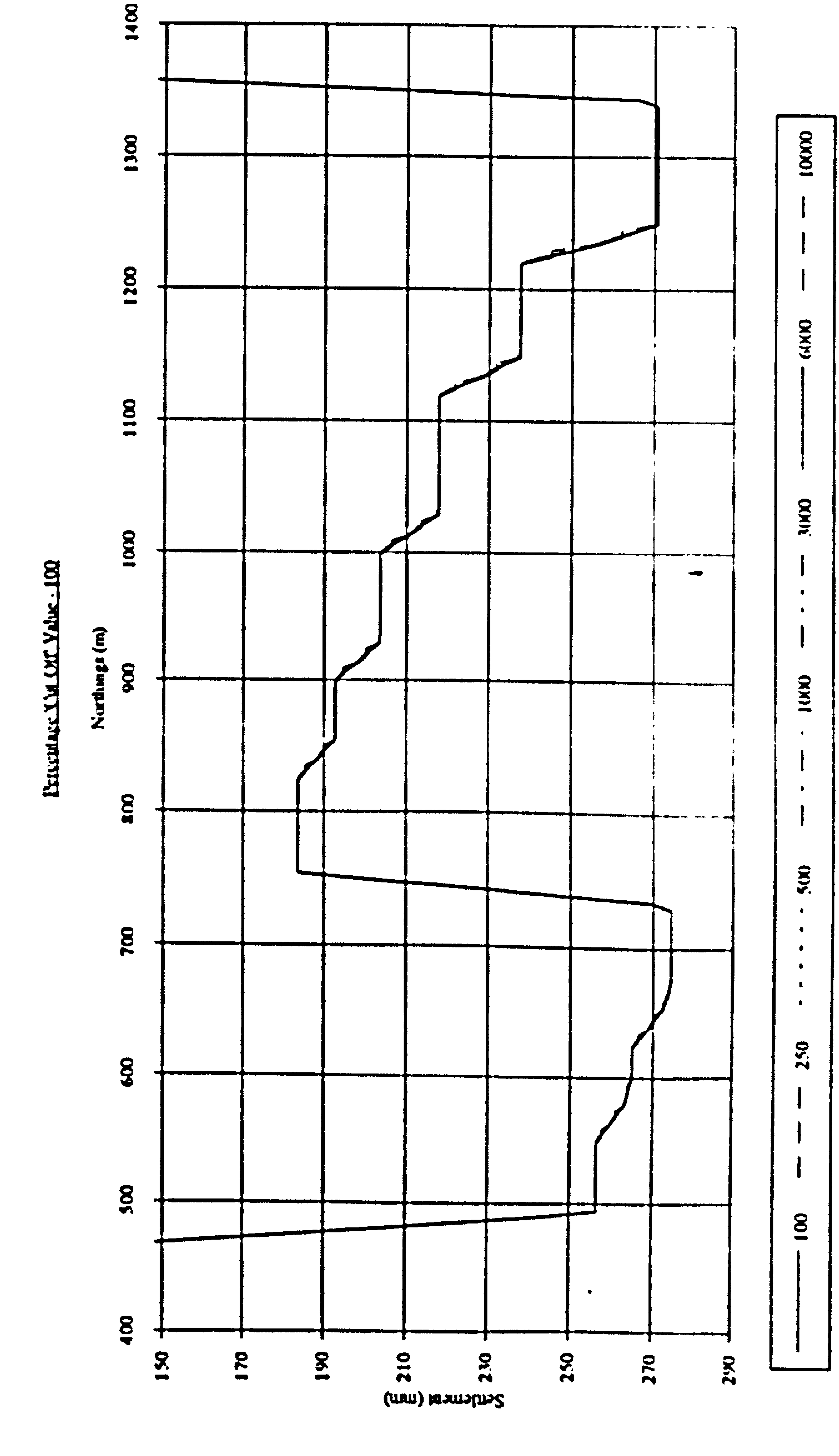


Figure 6.6. The influence of layer thickness (100 to 10,000mm) on the predicted settlement along section A-A' for scenario 1.

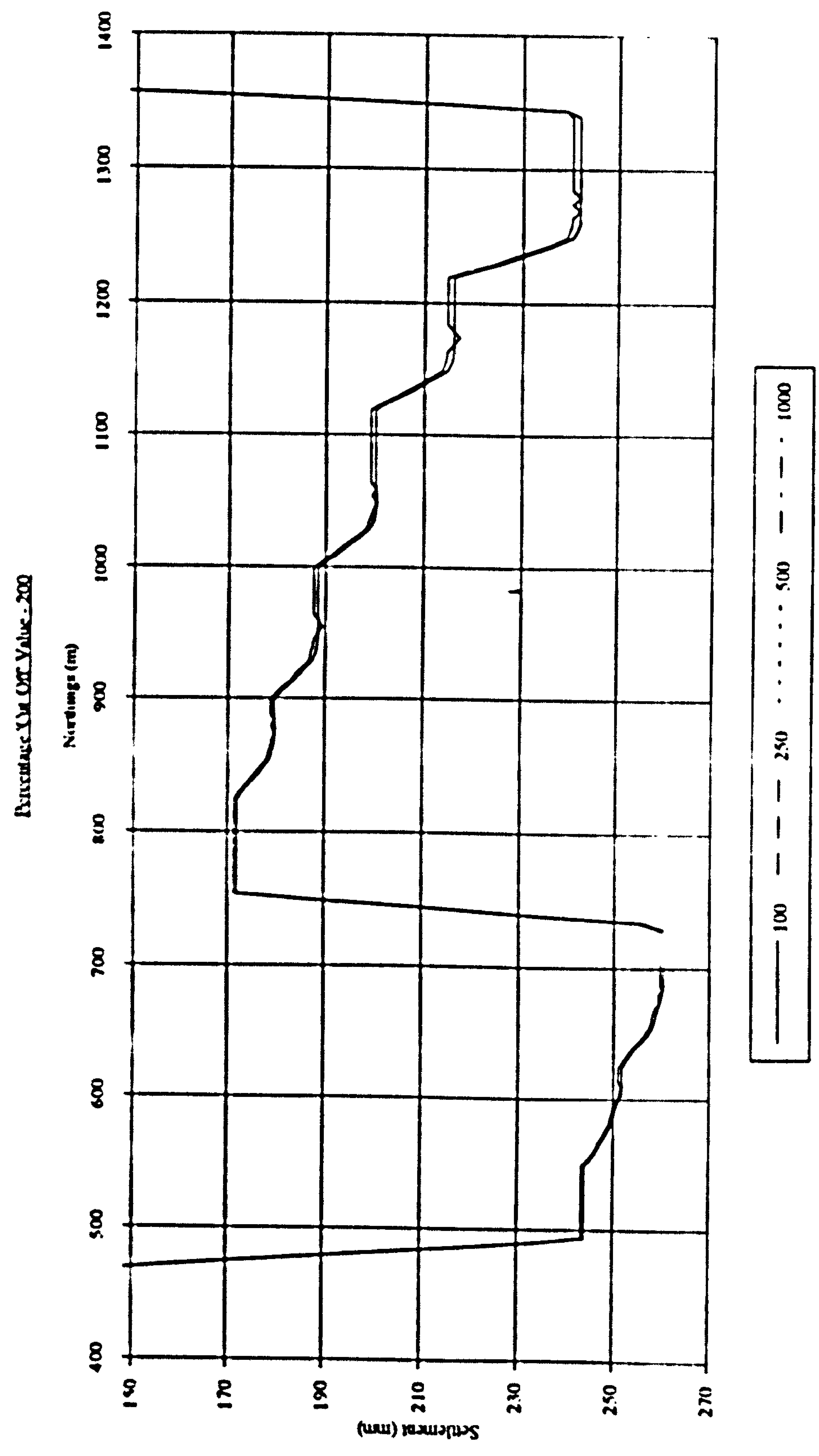


Figure 6.7. The influence of layer thickness (100 to 1,000mm) on the predicted settlement along section A-A' for scenario 1.

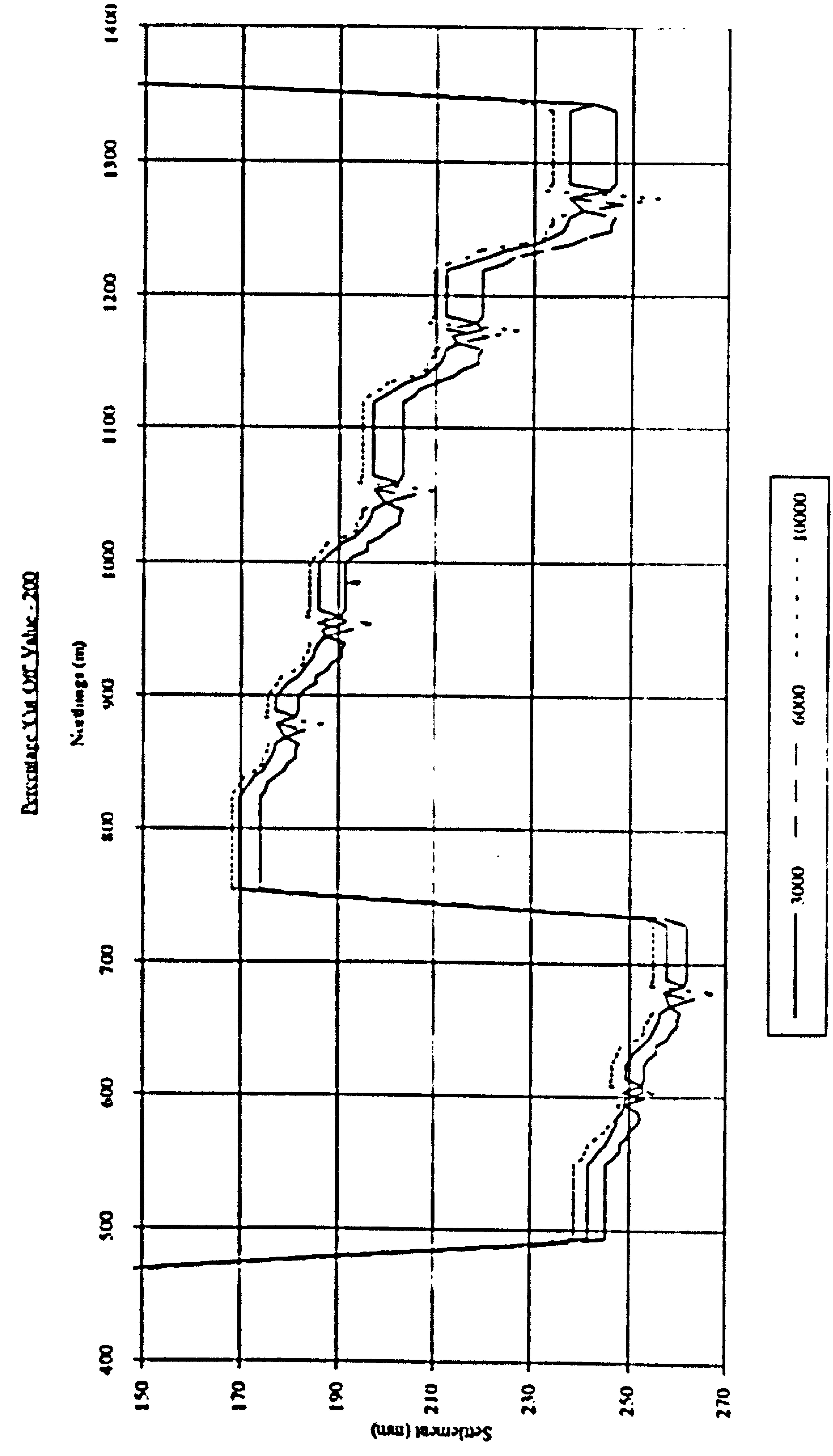


Figure 6.8. The influence of layer thickness (3,000 to 10,000mm) on the predicted settlement along section A-A' for scenario 1.

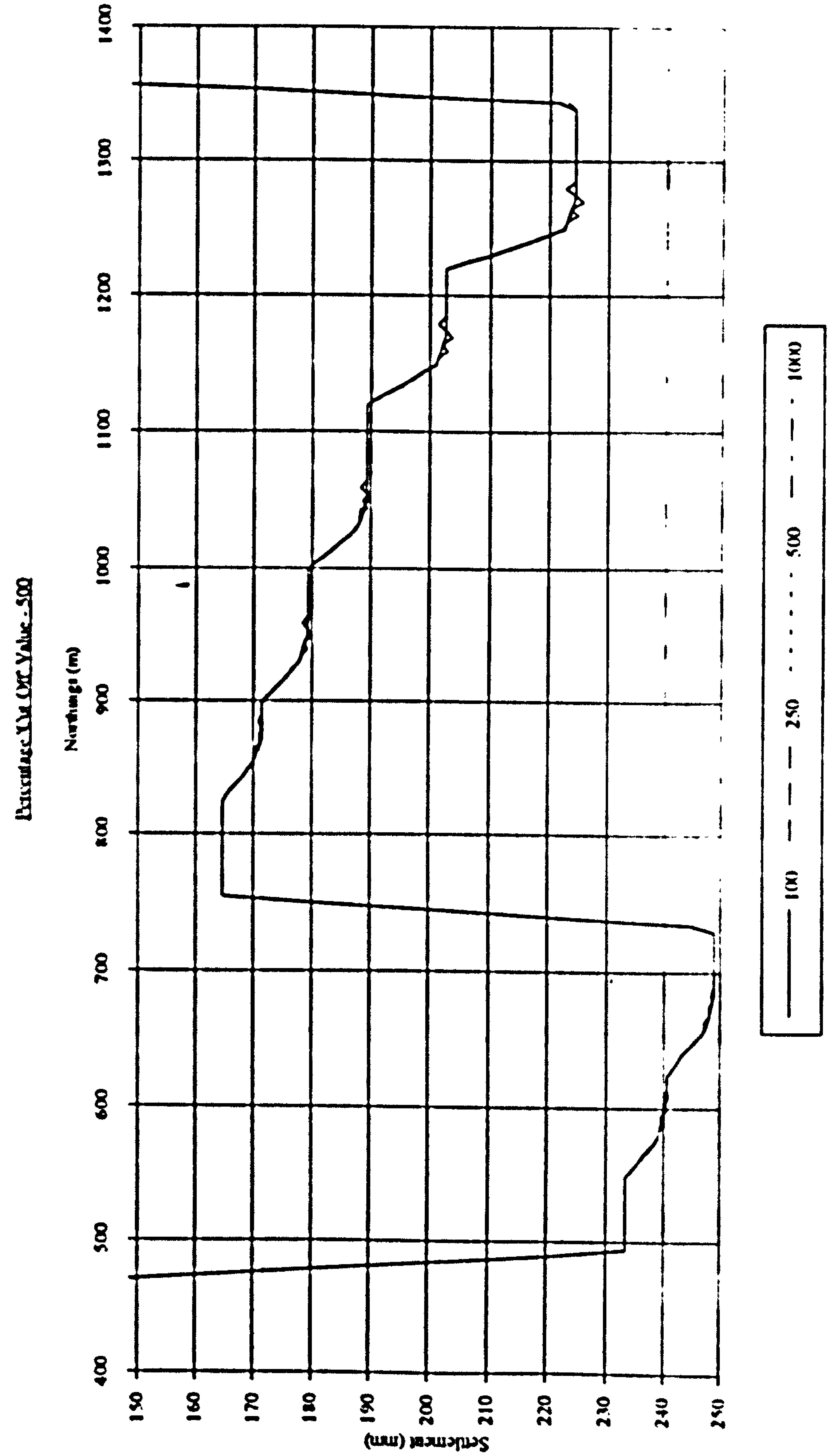


Figure 6.9. The influence of layer thickness (100 to 1,000mm) on the predicted settlement along section A-A' for scenario 1.

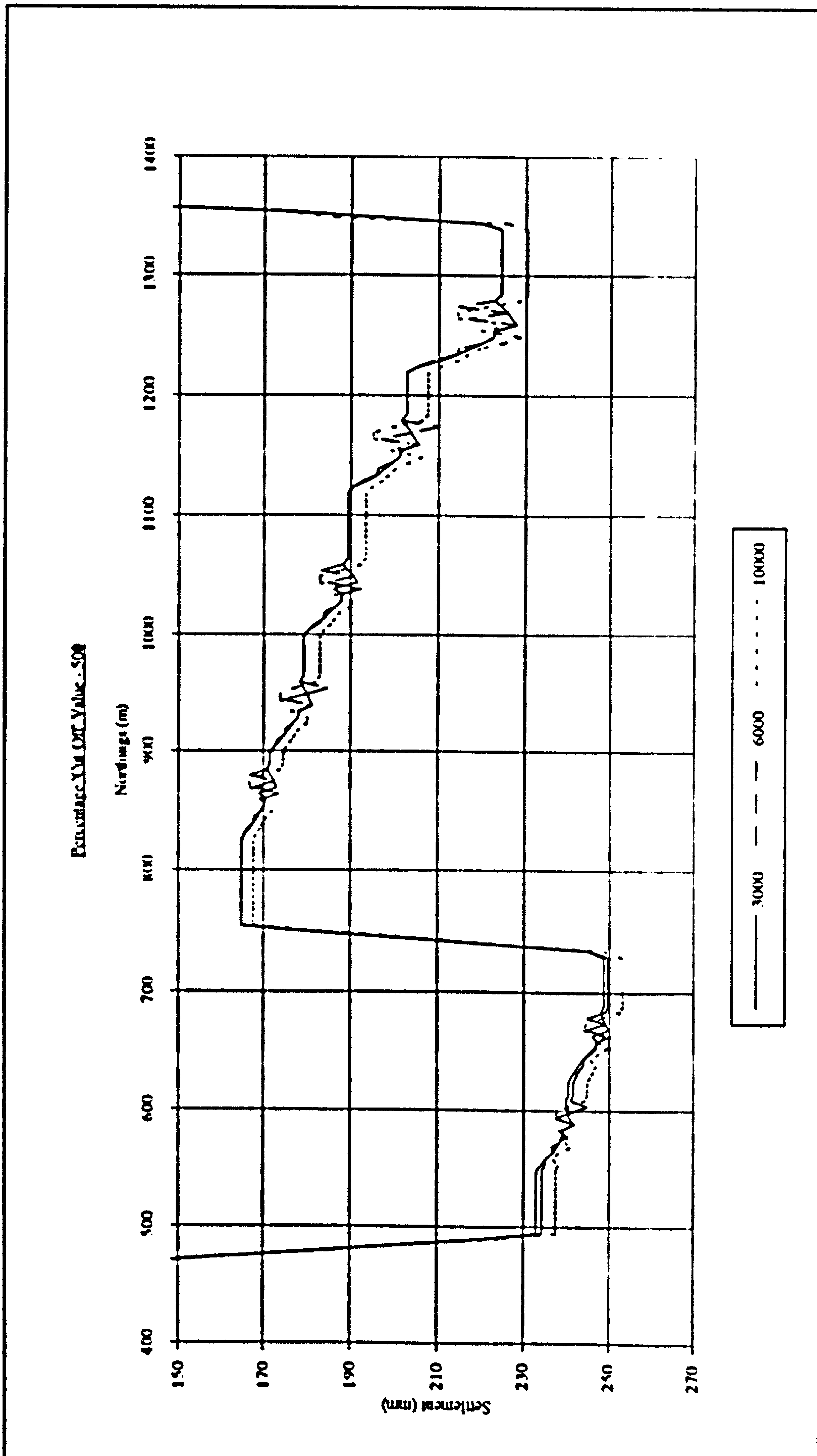


Figure 6.10. The influence of layer thickness (3,000 to 10,000mm) on the predicted settlement along section A-A' for scenario 1.

backfilling scenarios, 3, 4 and 5, as described in Tables 6.6 to 6.8. Test conditions are the same for each scenario, as summarised in Table 6.5 and the results are show in Figures 6.11 to 6.16.

Percentage Stress Cut-Off value(%)		Layer Thickness (mm)		POI Density	Number of Simulations	
Default (100)		Default (500)		1 to 9	N/A	
Prediction Period*		Groundwater Table			Surcharge	
From	To	Rise (m)	Period (days)		Height (m)	Location
30	9155	Not considered			Not considered	

Table 6.5. Test conditions for POI density testing.

* Number of days from the completion of backfilling operations

Block	Period of Placement (days)	Alpha (S.D.)	Collapse (S.D.)	Unit Weight (kN/m^3)
1 to 19	As scenario 2	0.15 (0.03)	0.25 (0.04)	20

Table 6.6. Block placement periods and properties for scenario 3.

Block	Period of Placement (days)	Alpha (S.D.)	Collapse (S.D.)	Unit Weight (kN/m^3)
1 to 19	As scenario 2	0.50 (0.15)	0.90 (0.28)	20

Table 6.7. Block placement periods and properties for scenario 4.

Block	Period of Placement (days)	Alpha (S.D.)	Collapse (S.D.)	Unit Weight (kN/m^3)
1 to 19	As scenario 2	0.80 (0.28)	1.20 (0.41)	20

Table 6.8. Block placement periods and properties for scenario 5.

The results show that for small POI density values irregular results are produced. As the POI value increases the accuracy of the adjustment also increases until a point is reached whereby any further increase in POI density gives very little further increased accuracy. The point at which minimal increased accuracy is obtained is different for the

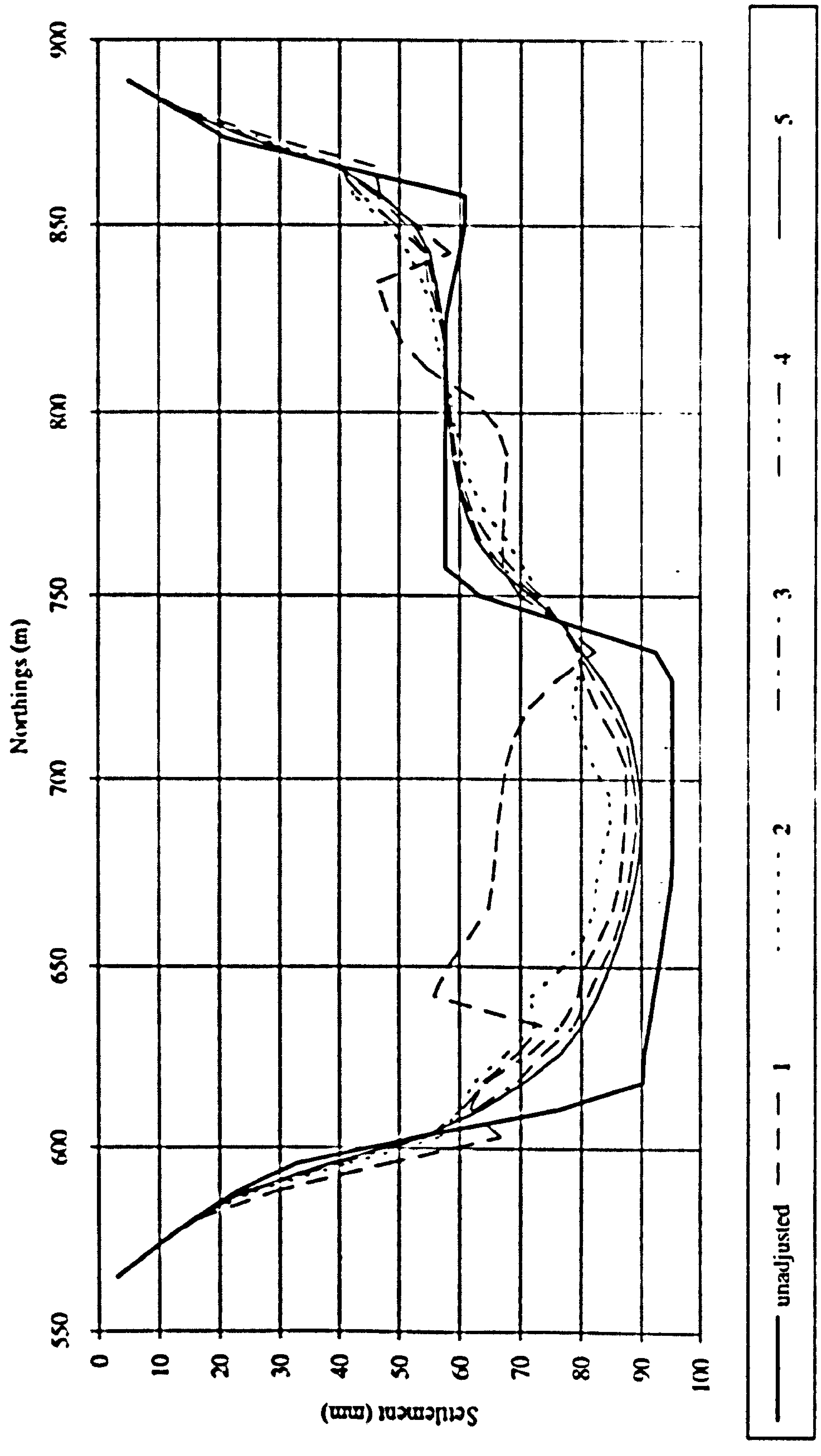


Figure 6.11. The influence of POI density (1 to 5) on the predicted settlement along section B-B' for scenario 3.

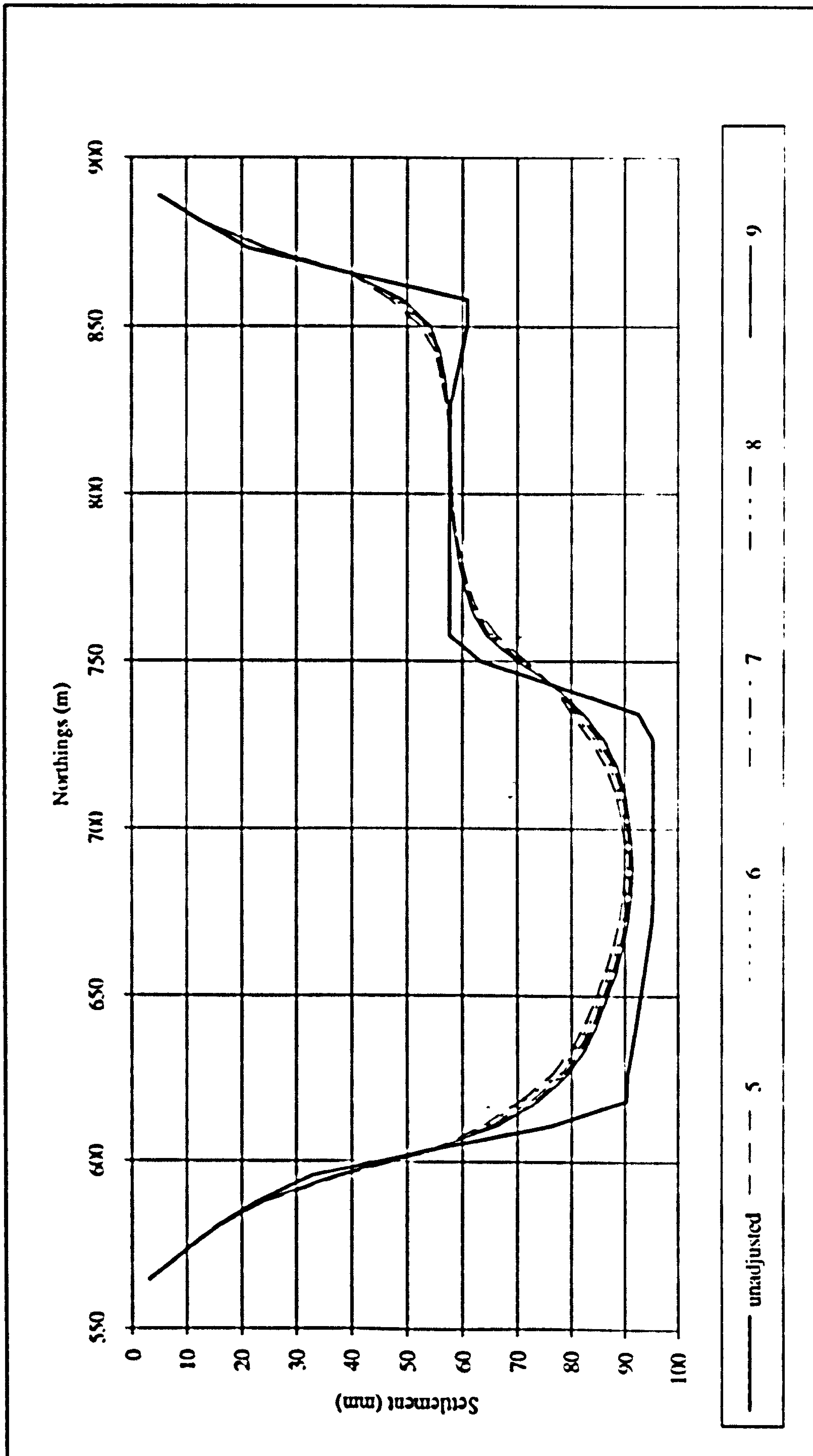


Figure 6.12. The influence of POI density (5 to 9) on the predicted settlement along section B-B' for scenario 3.

Figure 6.13. The influence of POI density (1 to 5) on the predicted settlement along section B-B' for scenario 4.

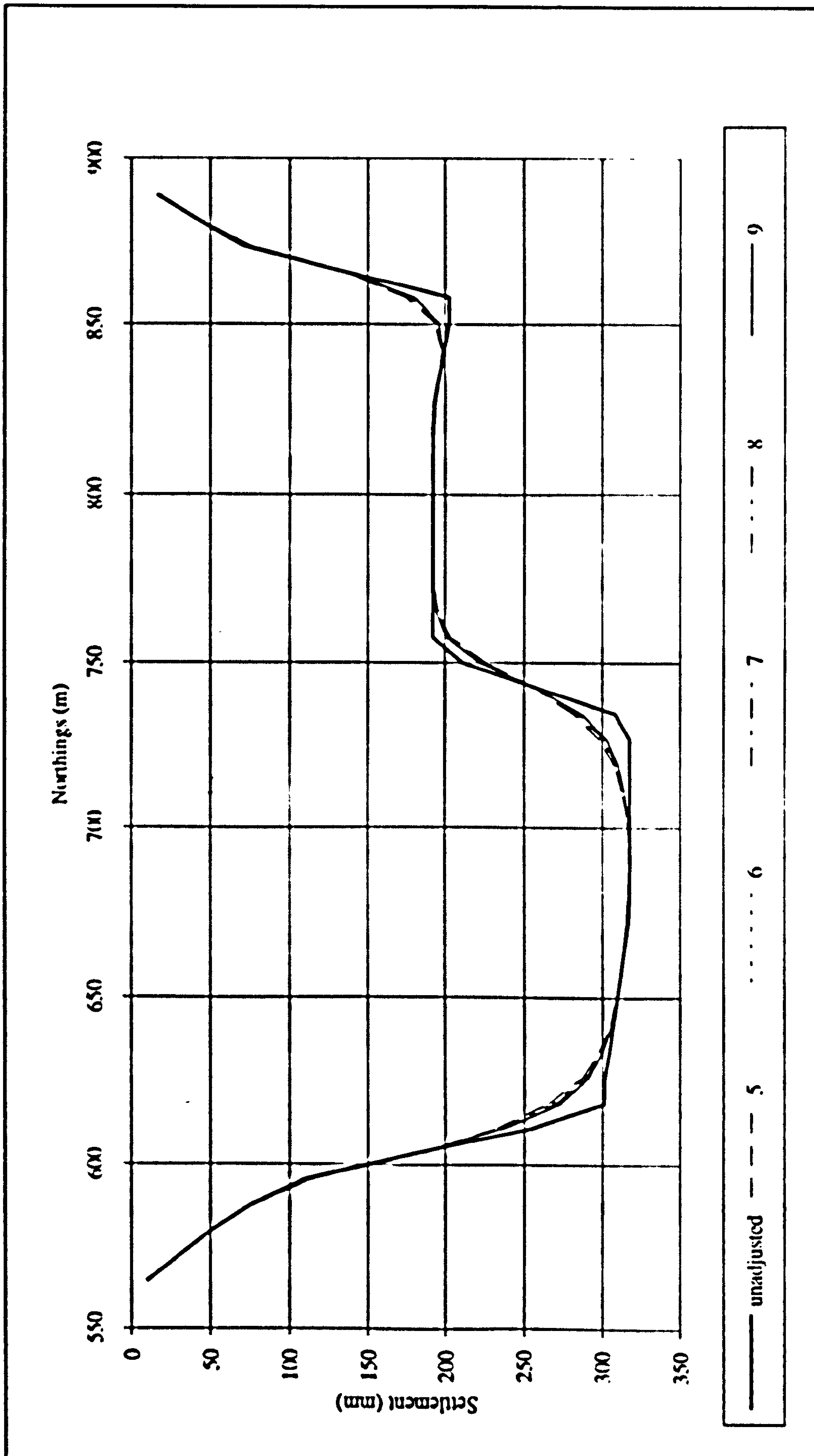


Figure 6.14. The influence of POI density (5 to 9) on the predicted settlement along section B-B' for scenario 4.

Figure 6.15. The influence of POI density (1 to 5) on the predicted settlement along section B-B' for scenario 5.

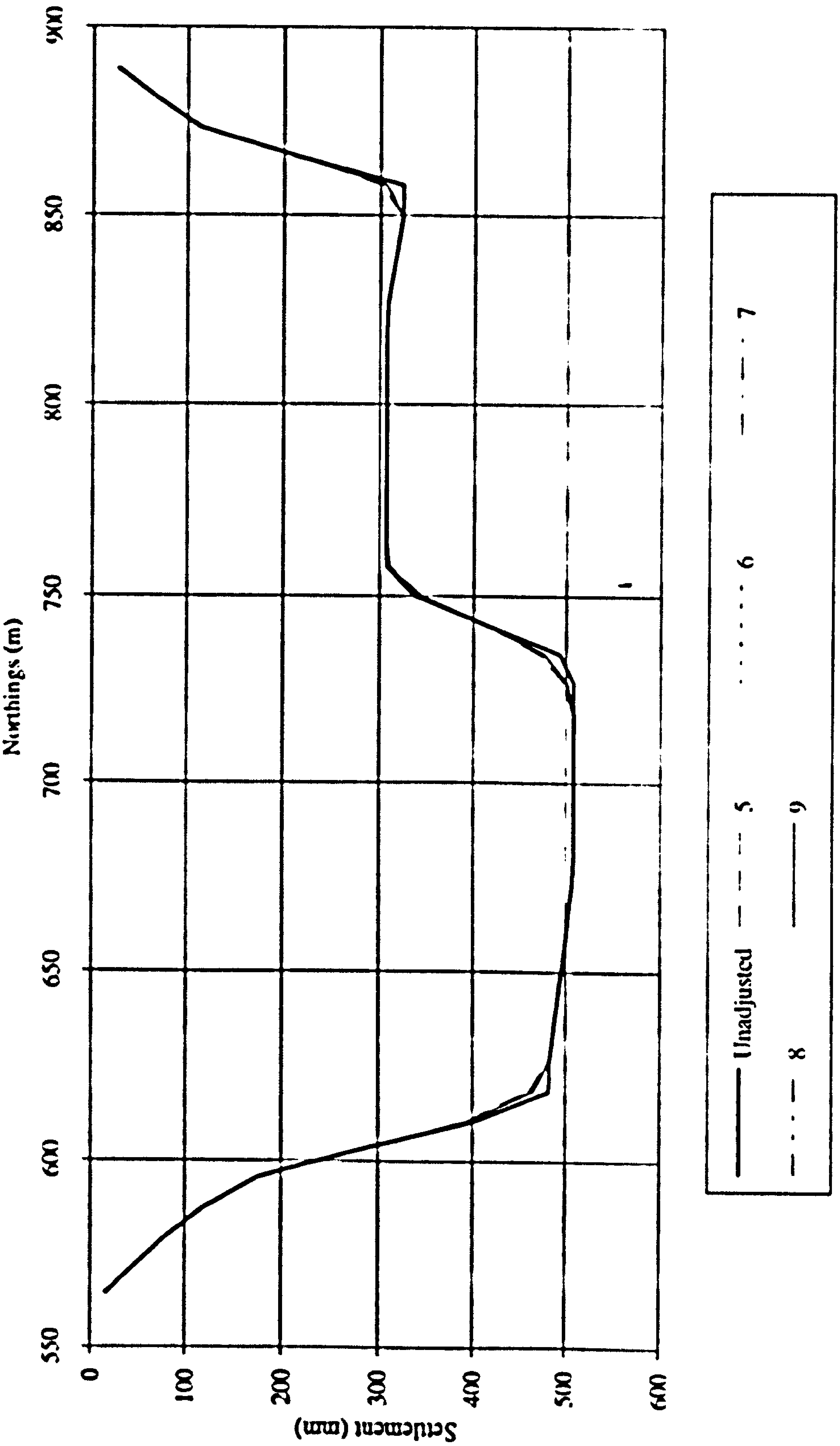


Figure 6.16. The influence of POI density (5 to 9) on the predicted settlement along section B-B' for scenario 5.

3 scenarios tested. For scenario 3 a value for the POI density of greater than 4 gives little improved accuracy, for scenario 4 a value of 3 is sufficient to give accurate results and for scenario 5 a value of 2 is appropriate. This is to be expected as to get an accurate adjustment for surrounding material the POI density must be such that a representative sample of surrounding material is obtained. The surrounding material that is of significance has been discussed in the previous chapter and is considered to be dependant upon the compactive state of the backfill as represented by alpha. The greater the compactive state the greater the area of influence. Thus for the highly compacted backfill of scenario 3, the area of influence is large thus a larger value for the POI density is required to get a representative sample than for the poorly compacted backfill of scenario 5.

In all cases examined it can be seen that very little increased accuracy is obtained for values of POI density greater than 4. It is therefore considered that 4 is an appropriate default value, however as the POI value has a significant influence on processing time (POI proportional to processing time), in the situation where the backfill is poorly compacted a POI value of less than 4 could be considered as processing would be speeded up with little loss in accuracy.

It is also of note from examination of the results that the magnitude of the adjustment due to surrounding material is proportional to the compactive state of the backfill. This is again as a result of the increased area of influence for more compact backfill, thus differential settlements are smaller over buried features for well compacted backfill than for poorly compacted backfill.

6.2.4 The number of differential settlement calculations

To determine the differential settlement between two points an absolute value is first determined using average values for both alpha and collapse with an adjustment for surrounding material. Then the variance from this average, due to backfill heterogeneity, is determined. This is done by repeatedly calculating the settlement at the two points of interest, with alpha and collapse values being taken randomly from a population of likely values, thus generating two samples of settlement values for each of the points. From these samples the variance of settlement at the two points can be calculated which in-turn enables the variance of the difference in settlement to be calculated.

As an example of how to determine a suitable number of repetitions, n, to get a representative sample of settlement values at a given point, the settlement can be calculated n times with the value of alpha and collapse being taken randomly for each calculation using the method of Monte Carlo Simulation (as discussed above); thus generating a sample of n settlement values. Taking the mean of this sample and repeating the simulation for each value of n ten times, the variance between the mean value of each generated sample can be determined. When this variance between the sample means becomes minimal a suitable value of n is being used.

This is demonstrated in the following example, where settlement has been calculated at point 1 for scenario 4 (as above) with test conditions as summarised in Table 6.9. The results are given in Table 6.10 from which it can be seen that selecting a value of n equal to 200 gives a suitably representative sample of settlement values.

6.3 Program Testing

The parameters that describe both the backfill model and the timing of the settlement predictions are:

- The morphology of the excavation as described by the pit base.
- The number, positioning and size of distinct backfill blocks.
- The timing of the placement of each backfill block.
- The mean creep compression rate and collapse strain parameters together with standard deviation values for each distinct backfill block.
- The magnitude and timing of any groundwater table rise.
- The settlement prediction period as defined from the end of the compaction operations.
- The positioning and timing of placement of any surcharge.

The effect these parameters have on the predicted settlement is examined in the following section. In each case default values are used for the 4 parameters examined above i.e.

Percentage Stress 'Cut-Off' value (%)	100
Layer Thickness (mm)	500
POI Density	4
Number of Simulations	200

Percentage Stress 'Cut-Off' value(%)		Layer Thickness (mm)		POI Density	Number of Simulations	
Default (100)		Default (500)		4 [1]	10 to 1000	
Prediction Period [2]		Groundwater Table		Surcharge		
From	To	Rise (m)	Period (days)	Height (m)	Location	
30	9155	Not considered		Not considered		

Table 6.9. Test conditions for determining the number of differential settlement calculations.

1 Adjustment for surrounding material is only carried out for determining the absolute differential settlement.

2 Number of days from the completion of backfilling operations

	Sample means for estimates of settlement given n number of repetitions						
Run Number	n = 10	25	50	100	200	500	1000
1	301.79	302.80	301.35	303.09	302.25	302.16	302.47
2	299.30	300.64	301.18	302.72	302.12	302.50	302.67
3	301.83	301.36	301.56	301.75	302.29	302.57	302.03
4	297.92	303.38	301.45	304.08	301.82	301.85	302.70
5	304.81	301.90	300.90	300.84	303.16	302.59	302.60
6	302.55	303.19	300.93	302.25	302.54	302.58	302.05
7	302.40	305.09	303.89	301.97	303.19	302.13	302.01
8	302.09	298.71	300.45	302.17	302.90	301.94	302.21
9	300.35	304.85	302.69	302.92	302.16	302.30	301.92
10	300.50	300.94	303.34	303.75	301.99	301.80	302.46
mean	301.35	302.29	301.77	302.55	302.44	302.24	302.31
variance	3.69	3.89	1.30	0.93	0.24	0.097	0.09

Table 6.10. Mean settlement results from samples generated by n repetitions

6.3.1 Backfill thickness

In this example the settlement is calculated for the case where the 1 to 19 blocks shown in Figure 6.1 are considered as one single block, scenario 6, the properties of which are shown in Table 6.11. This simply leaves the thickness of the block and its compaction properties as the main components influencing settlement. Thus the influence backfill thickness which is defined by the morphology of the pit floor and restoration surface, has on the predicted settlement can be seen. This is demonstrated in Figures 6.17 and 6.18. Figure 6.17 shows the predicted settlement contours clearly mirroring the backfill thickness contours shown in Figure 6.18 which due to the restoration surface being level also represent the pit base contours.

Therefore in this simplified case the absolute surface settlement is shown to be directly proportional to the thickness of the backfill. However as a backfilling operation generally takes a considerable period of time and a backfill cannot be considered as simply one homogenous block, the backfill needs to be split up into distinct blocks delineated by both time and changes in material properties. Thus factors other than simply backfill thickness become influential.

Block	Period of Placement (days) *		Alpha (S.D.)	Collapse (S.D.)	Unit Weight (kN/m^3)
1	0	990	0.5 (0.15)	0.9 (0.28)	20

Table 6.11. Block placement periods and properties for scenario 6.

* Number of days from the start of backfilling operations

Prediction Period*		Groundwater Table		Surcharge	
From	To	Rise (m)	Period (days)	Height (m)	Location
30	9155	Not considered		Not considered	

Table 6.12. Test conditions for the prediction of settlement for a backfill consisting of one block.

* Number of days from the completion of backfilling operations

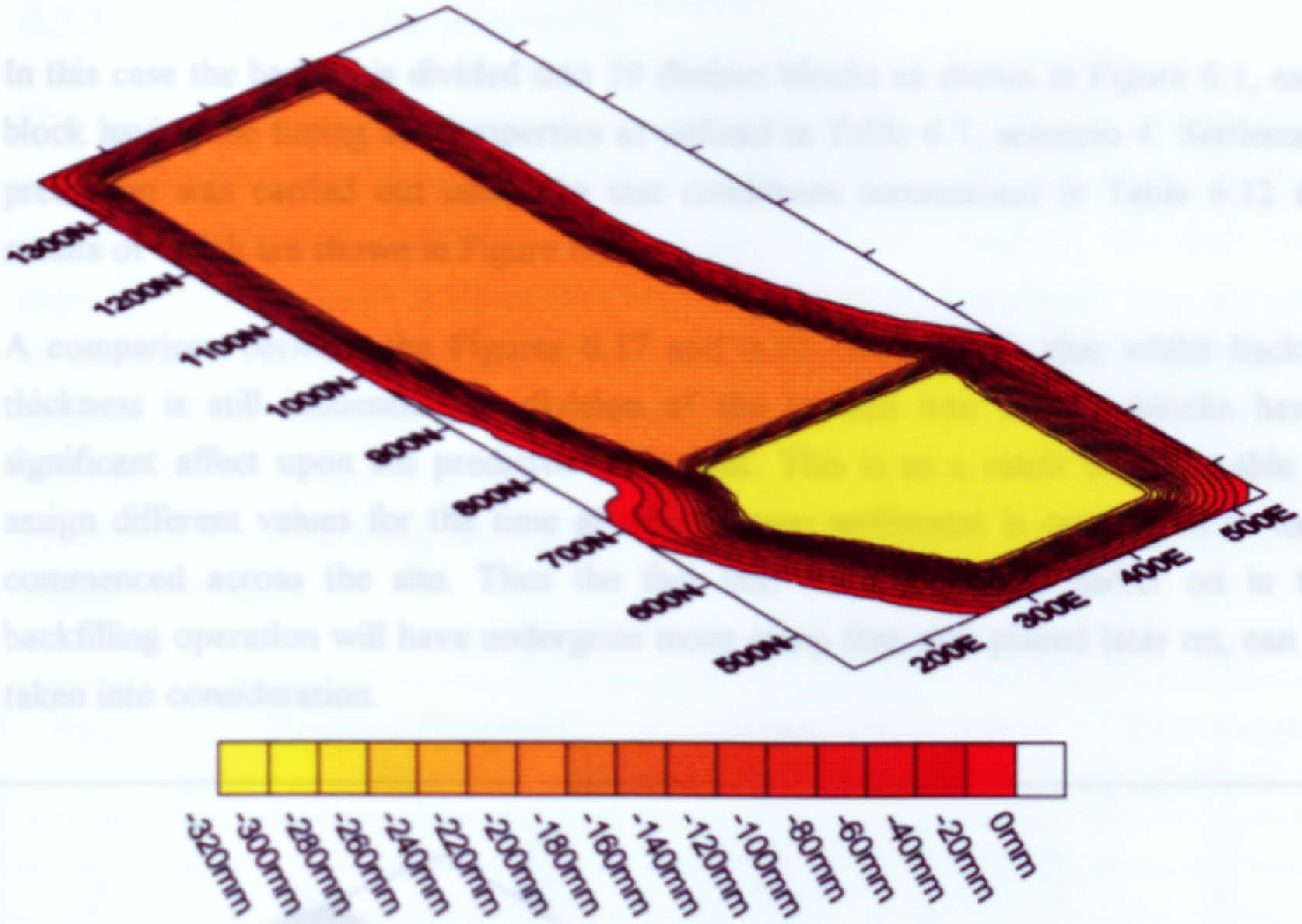


Figure 6.17. Predicted settlement contours for scenario 6, test conditions as Table 6.12.

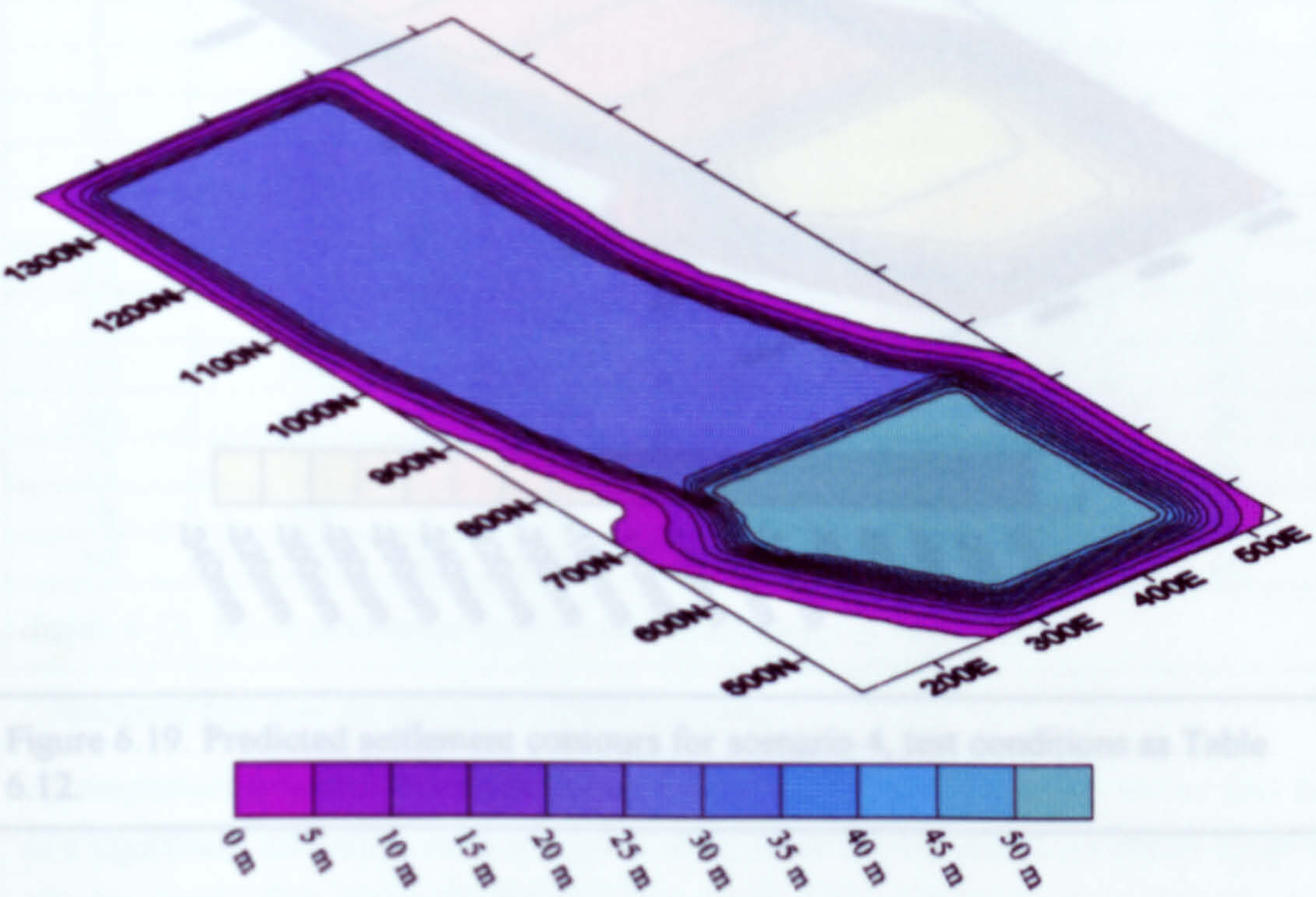


Figure 6.18. Backfill thickness / pit base depth contours.

6.3.2 Dividing a backfill into distinct blocks

In this case the backfill is divided into 19 distinct blocks as shown in Figure 6.1, each block having the timing and properties as defined in Table 6.7, scenario 4. Settlement prediction was carried out using the test conditions summarised in Table 6.12 the results of which are shown in Figure 6.19.

A comparison between the Figures 6.17 and 6.19 clearly show that whilst backfill thickness is still influential the division of the backfill into distinct blocks has a significant affect upon the predicted settlement. This is as a result of being able to assign different values for the time at which creep settlement is considered to have commenced across the site. Thus the fact that material placed earlier on in the backfilling operation will have undergone more creep than that placed later on, can be taken into consideration.

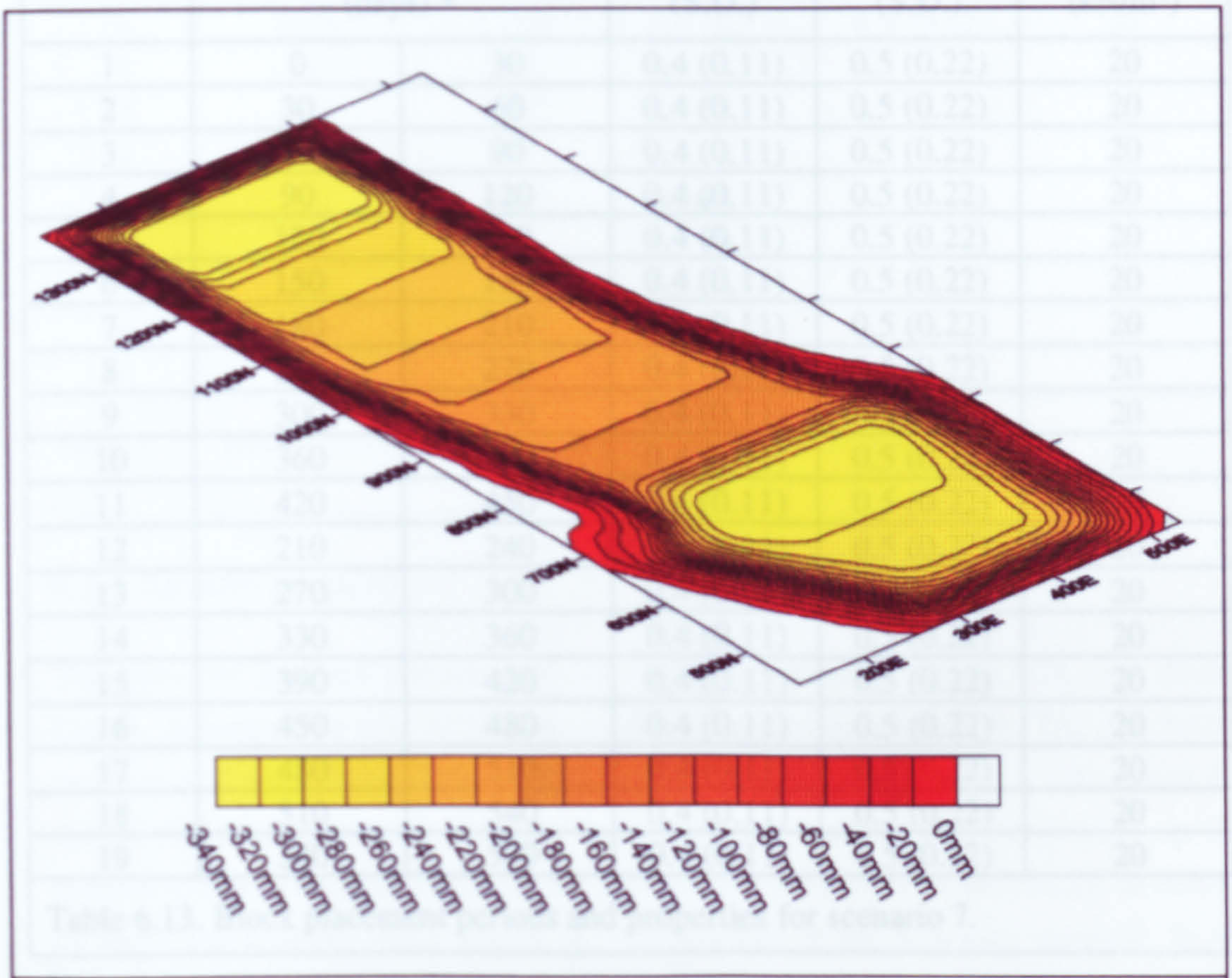


Figure 6.19. Predicted settlement contours for scenario 4, test conditions as Table 6.12.

6.3.3 Timing of backfill placement

The previous section demonstrated that by splitting the backfill into distinct blocks predicted settlement can be significantly affected. This is largely due to being able to assign different time origins for the commencement of creep to individual blocks dependant upon the time at which they and blocks above, were placed. Thus the timing of the backfill blocks will also be influential and can be demonstrated by the following example. Two backfills are considered placed over different time periods as summarised in Tables 6.13 and 6.14, scenarios 7 and 8 respectively. Settlement predictions are made using the test conditions given in Table 6.12 and are presented in Figures 6.20 and 6.21.

Block	Period of Placement (days) *		Alpha (S.D.)	Collapse (S.D.)	Unit Weight (kN/m³)
1	0	30	0.4 (0.11)	0.5 (0.22)	20
2	30	60	0.4 (0.11)	0.5 (0.22)	20
3	60	90	0.4 (0.11)	0.5 (0.22)	20
4	90	120	0.4 (0.11)	0.5 (0.22)	20
5	120	150	0.4 (0.11)	0.5 (0.22)	20
6	150	180	0.4 (0.11)	0.5 (0.22)	20
7	180	210	0.4 (0.11)	0.5 (0.22)	20
8	240	270	0.4 (0.11)	0.5 (0.22)	20
9	300	330	0.4 (0.11)	0.5 (0.22)	20
10	360	390	0.4 (0.11)	0.5 (0.22)	20
11	420	450	0.4 (0.11)	0.5 (0.22)	20
12	210	240	0.4 (0.11)	0.5 (0.22)	20
13	270	300	0.4 (0.11)	0.5 (0.22)	20
14	330	360	0.4 (0.11)	0.5 (0.22)	20
15	390	420	0.4 (0.11)	0.5 (0.22)	20
16	450	480	0.4 (0.11)	0.5 (0.22)	20
17	480	510	0.4 (0.11)	0.5 (0.22)	20
18	510	540	0.4 (0.11)	0.5 (0.22)	20
19	540	570	0.4 (0.11)	0.5 (0.22)	20

Table 6.13. Block placement periods and properties for scenario 7.

* Number of days from the start of backfilling operations

A comparison between the predicted settlement for the two scenarios shows that there is a significant difference even though in both cases the backfill compaction properties are the same. This is due to the fact that as in the previous example, blocks placed early on in the compaction operation undergo a significant amount of creep strain prior to the completion of backfilling thus predicted settlements will be less than for blocks placed closer to the end of operations. Thus the timing of block placement is shown to have a significant influence on the resultant settlement.

Block	Period of Placement (days) *		Alpha (S.D.)	Collapse (S.D.)	Unit Weight (kN/m^3)
1	0	75	0.4 (0.11)	0.5 (0.22)	20
2	75	150	0.4 (0.11)	0.5 (0.22)	20
3	150	225	0.4 (0.11)	0.5 (0.22)	20
4	225	300	0.4 (0.11)	0.5 (0.22)	20
5	375	450	0.4 (0.11)	0.5 (0.22)	20
6	525	600	0.4 (0.11)	0.5 (0.22)	20
7	675	750	0.4 (0.11)	0.5 (0.22)	20
8	825	900	0.4 (0.11)	0.5 (0.22)	20
9	975	1050	0.4 (0.11)	0.5 (0.22)	20
10	1125	1200	0.4 (0.11)	0.5 (0.22)	20
11	1275	1350	0.4 (0.11)	0.5 (0.22)	20
12	300	375	0.4 (0.11)	0.5 (0.22)	20
13	450	525	0.4 (0.11)	0.5 (0.22)	20
14	600	675	0.4 (0.11)	0.5 (0.22)	20
15	750	825	0.4 (0.11)	0.5 (0.22)	20
16	900	975	0.4 (0.11)	0.5 (0.22)	20
17	1050	1125	0.4 (0.11)	0.5 (0.22)	20
18	1200	1275	0.4 (0.11)	0.5 (0.22)	20
19	1350	1425	0.4 (0.11)	0.5 (0.22)	20

Table 6.14. Block placement periods and properties for scenario 8.

* Number of days from the start of backfilling operations

6.3.4 Backfill properties

The previous sections demonstrate the influence dividing the backfill into distinct blocks, delineated by time of placement, has on predicted settlement. The compactive properties of the individual blocks will also be of significance and it is considered that this is the other important property by which the backfill must be divided up into distinct blocks. The significance of the compactive properties of the blocks, as defined by alpha and collapse strain values, is demonstrated in the following example. In this case settlement is calculated at points 1 and 2 for four different backfilling scenarios 3, 4, 5 and 9 under test conditions as summarised in Table 6.16. Scenarios 3, 4 and 5 are defined in Tables 6.6, 6.7 and 6.8 respectively whilst scenario 9 is defined in Table 6.15. The results, presented as settlement / time plots, are shown in Figures 6.22 and 6.23.

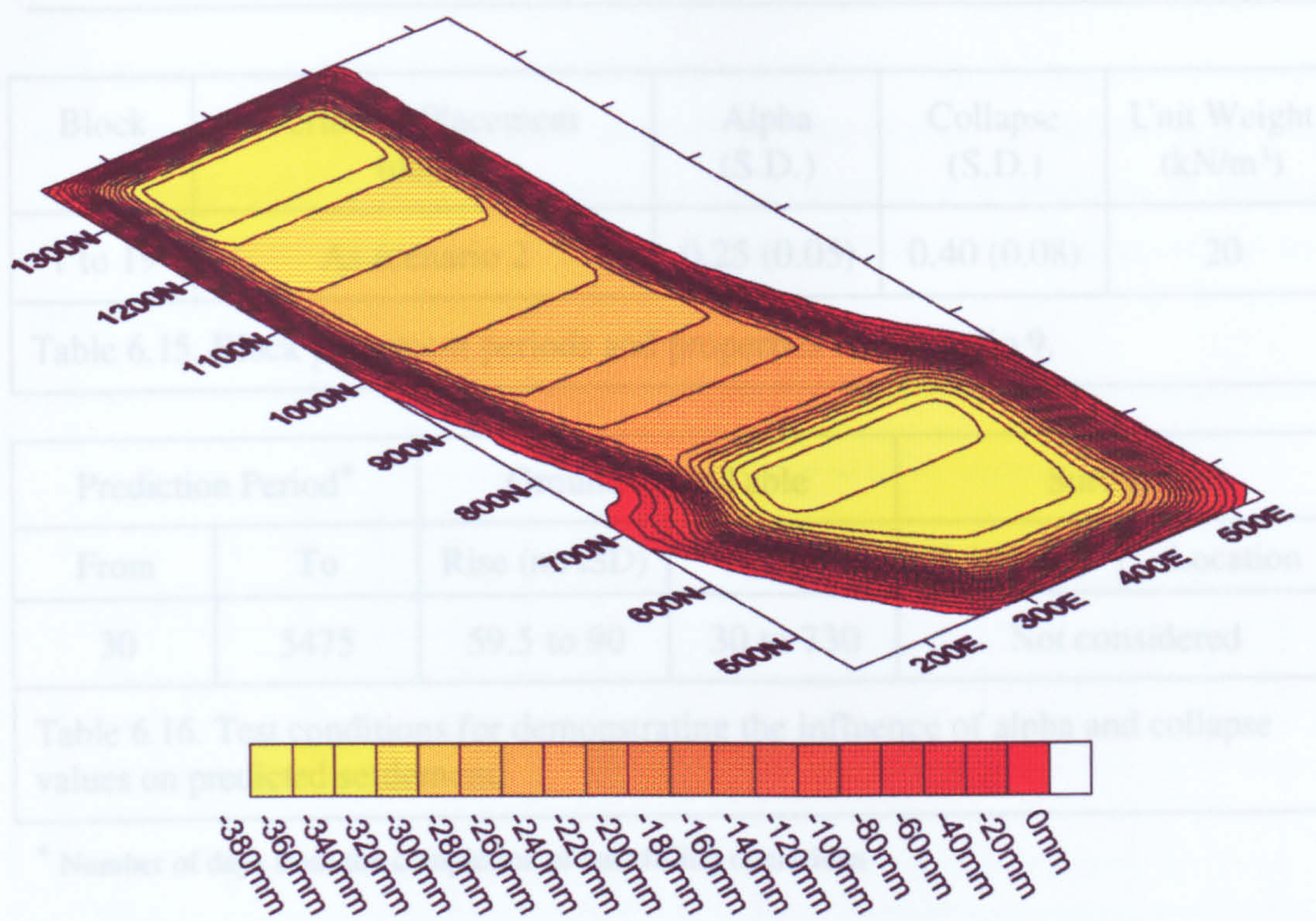


Figure 6.20. Predicted settlement contours for scenario 7, test conditions as Table 6.12.

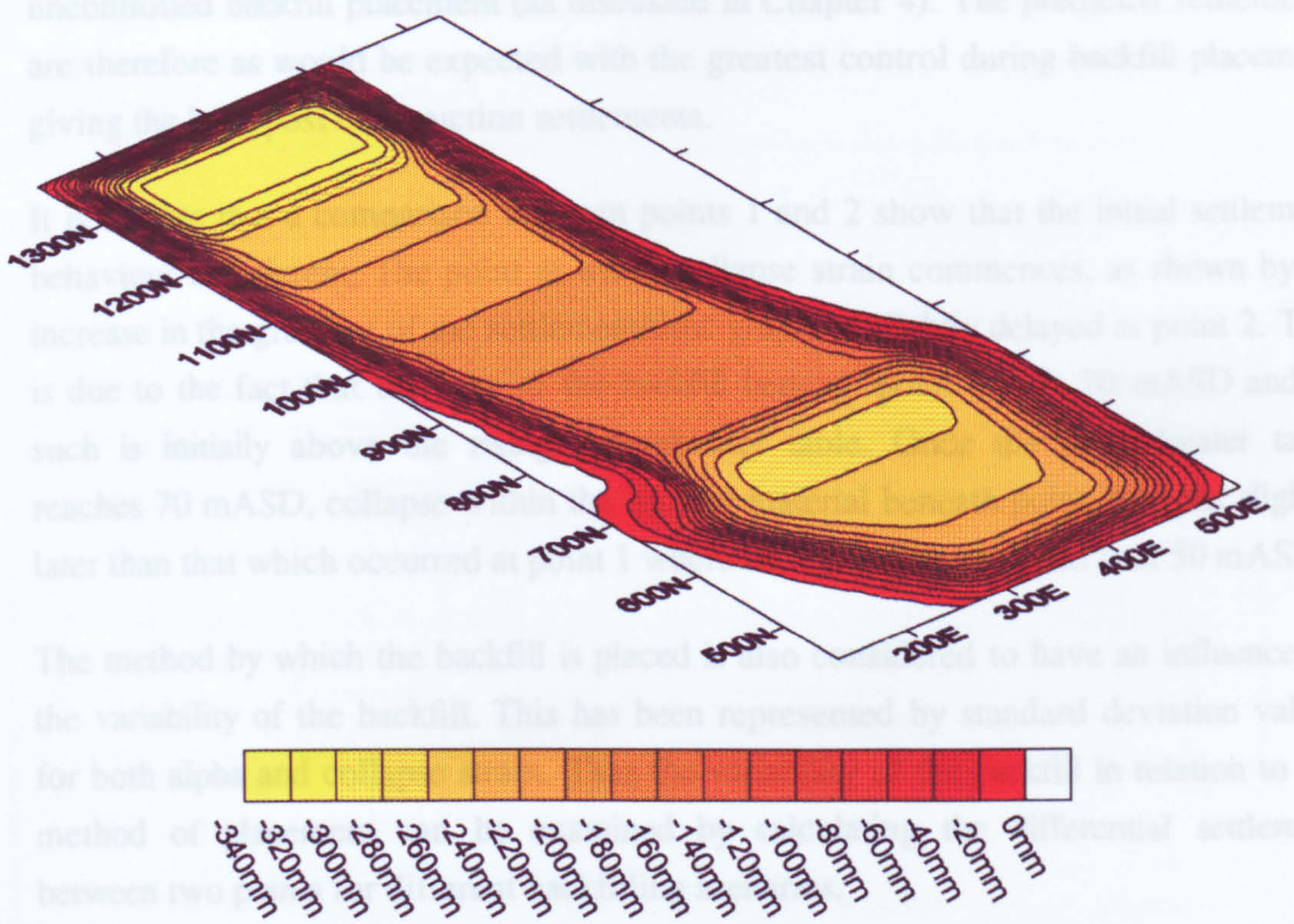


Figure 6.21. Predicted settlement contours for scenario 8, test conditions as Table 6.12.

Block	Period of Placement (days)	Alpha (S.D.)	Collapse (S.D.)	Unit Weight (kN/m³)
1 to 19	As scenario 2	0.25 (0.05)	0.40 (0.08)	20

Table 6.15. Block placement periods and properties for scenario 9.

Prediction Period*		Groundwater Table		Surcharge	
From	To	Rise (mASD)	Period*	Height (m)	Location
30	5475	59.5 to 90	30 to 730	Not considered	

Table 6.16. Test conditions for demonstrating the influence of alpha and collapse values on predicted settlement.

* Number of days from the completion of backfilling operations

Scenarios 3, 9 and 4 have been given compaction properties considered typical of backfill placed in a controlled manner as defined by a performance, method and thick layer specification respectively whilst scenario 5 is given properties associated with uncontrolled backfill placement (as discussed in Chapter 4). The predicted settlements are therefore as would be expected with the greatest control during backfill placement giving the least post construction settlements.

It is of note that a comparison between points 1 and 2 show that the initial settlement behaviour is different. The point at which collapse strain commences, as shown by an increase in the gradient of the settlement/time graph, is slightly delayed at point 2. This is due to the fact that the base of the backfill beneath point 2 is at 70 mASD and as such is initially above the rising groundwater table. Once the groundwater table reaches 70 mASD, collapse within the backfill material beneath point 2 begins slightly later than that which occurred at point 1 where the base of the backfill is at 50 mASD.

The method by which the backfill is placed is also considered to have an influence on the variability of the backfill. This has been represented by standard deviation values for both alpha and collapse strain. Thus the variability of the backfill in relation to the method of placement can be examined by calculating the differential settlement between two points for different backfilling scenarios.

Differential settlement has been calculated between points 3 and 4 under the test conditions given in Table 6.16 for the four backfilling scenarios 3, 4, 5 and 9; the results of which are presented in Figures 6.24. This shows the average difference in the settlement between the two points increases, together with the population variance, with decreasing backfill compaction and control. The increase in variance is as a result of increased variability and the increase in the average is a combination of increasing alpha and collapse strain values together with a reduction in the adjustment due to the influence of surrounding material.

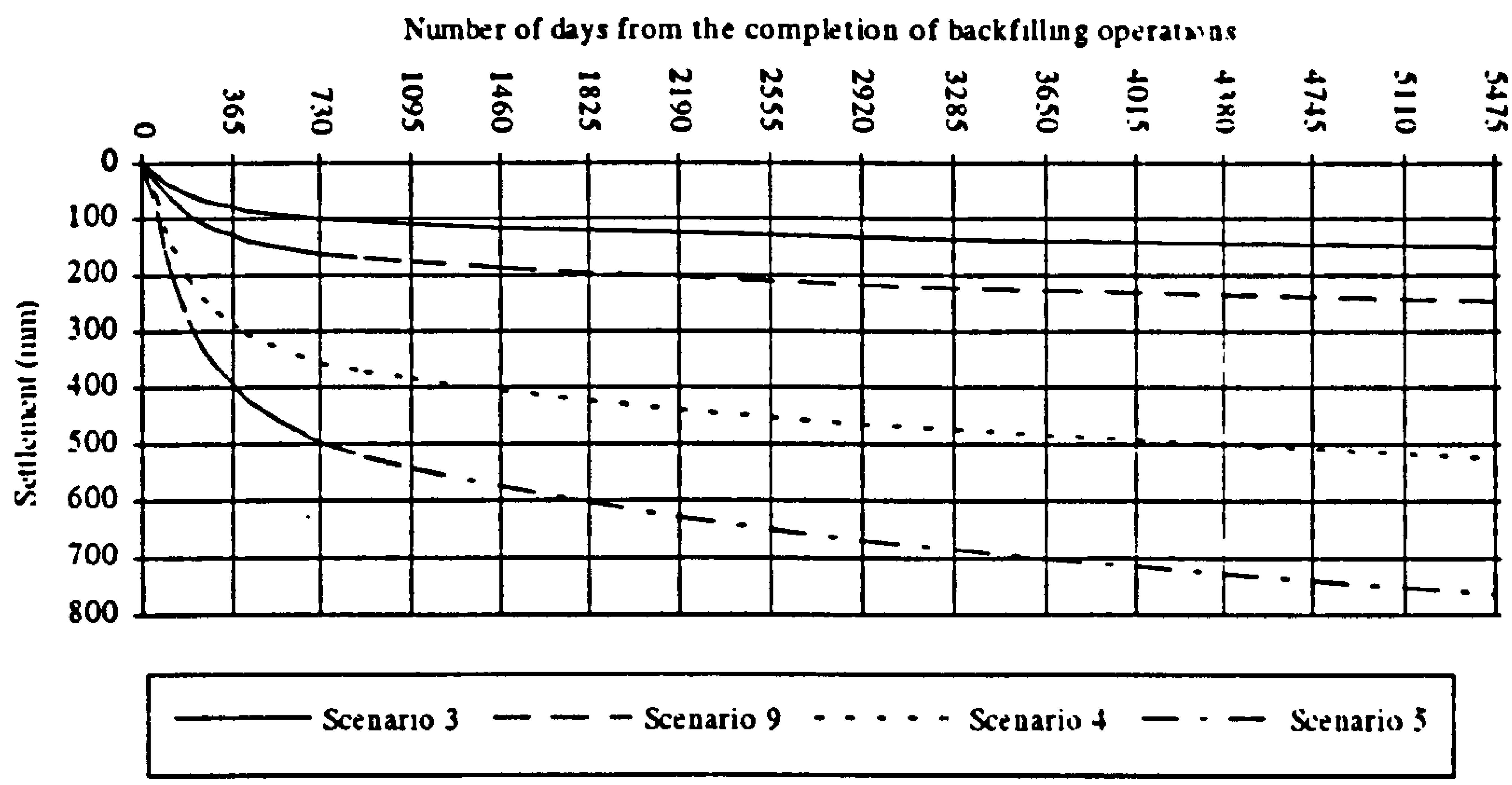


Figure 6.22. Predicted settlement / time plots for scenarios 3, 4, 5 and 9, point 1, test conditions as Table 6.16.

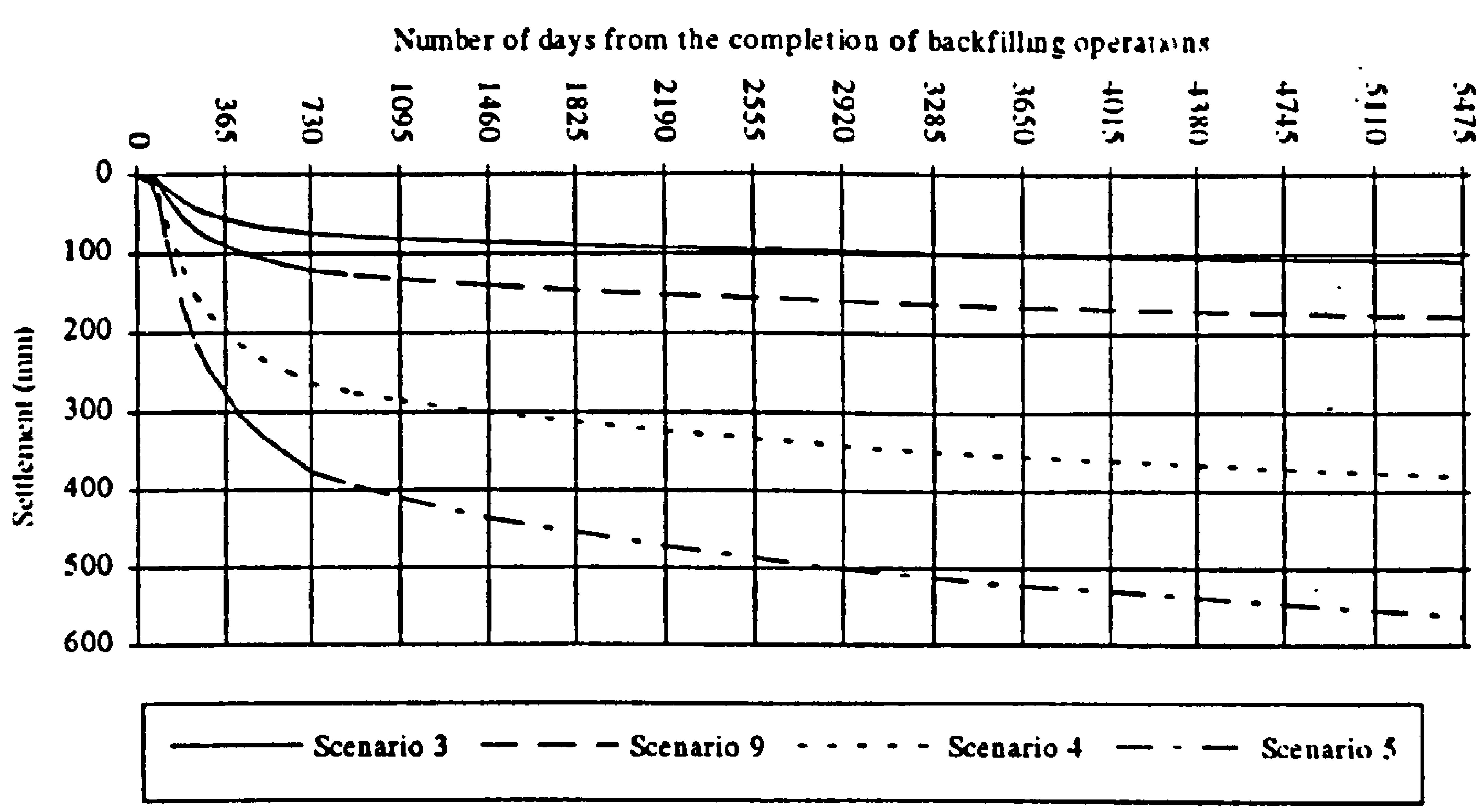


Figure 6.23. Predicted settlement / time plots for scenarios 3, 4, 5 and 9, point 2, test conditions as Table 6.16.

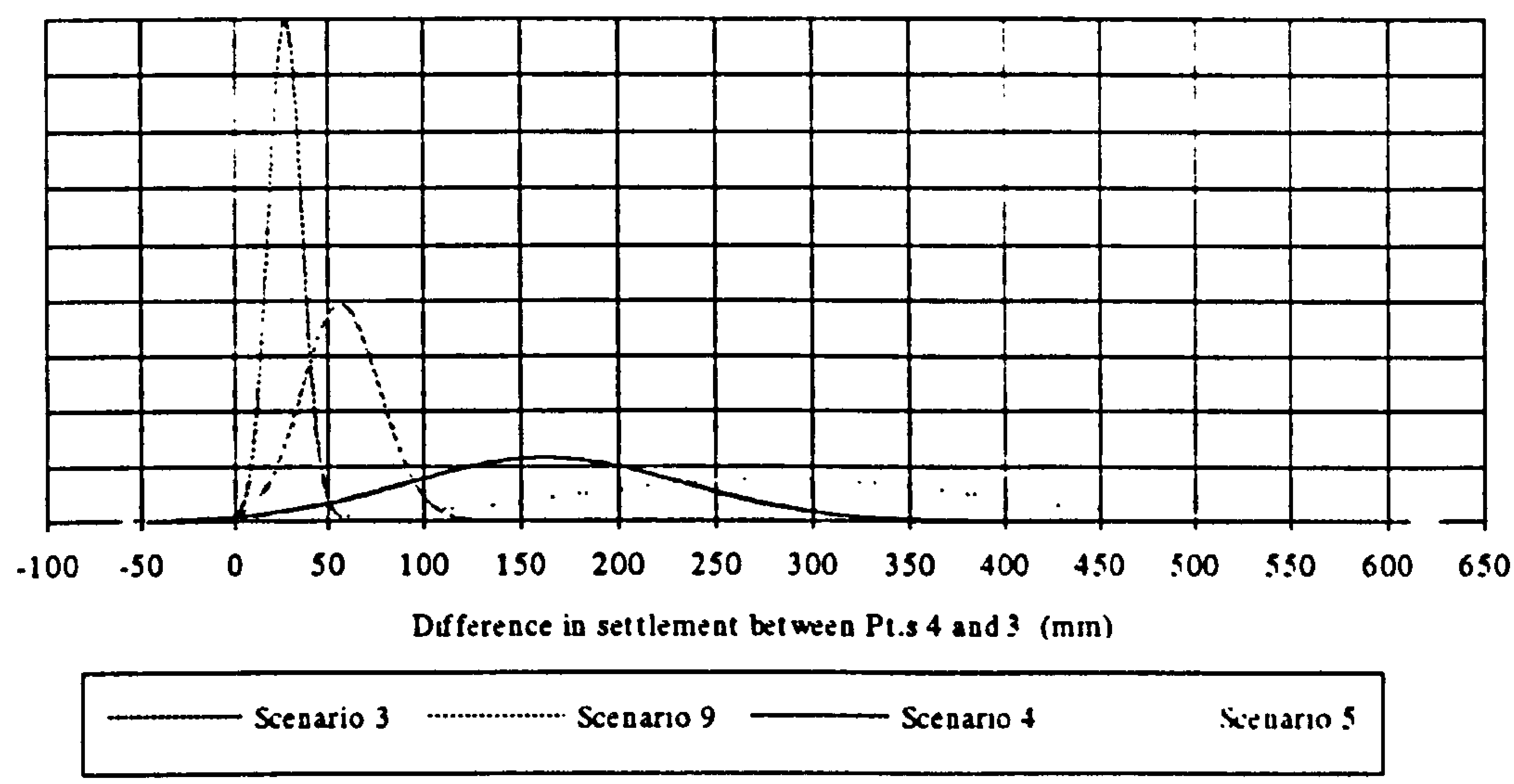


Figure 6.24. Normal distribution curves for the range of the difference in settlement between points 4 and 3 due to backfill heterogeneity.

6.3.5 Groundwater table rise

Having examined the influence of dividing the backfill into distinct blocks, the next important factor influencing settlement is that of the groundwater table. The behaviour of the groundwater table rise has a significant effect on predicted settlements. This is examined by calculating settlement at points 1 and 2, scenario 4, under the test conditions given in Table 6.17. Four different groundwater tables are considered as shown in Figure 6.25. Groundwater tables 1, 2 and 3 rise through the backfill immediately upon completion of backfilling operations reaching a final restoration level after 2 years. Groundwater table 4 is delayed by one year reaching its restoration level after only one year. All four groundwater tables start to rise from 50 mASD with 1, 2 and 4 reaching a restoration level of 90 mASD whilst 3 reaches a level of 75 mASD. The predicted settlement results are given in Figures 6.26 and 6.27.

The results show the periods over which collapse strain is occurring by the increased gradient of the settlement / time curves. These coincide with the periods during which the groundwater table is rising up through the backfill. Thus at point 1, gwt 4, creep strain is seen followed after 12 months by a rapid increase in settlement coinciding with the time at which the groundwater table rises through the backfill. At point 2 similar initial periods of creep are seen for gwt's 1, 3 and 4 coinciding with the time taken for the groundwater table to reach 70 mASD, the base of the backfill beneath this point.

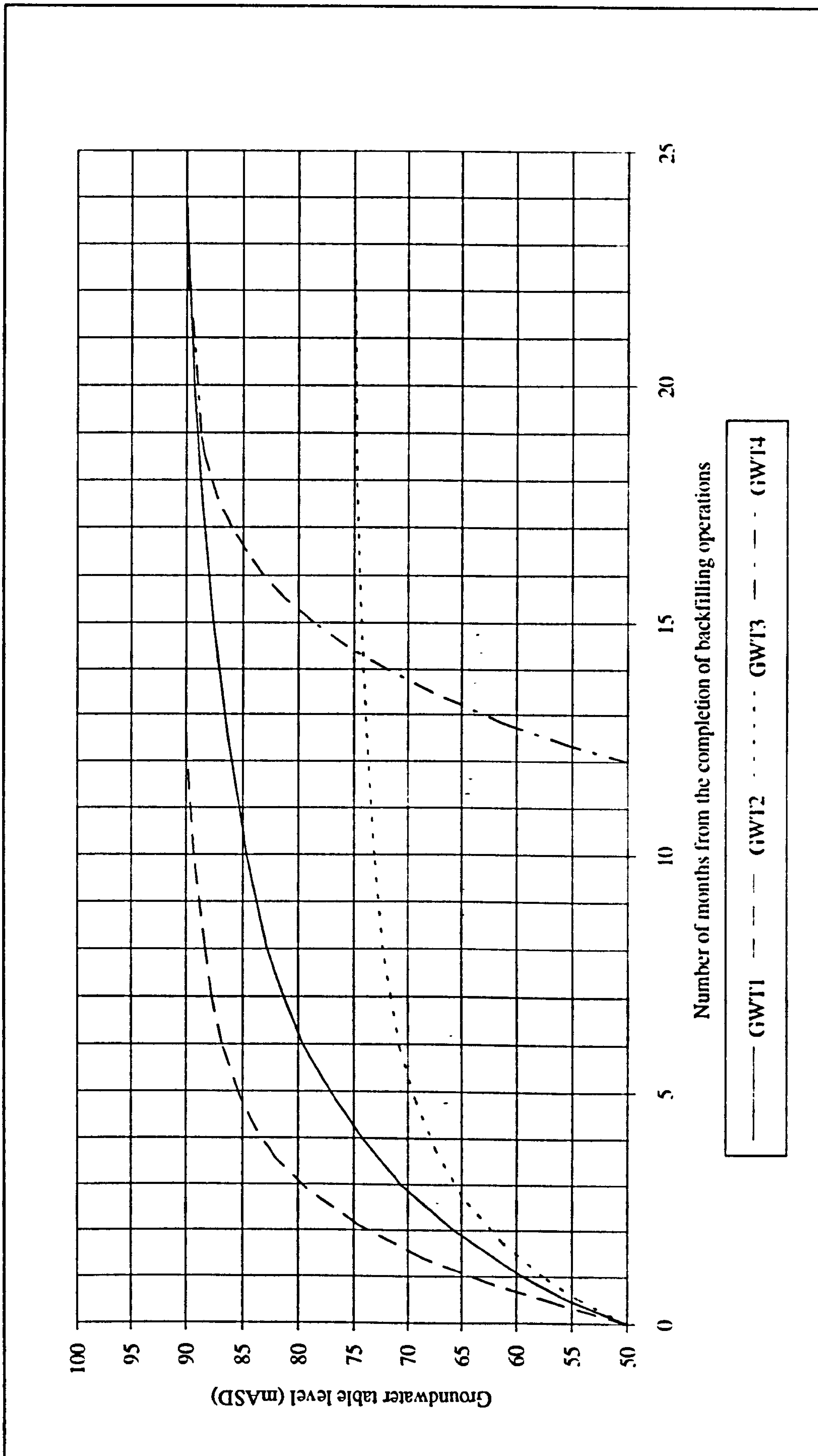


Figure 6.25. Groundwater table level / time plots for 4 different groundwater table rises (GWT1 to GWT4).

Prediction Period*		Groundwater Table	Surcharge	
From	To	4 gwt's considered. (see Figure 6.25)	Height (m)	Location
30	730		Not considered	

Table 6.17. Test conditions for demonstrating the influence of the rise in groundwater table on predicted settlements.

* Number of days from the completion of backfilling operations

A comparison between the settlement as a result of gwt 1 and 2 shows that the rate of collapse is proportional to the rate of groundwater table rise. Also gwt 3 shows that the magnitude of the total settlement is proportional to the thickness of backfill to become saturated.

Therefore the way in which the groundwater table rises through a backfill determines the magnitude of the resultant collapse settlement and the timing and rate at which this settlement occurs.

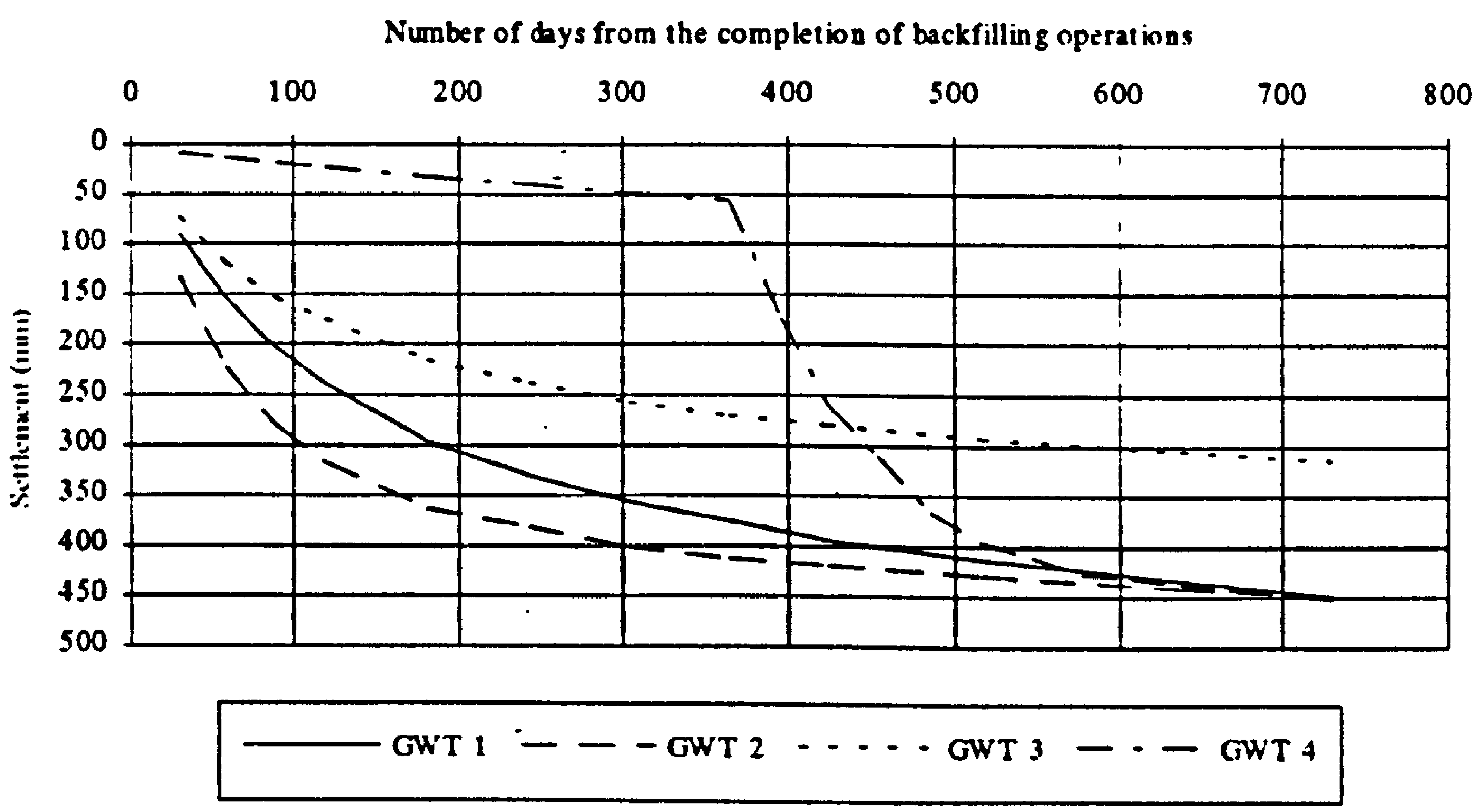


Figure 6.26. Predicted settlement / time plots for scenario 4, point 1, test conditions as Table 6.17.

6.3.6 Settlement Prediction Period

In section 6.3.3 the timing of the placement of the backfill blocks was shown to influence predicted settlements. This was as a result of the time a block is in place prior to the start of the settlement prediction period. It is therefore clear that the timing of the prediction period will also influence predicted settlements. This is demonstrated by calculating the settlement across the whole site for four different prediction periods as summarised in Table 6.18, scenario 4; the results are given in Figures 6.28 to 6.31.

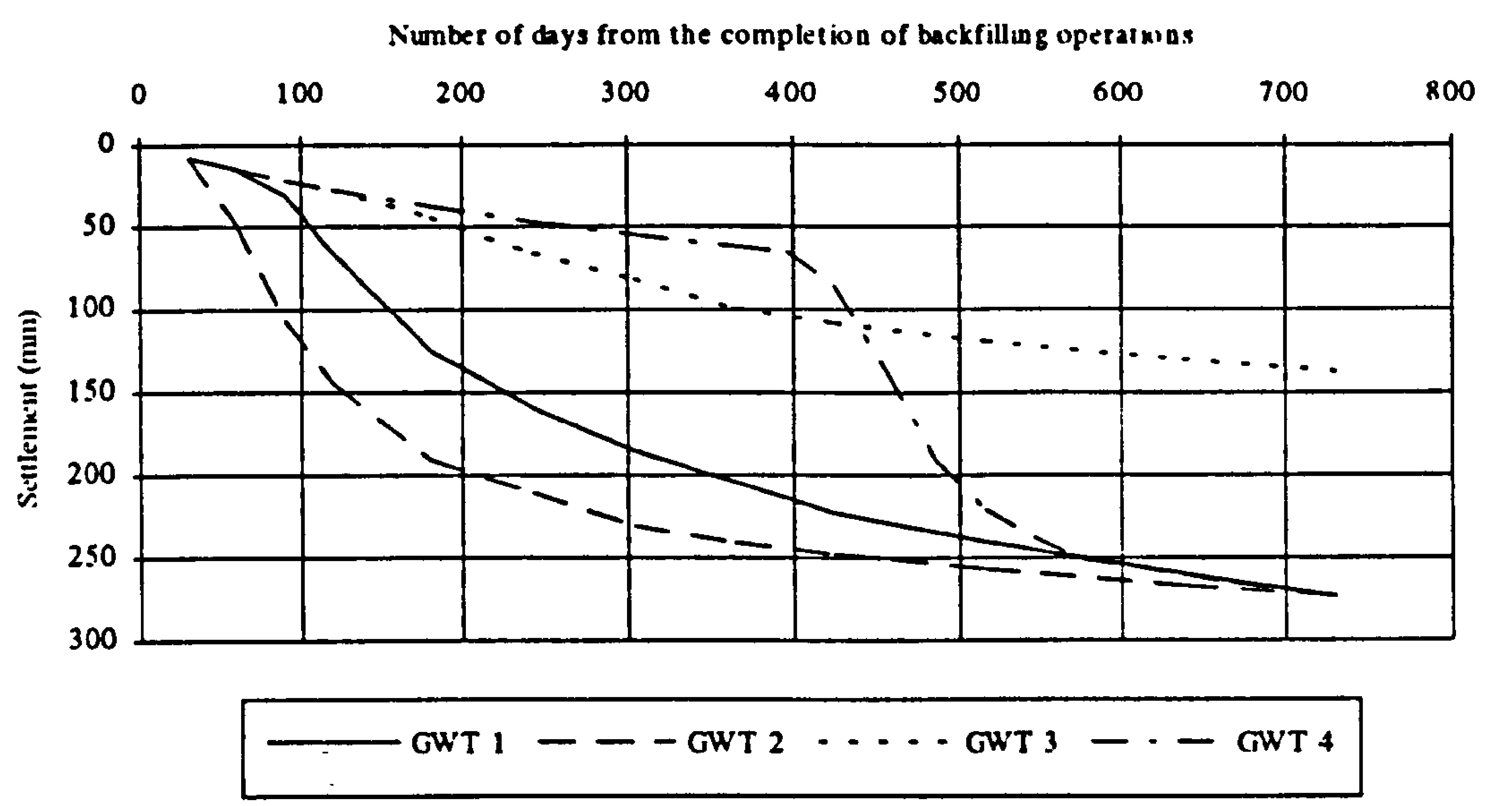


Figure 6.27. Predicted settlement / time plots for scenario 4, point 2, test conditions as Table 6.17.

The results clearly show that for the same prediction period, given a greater delay between the completion of backfilling operations and the start of the period the predicted settlements are less.

Prediction Period*	From	30	365	730	1825
	To	9155	9490	9855	10950
Groundwater Table - Not considered			Surcharge - Not considered		

Table 6.18. Test conditions for demonstrating the influence of the prediction period on predicted settlement.

* Number of days from the completion of backfilling operations

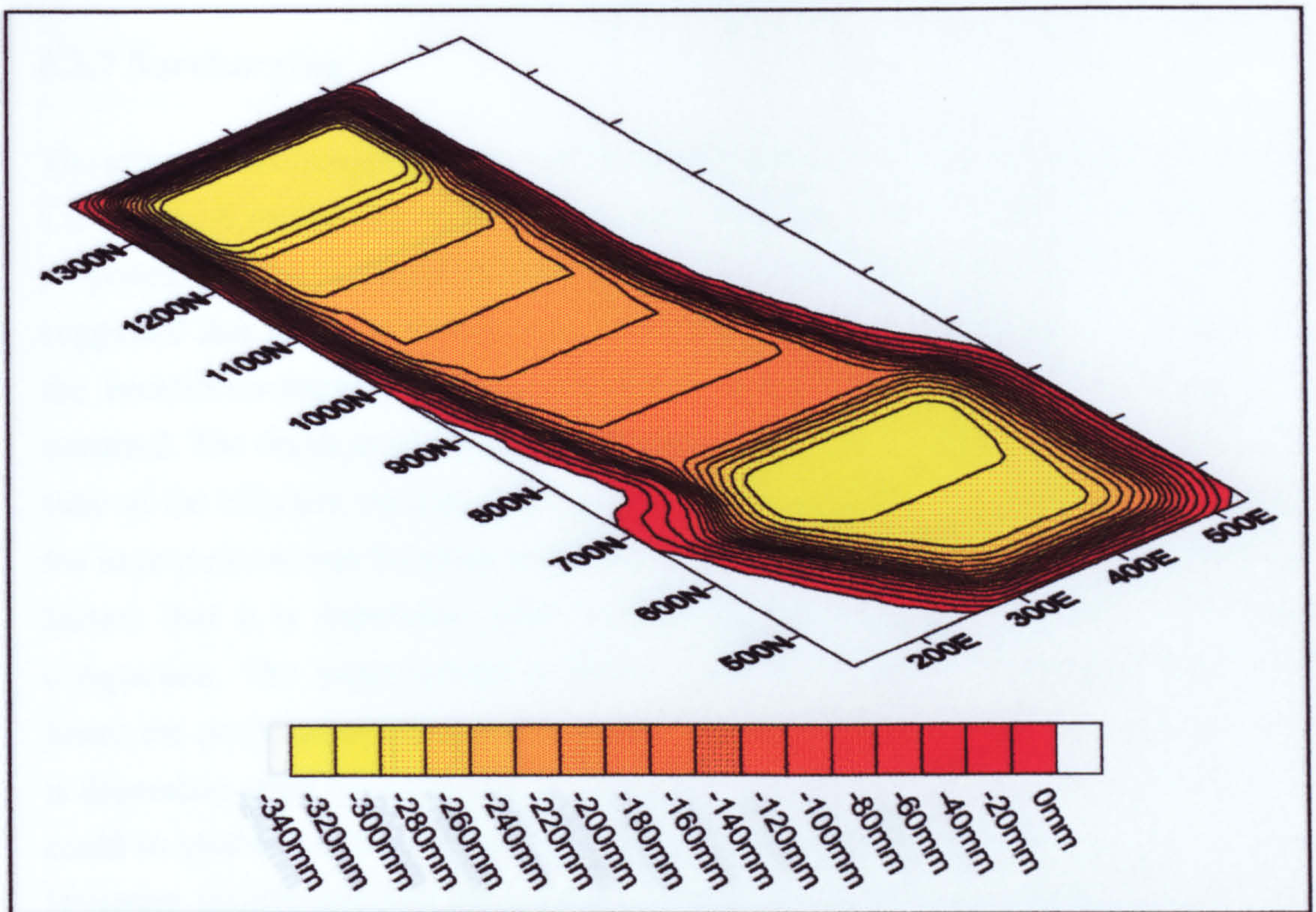


Figure 6.28. Predicted settlement contours for scenario 4, prediction period 30 to 9155 days, test conditions as Table 6.18.

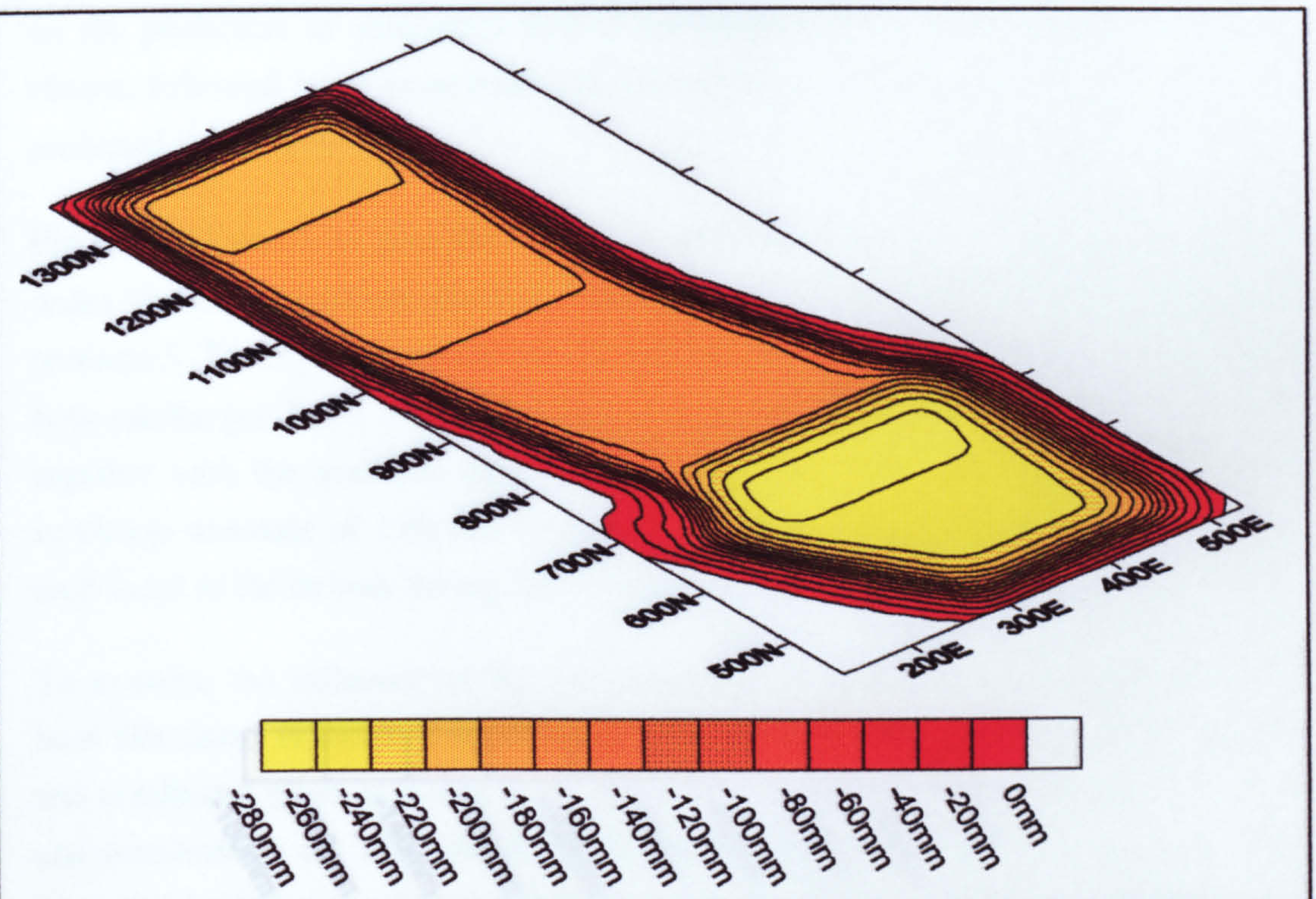


Figure 6.29. Predicted settlement contours for scenario 4, prediction period 365 to 9490 days, test conditions as Table 6.18.

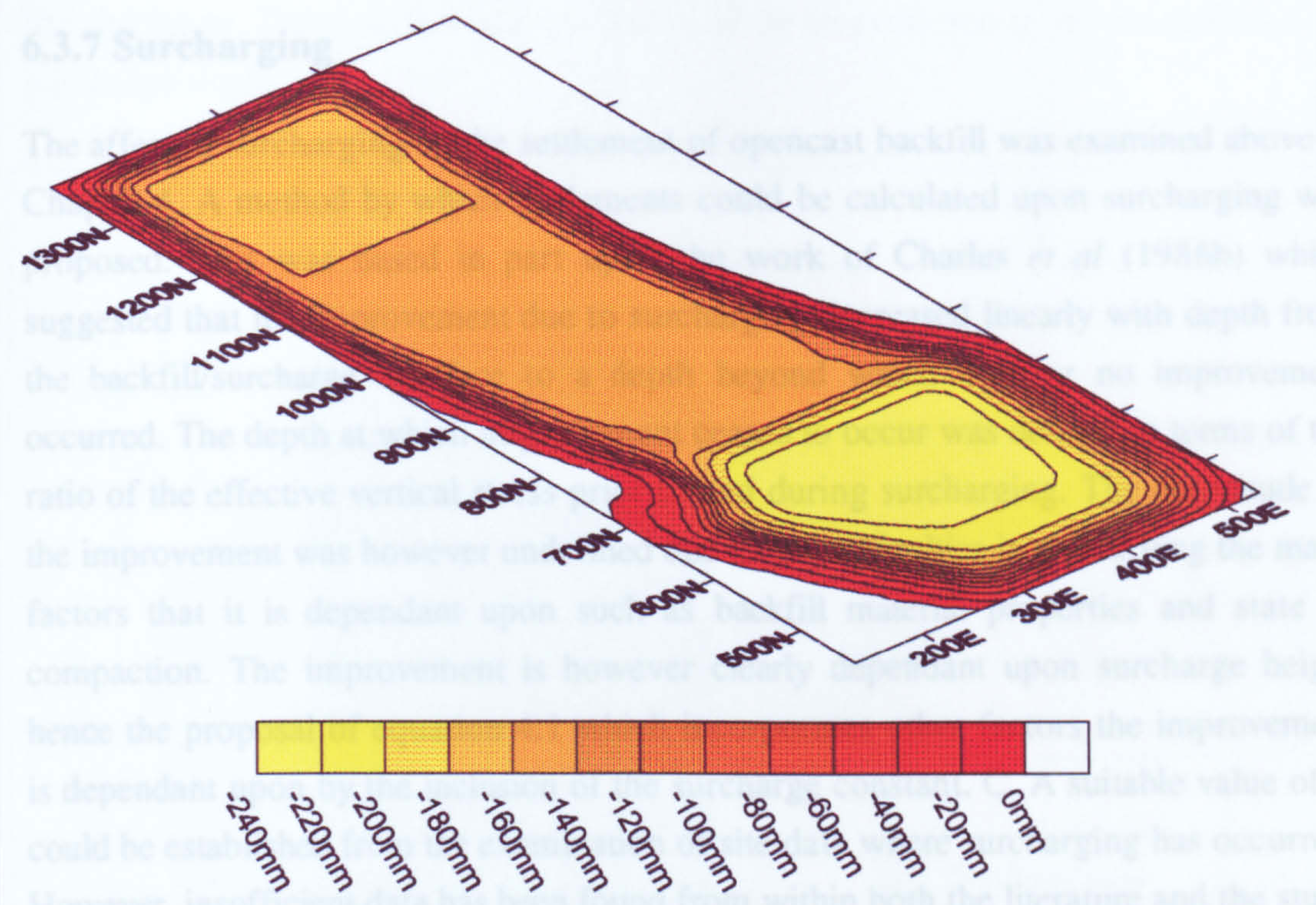


Figure 6.30. Predicted settlement contours for scenario 4, prediction period 730 to 9855 days, test conditions as Table 6.18.

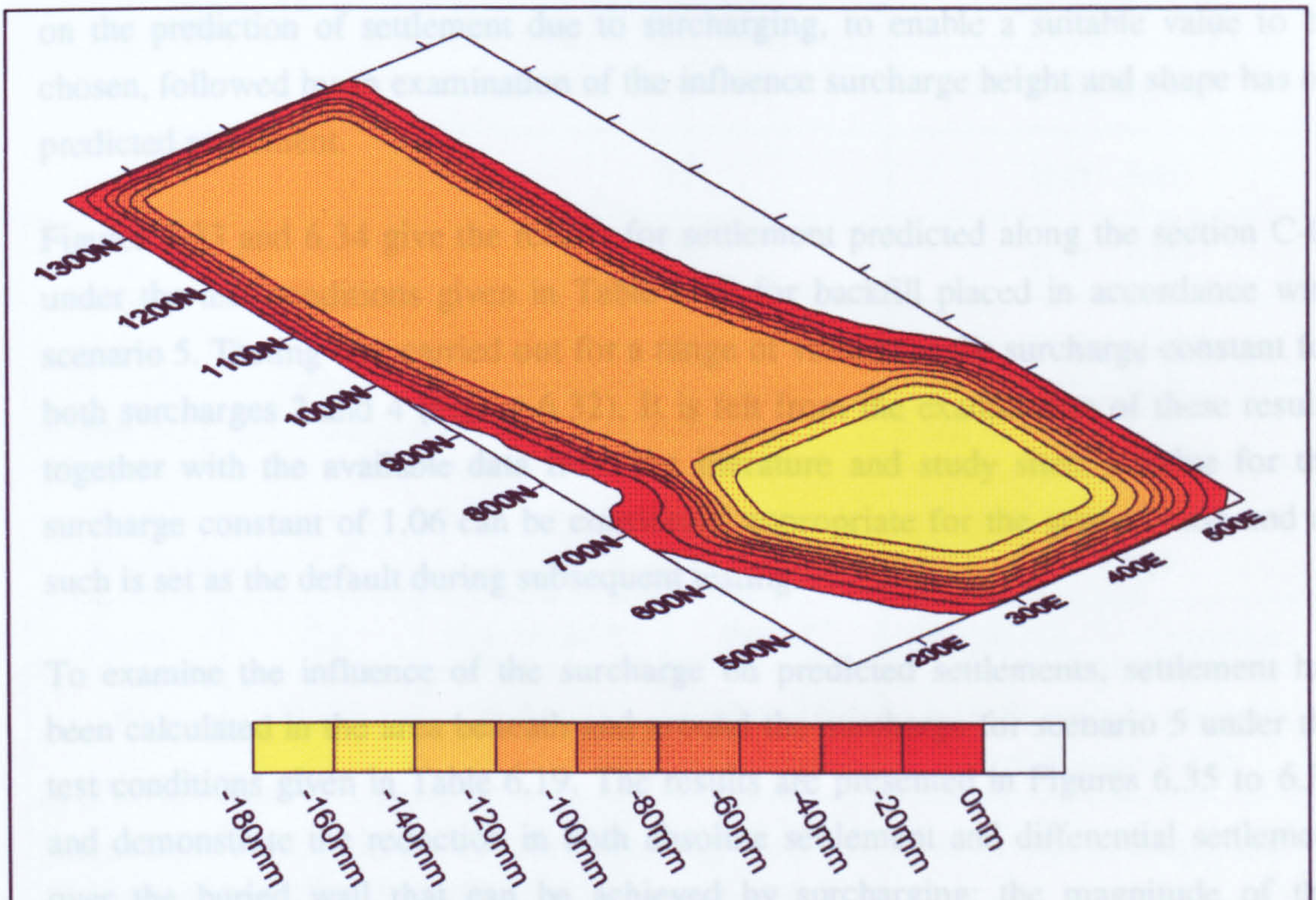


Figure 6.31. Predicted settlement contours for scenario 4, prediction period 1825 to 10950 days, test conditions as Table 6.18.

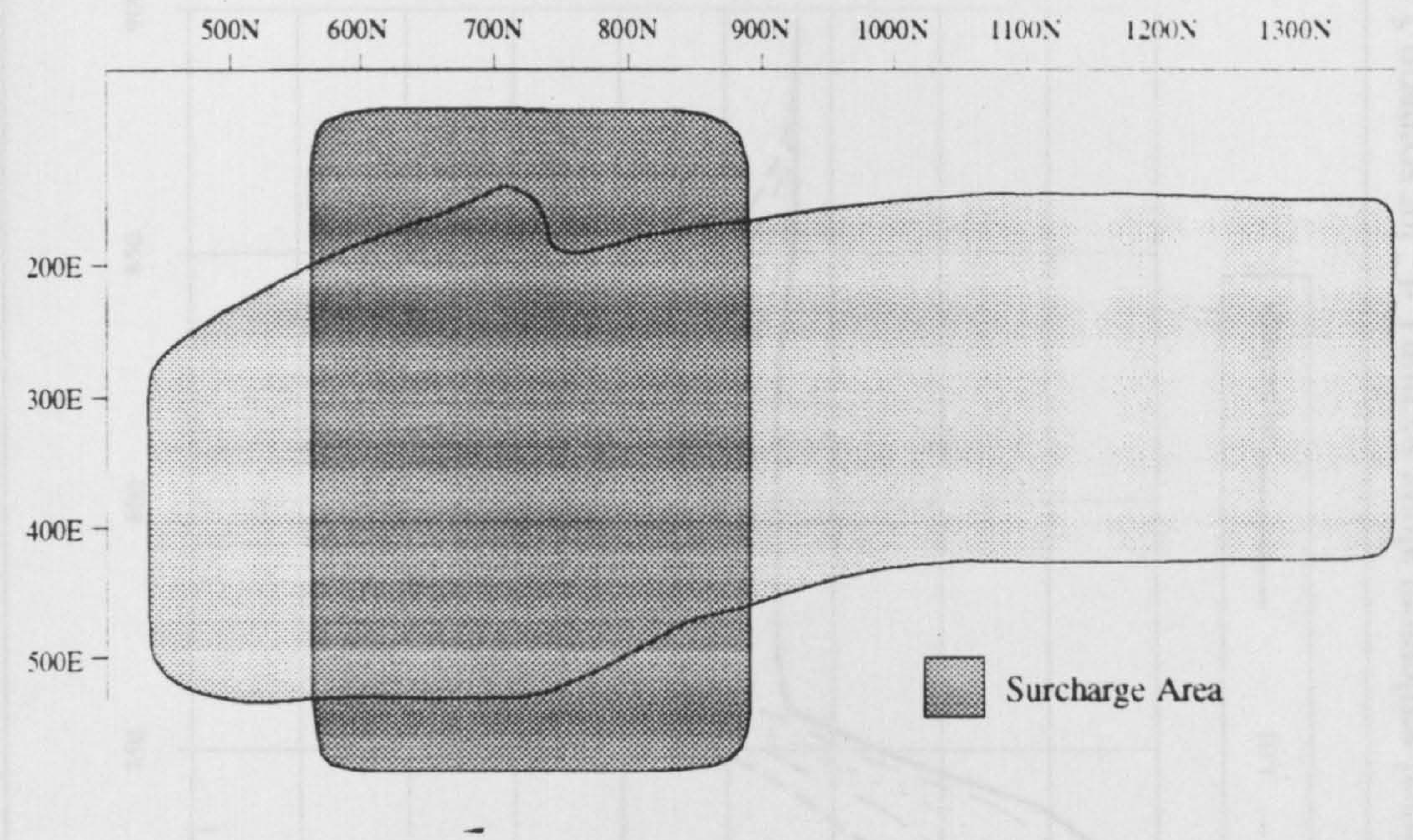
6.3.7 Surcharging

The affect of surcharging on the settlement of opencast backfill was examined above in Chapter 4. A method by which settlements could be calculated upon surcharging was proposed. This was based in part upon the work of Charles *et al* (1986b) which suggested that the improvement due to surcharging decreased linearly with depth from the backfill/surcharge interface to a depth beyond which little or no improvement occurred. The depth at which improvement ceases to occur was defined in terms of the ratio of the effective vertical stress prior to and during surcharging. The magnitude of the improvement was however undefined due to the difficulties in quantifying the many factors that it is dependant upon such as backfill material properties and state of compaction. The improvement is however clearly dependant upon surcharge height hence the proposal of equation 4.1 which incorporates other factors the improvement is dependant upon by the inclusion of the surcharge constant, C. A suitable value of C could be established from the examination of site data where surcharging has occurred. However, insufficient data has been found from within both the literature and the study site data to accurately establish a value for C.

The following testing therefore initially examines the affect the surcharge constant has on the prediction of settlement due to surcharging, to enable a suitable value to be chosen, followed by an examination of the influence surcharge height and shape has on predicted settlement.

Figures 6.33 and 6.34 give the results for settlement predicted along the section C-C' under the test conditions given in Table 6.19 for backfill placed in accordance with scenario 5. Testing was carried out for a range of values for the surcharge constant for both surcharges 2 and 4 (Figure 6.32). It is felt from the examination of these results together with the available data from the literature and study sites, a value for the surcharge constant of 1.06 can be considered appropriate for the general case and as such is set as the default during subsequent testing.

To examine the influence of the surcharge on predicted settlements, settlement has been calculated in the area beneath and around the surcharge for scenario 5 under the test conditions given in Table 6.19. The results are presented in Figures 6.35 to 6.39 and demonstrate the reduction in both absolute settlement and differential settlement over the buried wall that can be achieved by surcharging; the magnitude of this reduction being proportional to the surcharge height. Also shown is the influence the surcharge slope angle has on differential settlements that occur around the periphery of the surcharge. These settlements are shown to be proportional to the steepness of the surcharge slopes.



Surcharge	Height (m)	Slope Gradient
1	10	1(v) : 6(H)
2	20	1(v) : 3(H)
3	20	1(v) : 1(H)
4	30	1(v) : 2(H)

Figure 6.32. Size, shape and location of surcharges 1, 2, 3 and 4.

Prediction Period*		Groundwater Table		Surcharge	
From	To	Rise (mASD)	Period*	Height (m)	Location
30	9155	59.5 to 90	30 to 730	4 surcharge's considered. (see Figure 6.32)	

Table 6.19. Test conditions for demonstrating the influence of surcharge height and shape on predicted settlement.

* Number of days from the completion of backfilling operations

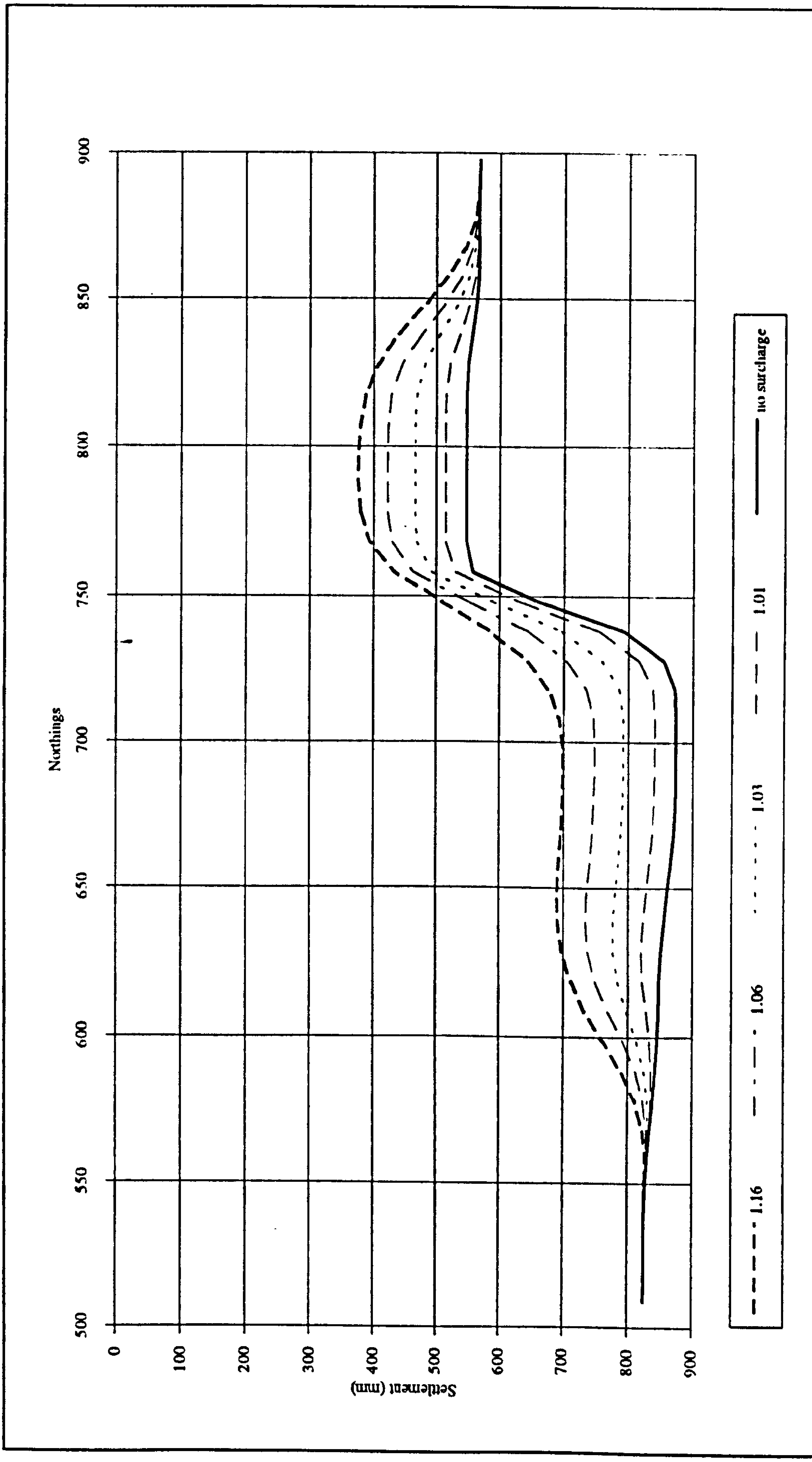


Figure 6.33. The influence of the surcharge constant (1.01 to 1.16) on the predicted settlement along section C-C' for scenario 5, surcharge 2.

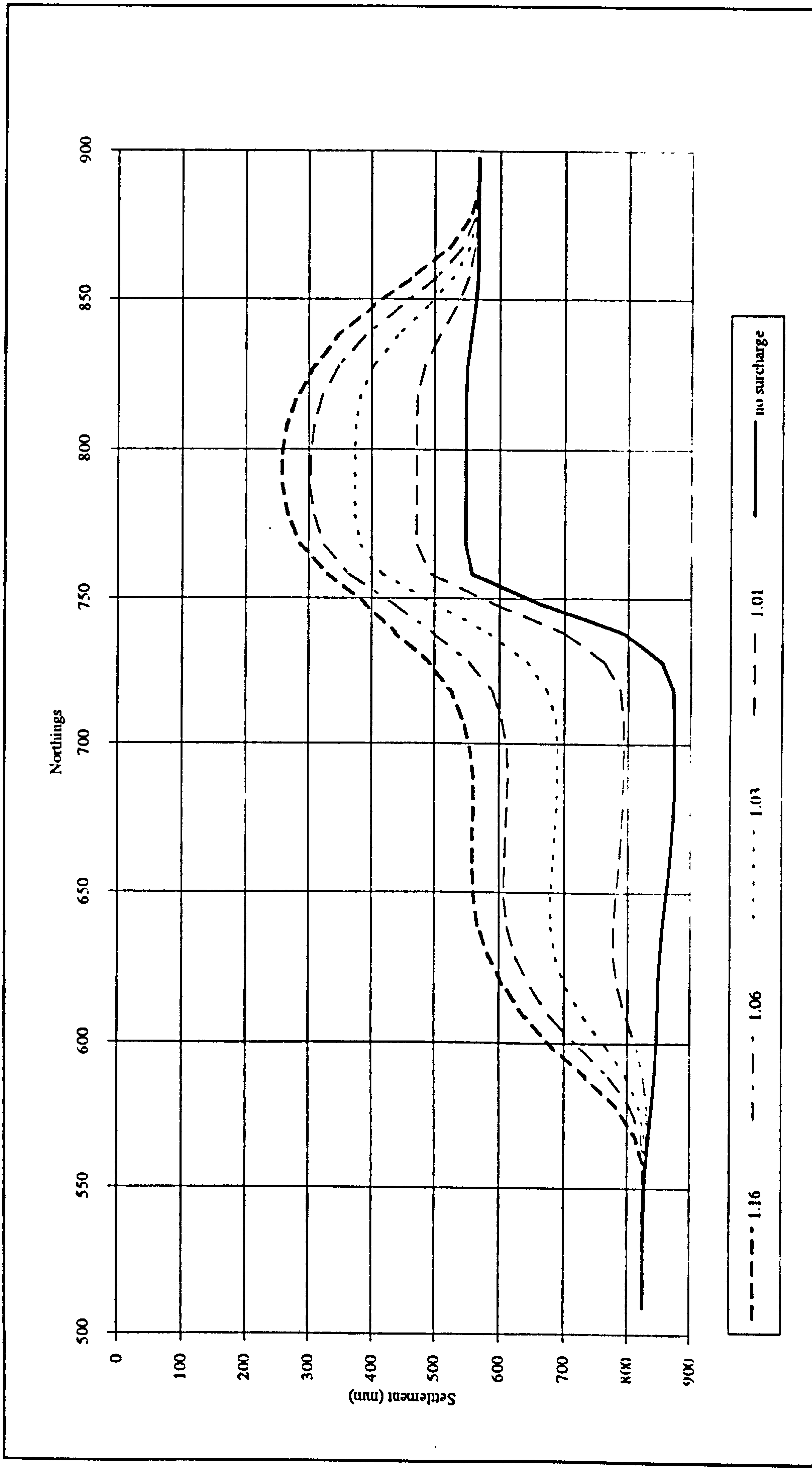


Figure 6.34. The influence of the surcharge constant (1.01 to 1.16) on the predicted settlement along section C-C' for scenario 5, surcharge 4.

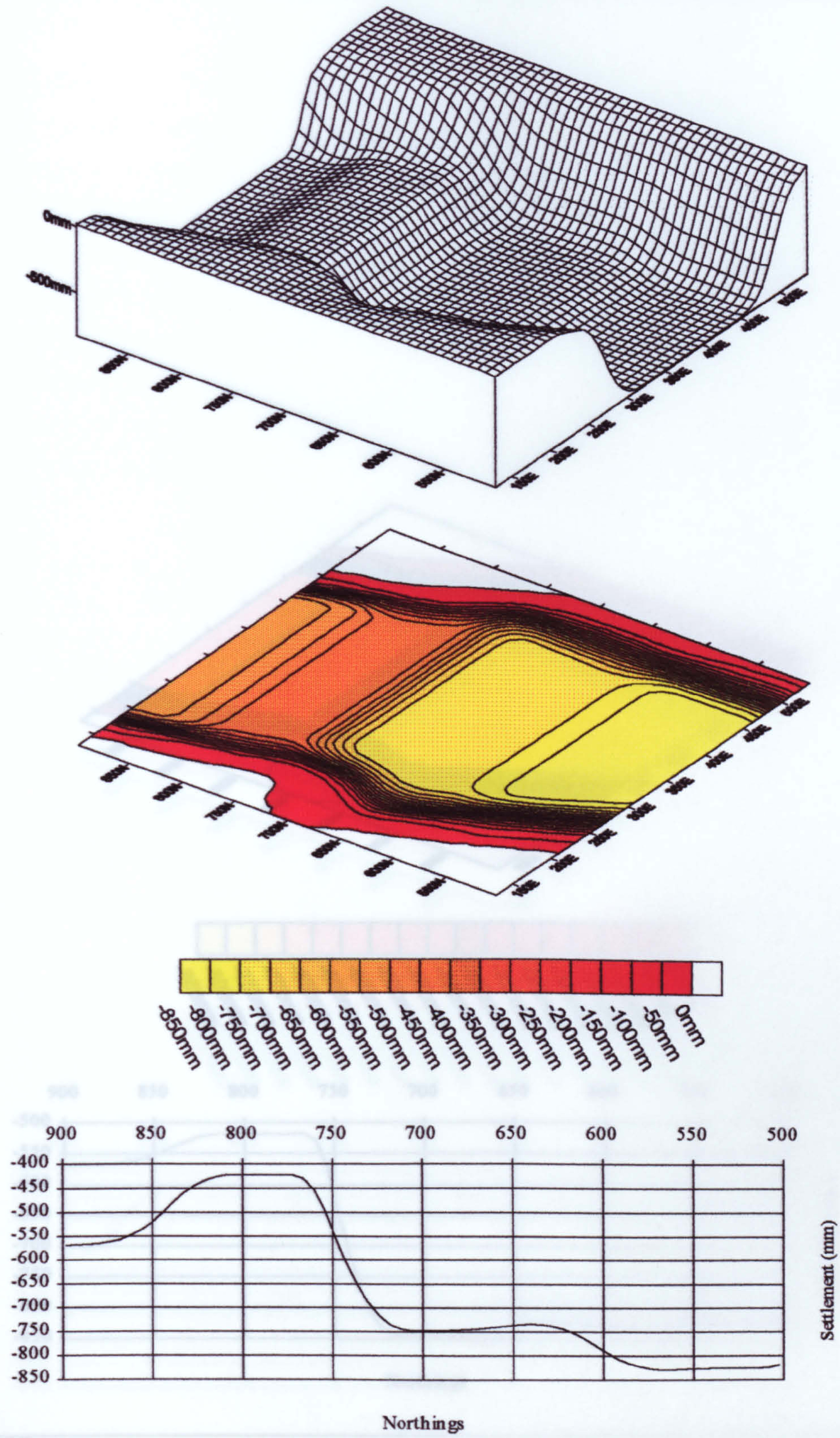


Figure 6.35. Predicted settlement for scenario 5, surcharge 1, test conditions as Table 6.1.

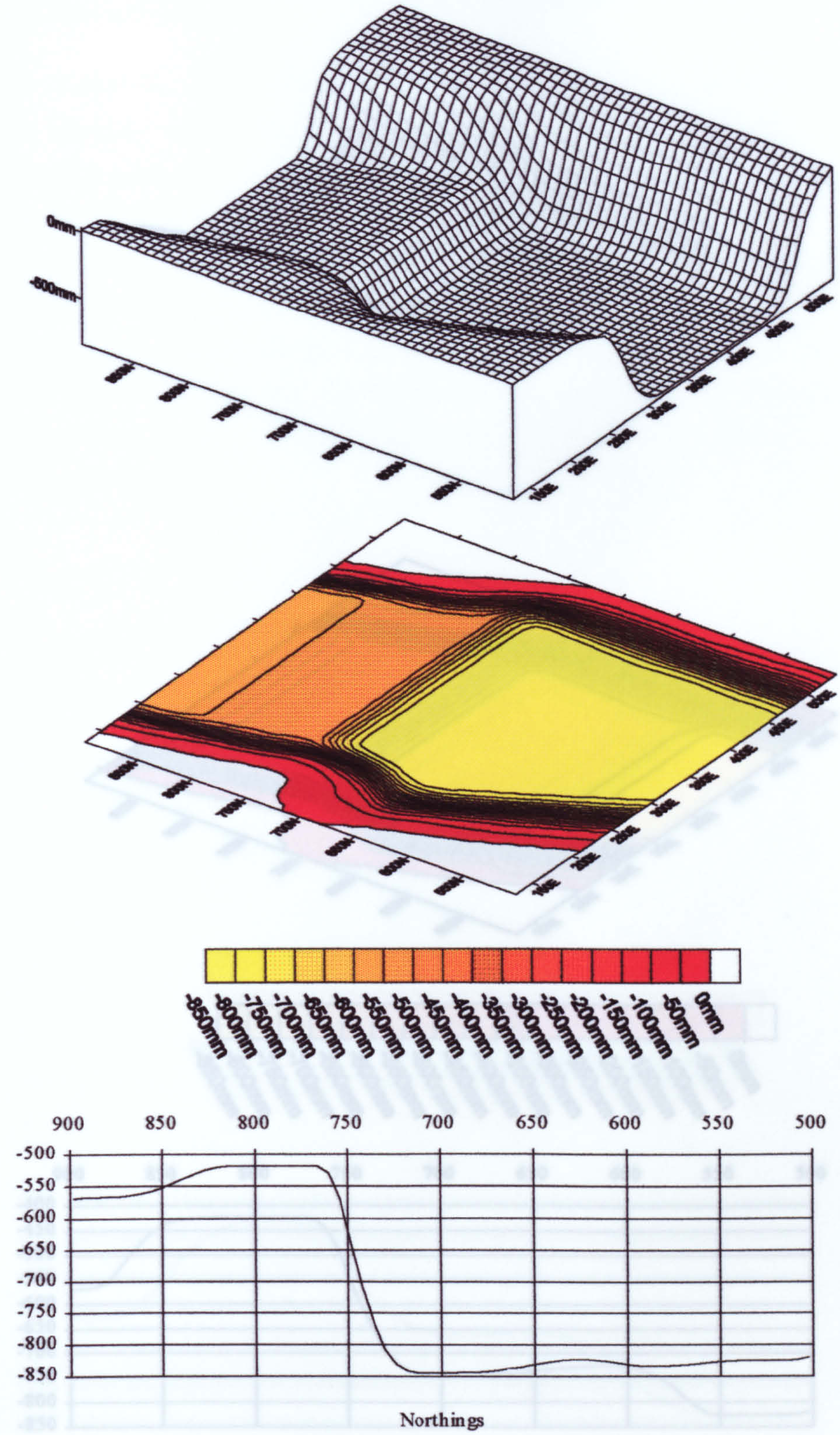


Figure 6.36. Predicted settlement for scenario 5, surcharge 2, test conditions as Table 6.1.

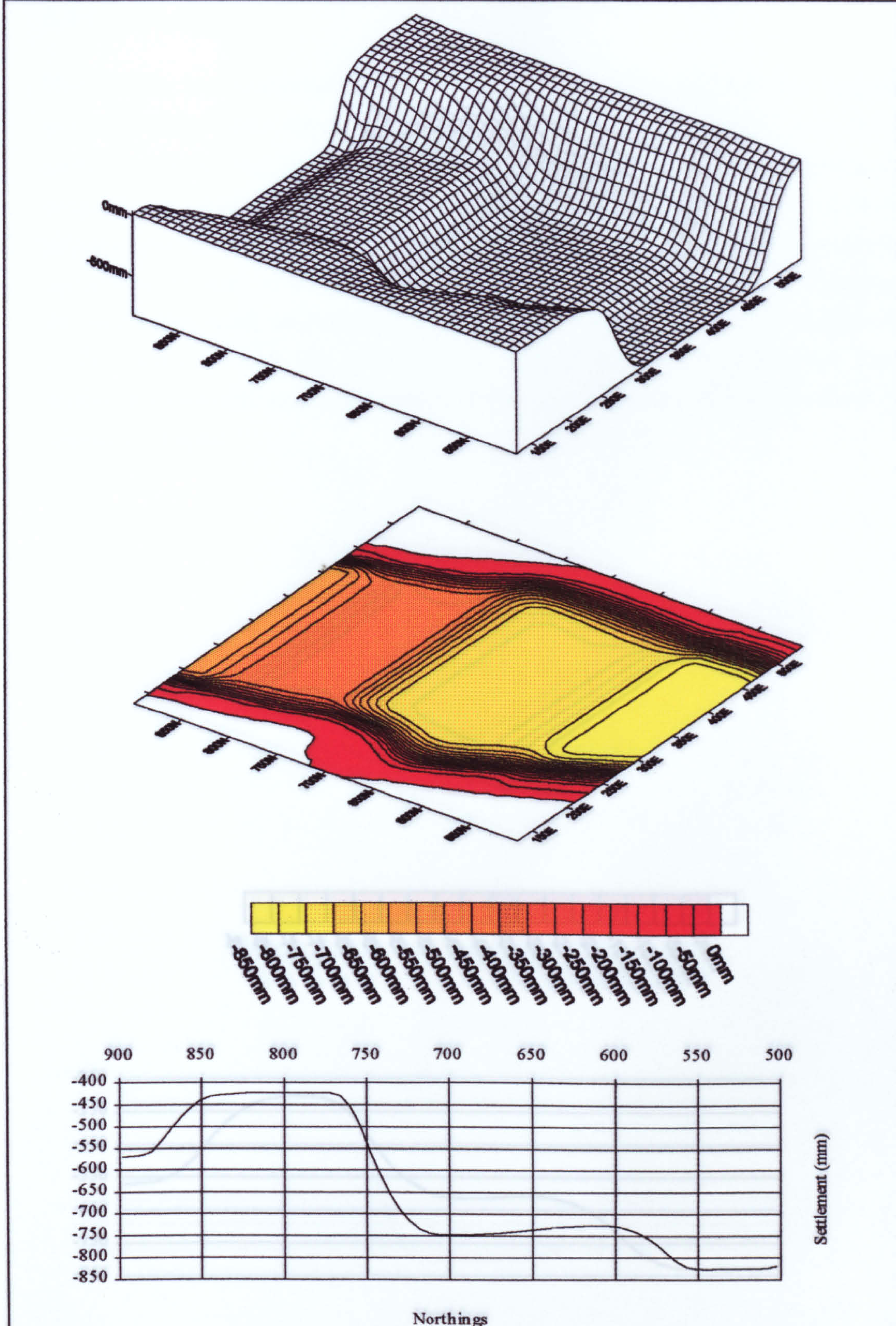


Figure 6.37. Predicted settlement for scenario 5, surcharge 3, test conditions as Table 6.1.

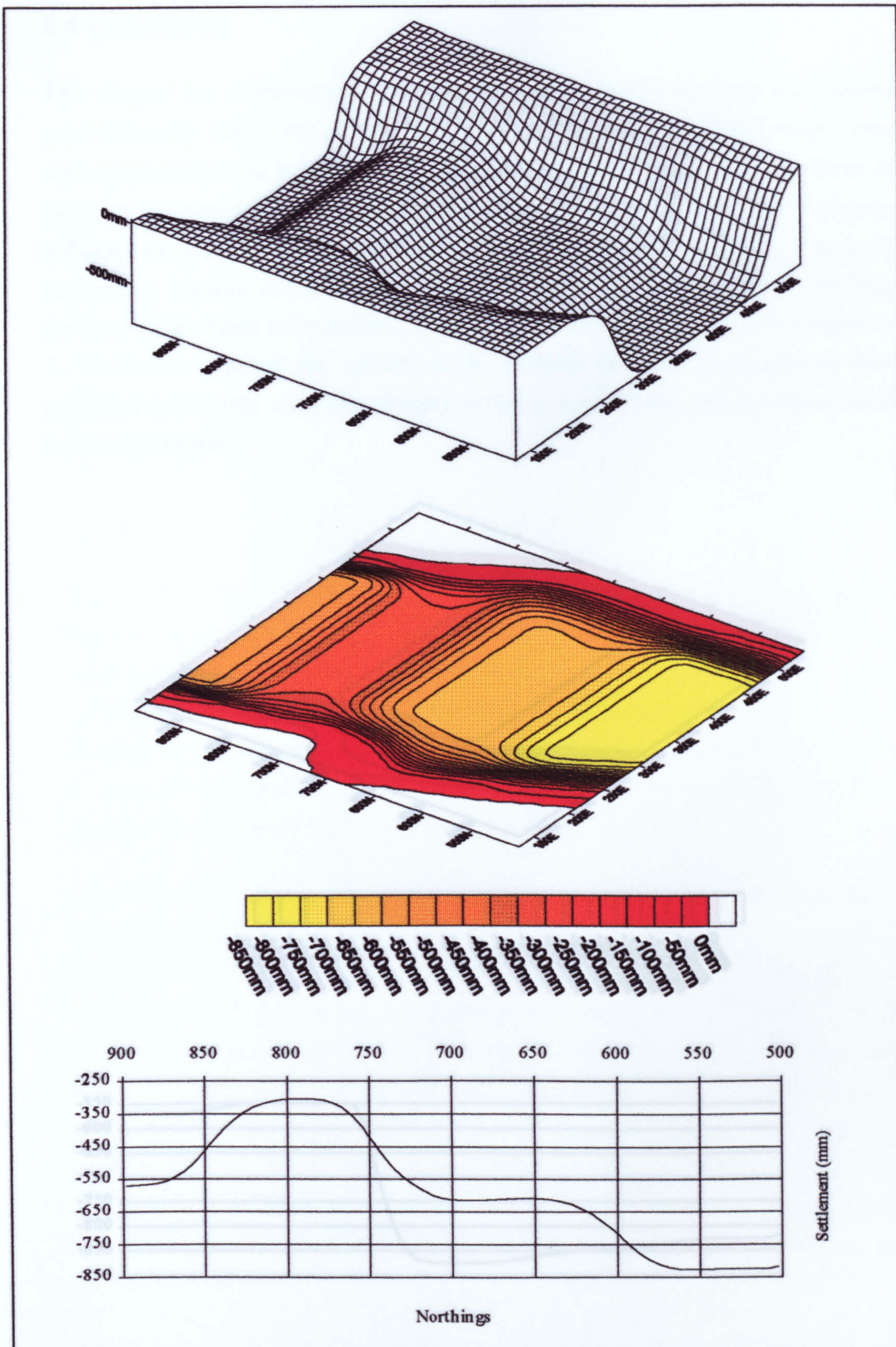


Figure 6.38. Predicted settlement for scenario 5, surcharge 4, test conditions as Table 6.1.

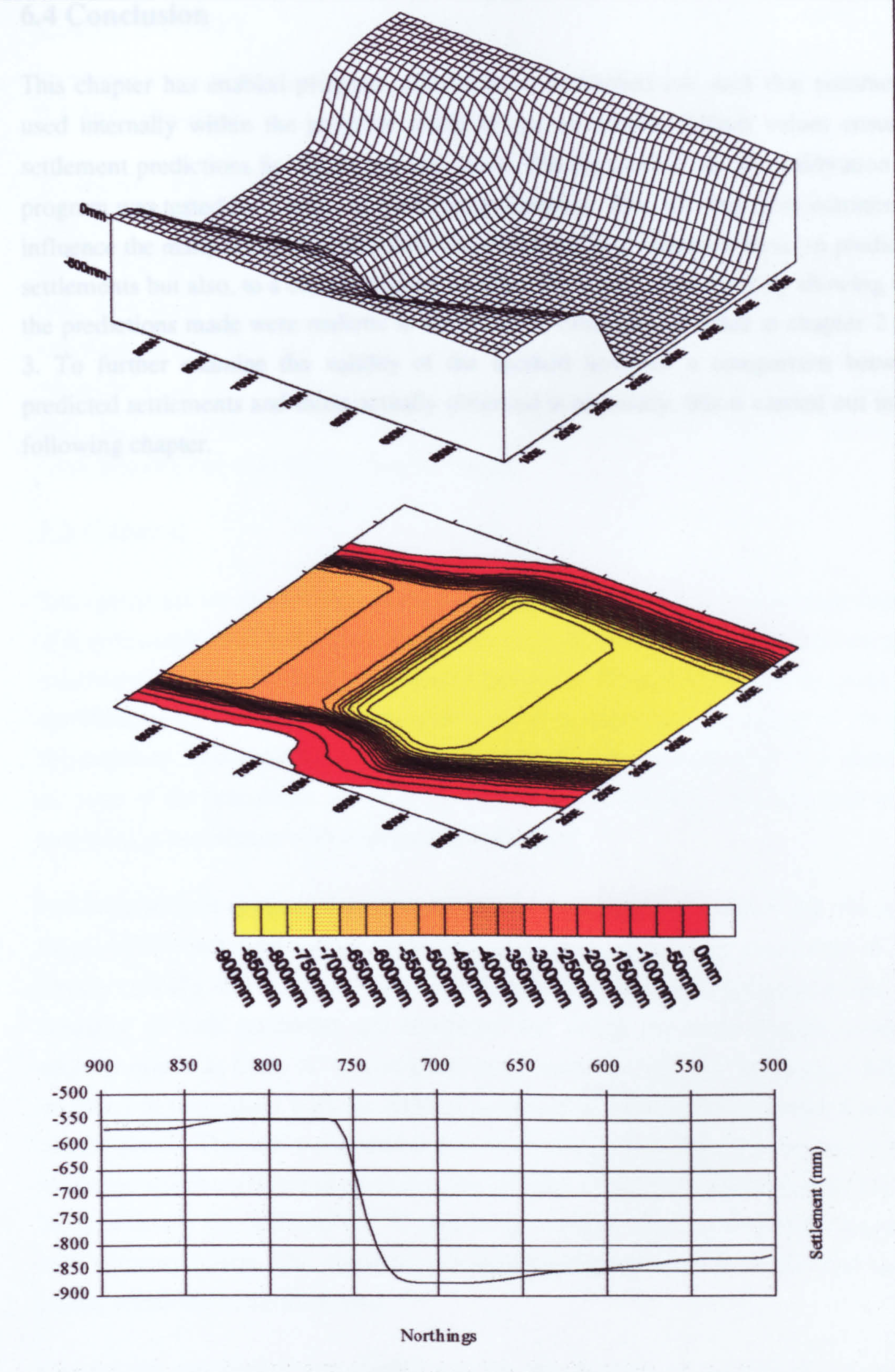


Figure 6.39. Predicted settlement for scenario 5, no surcharge, test conditions as Table 6.1.

6.4 Conclusion

This chapter has enabled program calibration to be carried out such that parameters used internally within the program could be set to suitable default values ensuring settlement predictions lie within realistic limits. Having carried out this calibration the program was tested for a range of backfilling scenarios. This not only demonstrates the influence the many parameters that describe the backfilling operation have on predicted settlements but also, to a certain degree, validate the proposed method by showing that the predictions made were realistic in terms of the observations made in chapter 2 and 3. To further examine the validity of the method however a comparison between predicted settlements and those actually observed is necessary; this is carried out in the following chapter.

CASE STUDY

7.1 Introduction

To examine the validity of the method of settlement prediction being proposed it is important to be able to make a comparison between predicted settlements and those actual observed at a real site. To achieve an accurate comparison, a complete set of data describing the test site is required. Unfortunately at non of the study sites examined above was this complete set of data made available. However a near complete set of data was made available from the site studied below and it is with this data that a comparison has been made. This case study site for reasons of confidentiality can only be described as Site A.

7.2 General

Site operations involved the excavation and extraction of coal to a maximum depth of 95m with overburden being placed back into the void in a controlled manner across the majority of the site. The backfill consisted predominantly of Coal Measures strata. The specification for reinstatement was drawn up with respect to the proposed after site development. This resulted, together with some late amendments due to a change in the scale of the operations, to the requirement that the majority of the backfill was to be placed in accordance with a method specification.

Problems became apparent however during the initial stages of backfilling due to the nature of the rock which upon excavation contained a significant proportion of large blocks, typically 600 to 800 mm in size, but some with dimensions as great as 2m. This consisted of both sandstone and mudstone and whilst the more durable sandstone could be placed as Class 1C 'coarse granular material' (rockfill) the mudstone could not be placed in accordance with the original specification without being broken down into smaller pieces. This led to a modified procedure being adopted for the placement of the mudstone which consisted of spreading the material in layers of approximately 800 mm loose thickness (as opposed to 275 mm in the original specification) and compacting with vibratory rollers. The mudstone comprised a large proportion of the total backfill placed within the controlled zone.

Compaction monitoring was carried out for the duration of backfill placement and consisted of both in situ and laboratory testing. Backfill density and air void values were determined from a 'Strata Gauge' nuclear density gauge (NDG) together with

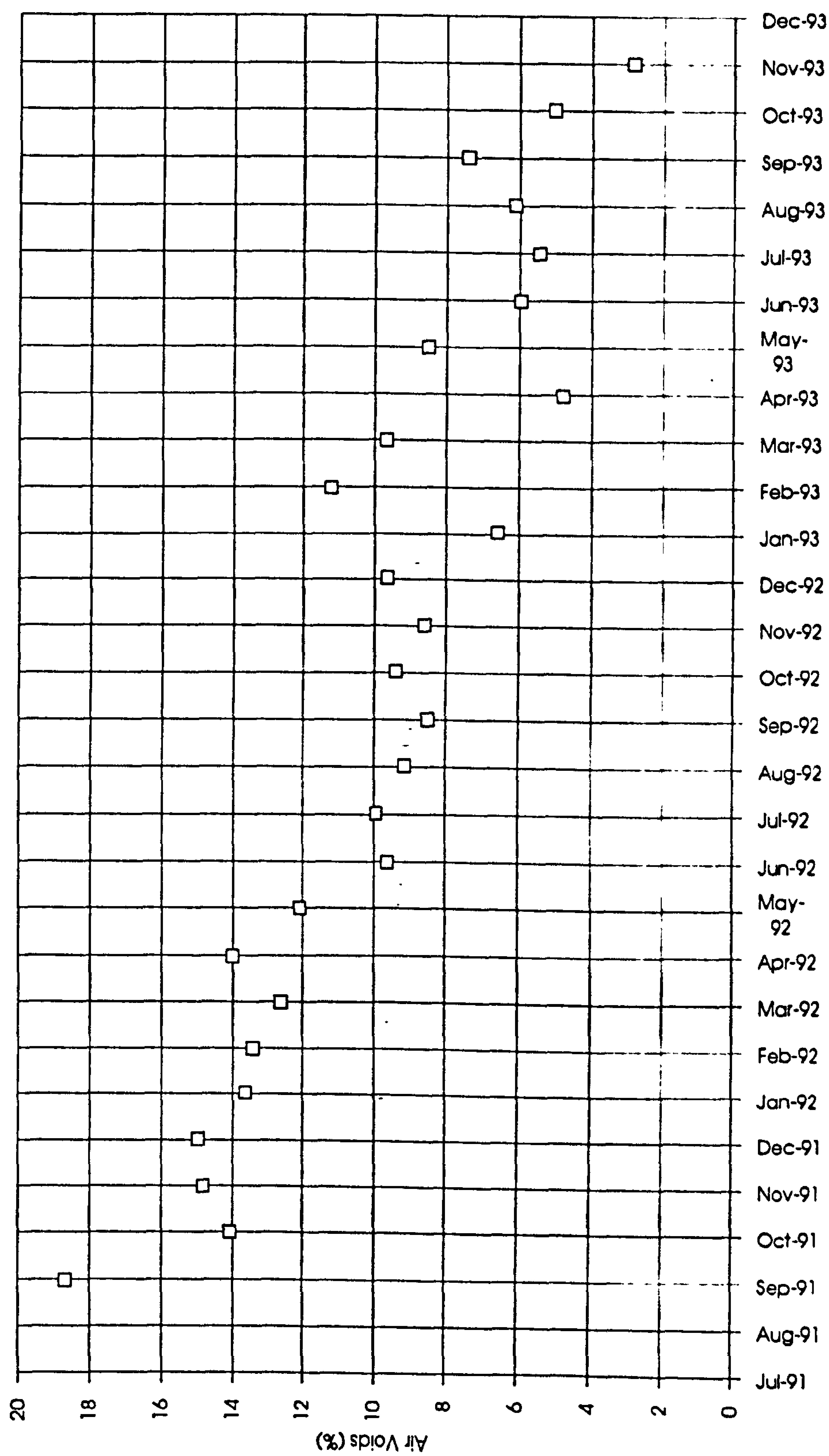


Figure 7.1. Site A compaction monitoring results - percentage air void values.

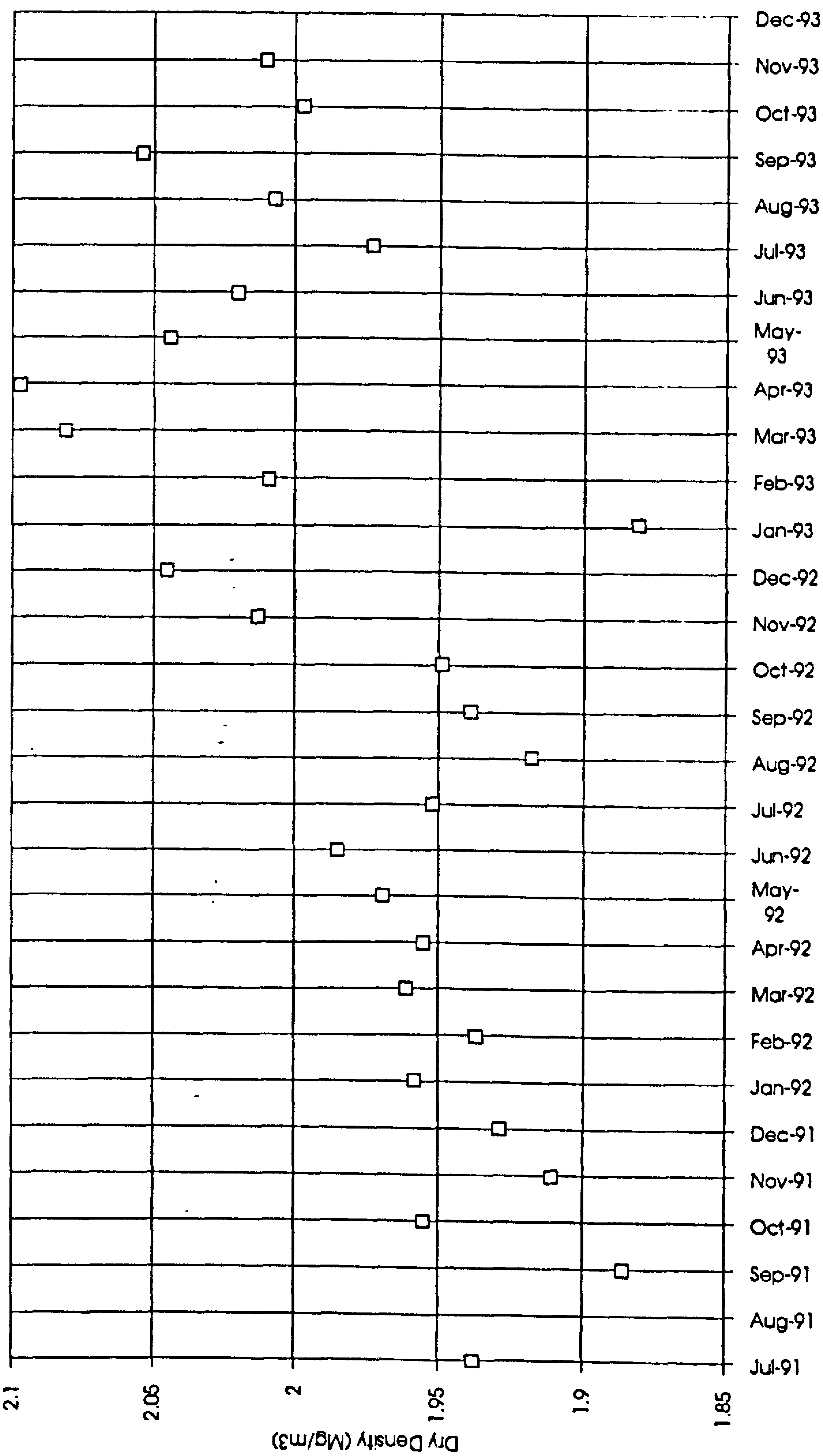


Figure 7.2. Site A compaction monitoring results - dry density values.

laboratory results to determine moisture content and particle density. The location of all NDG tests were recorded.

A total of 2480 in situ density test were carried out and the results of the monthly averages are presented in Figures 7.1 and 7.2. This demonstrates that whilst air void values are consistent with previously examined data from the study sites the dry density values appear to be unusually high considering the large proportion of the backfill placed as large fragments in thick layers. It is considered that this is as a consequence of the method by which these densities were measured. Due to the large fragment size the volume of material measured by the NDG would have been unrepresentative of the backfill as a whole. This would lead to measurements being biased towards the density of the actual rock fragments and as such unrealistically large.

Instrumentation upon completion of backfilling operations consisted of the installation of surface settlement markers, extensometers and piezometers. The extensometers and piezometers placed within the backfill and a proportion of the surface settlement markers were placed relatively recently and as such provide little information for purposes of comparison. However a significant number of surface settlement markers placed in a central region of the site together with groundwater information obtained from piezometers located around the periphery of the site enable a comparison between predicted and observed settlements to be made.

7.3 Backfill Modelling

To represent the backfilling operation in a computer model that will enable settlement predictions to be made using OBSett, the backfill is split up into monthly blocks. These are defined by DTMs that represent the upper surface of the blocks. The most accurate method by which these surfaces could be defined would be from regular surveying of the backfill surface as compaction operations commenced. This data was however not available for this site, but as all in-situ compaction tests where surveyed a picture of the backfilling surface could be made up especially beneath the line of the proposed development, section A-A', as represented in Figure 7.3.

Having generated the cross section shown in Figure 7.3, to produce surfaces for the monthly blocks this section has to be extended into the 3rd dimension. This was achieved with the use of AutoCAD and a DTM of the base of the mining operations. The intersection between a given block, the pit base and adjacent blocks can be traced to generate lines that enclose each block in plan view. These lines are represented by 3D polylines within AutoCAD such that each point on the line has an eastings,

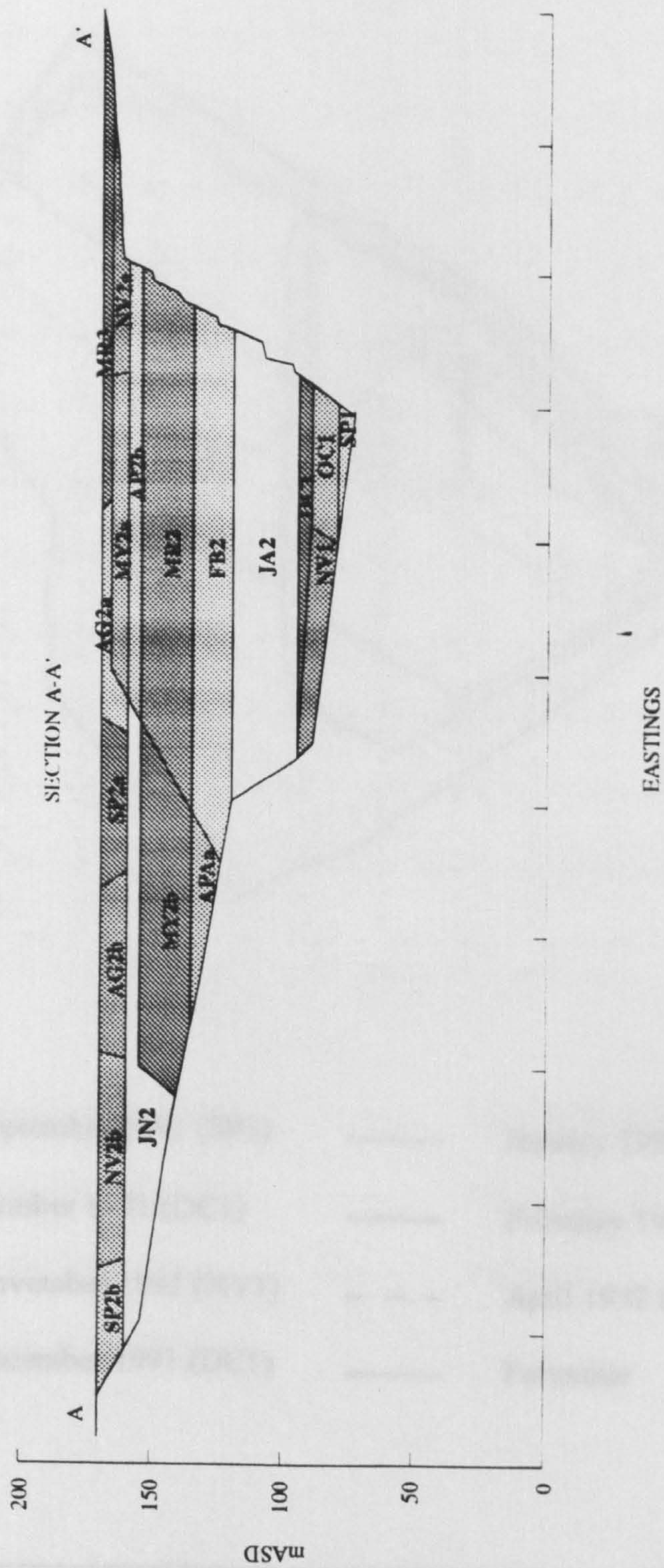


Figure 7.3. Site A, cross section A-A', showing the position of the backfilling blocks SP1 to MR3.

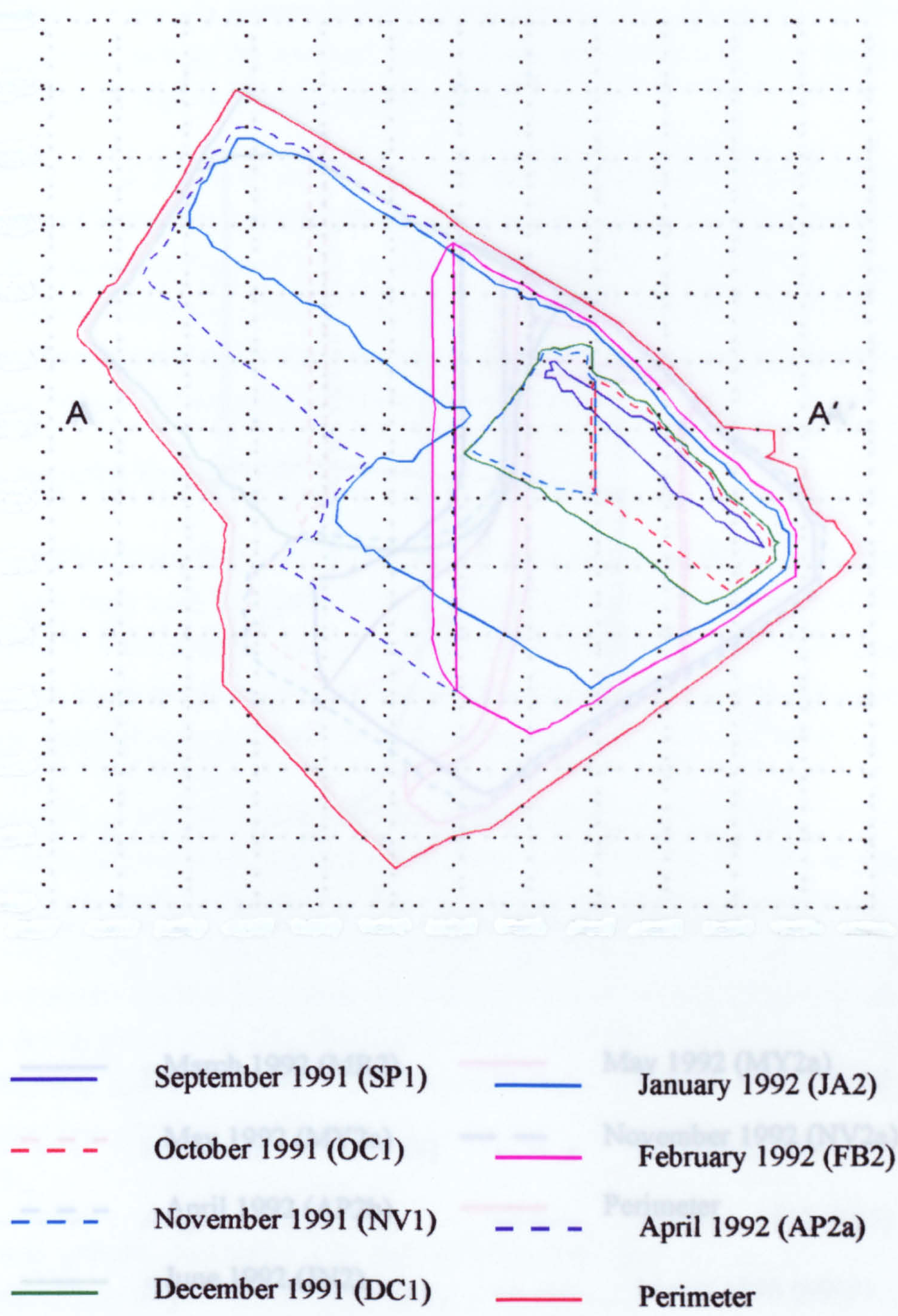


Figure 7.4. Polylines describing backfill blocks SP1 to AP2a, viewed in plan.

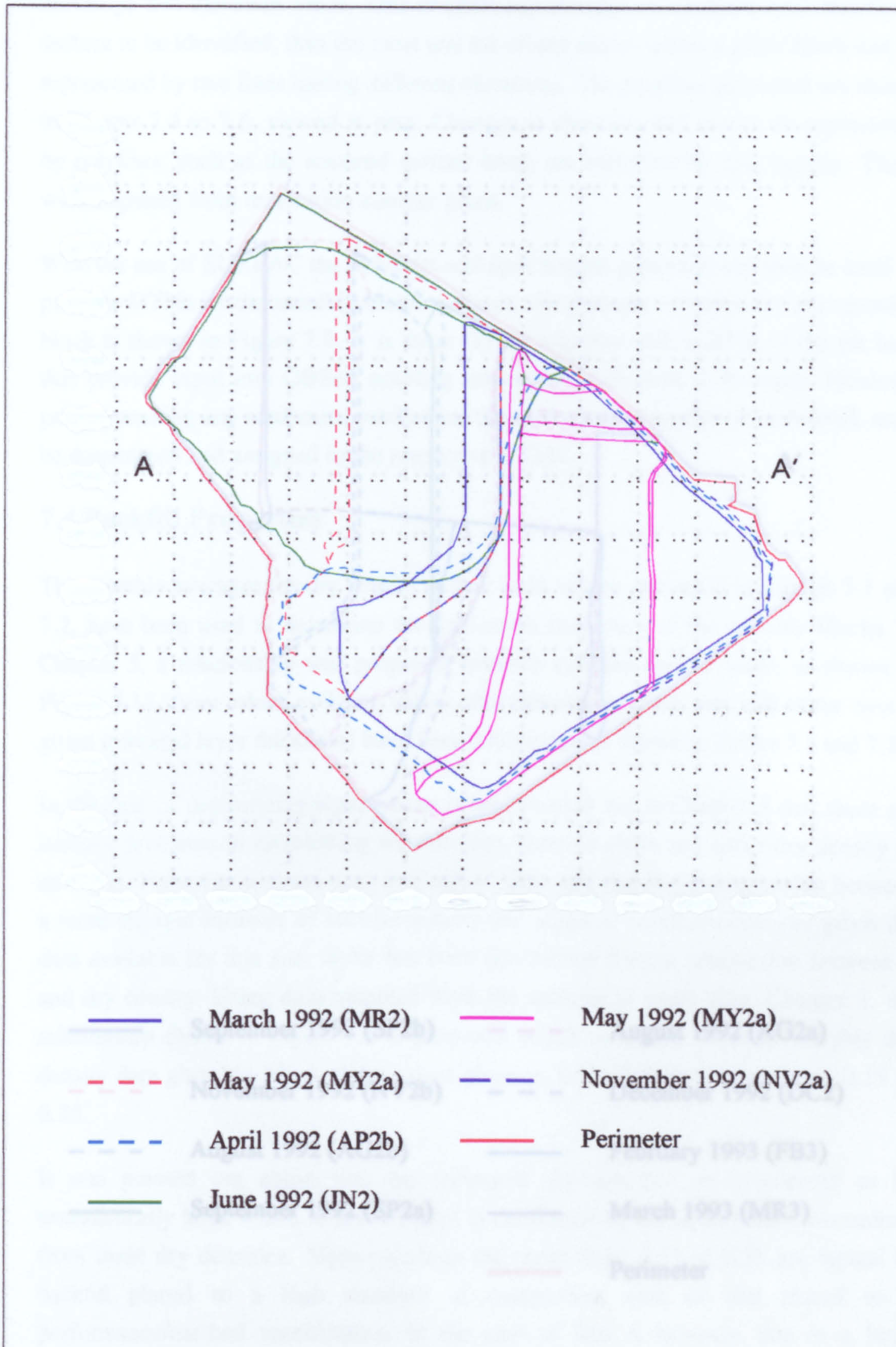


Figure 7.5. Polylines describing backfill blocks MR2 to NV2a, viewed in plan.

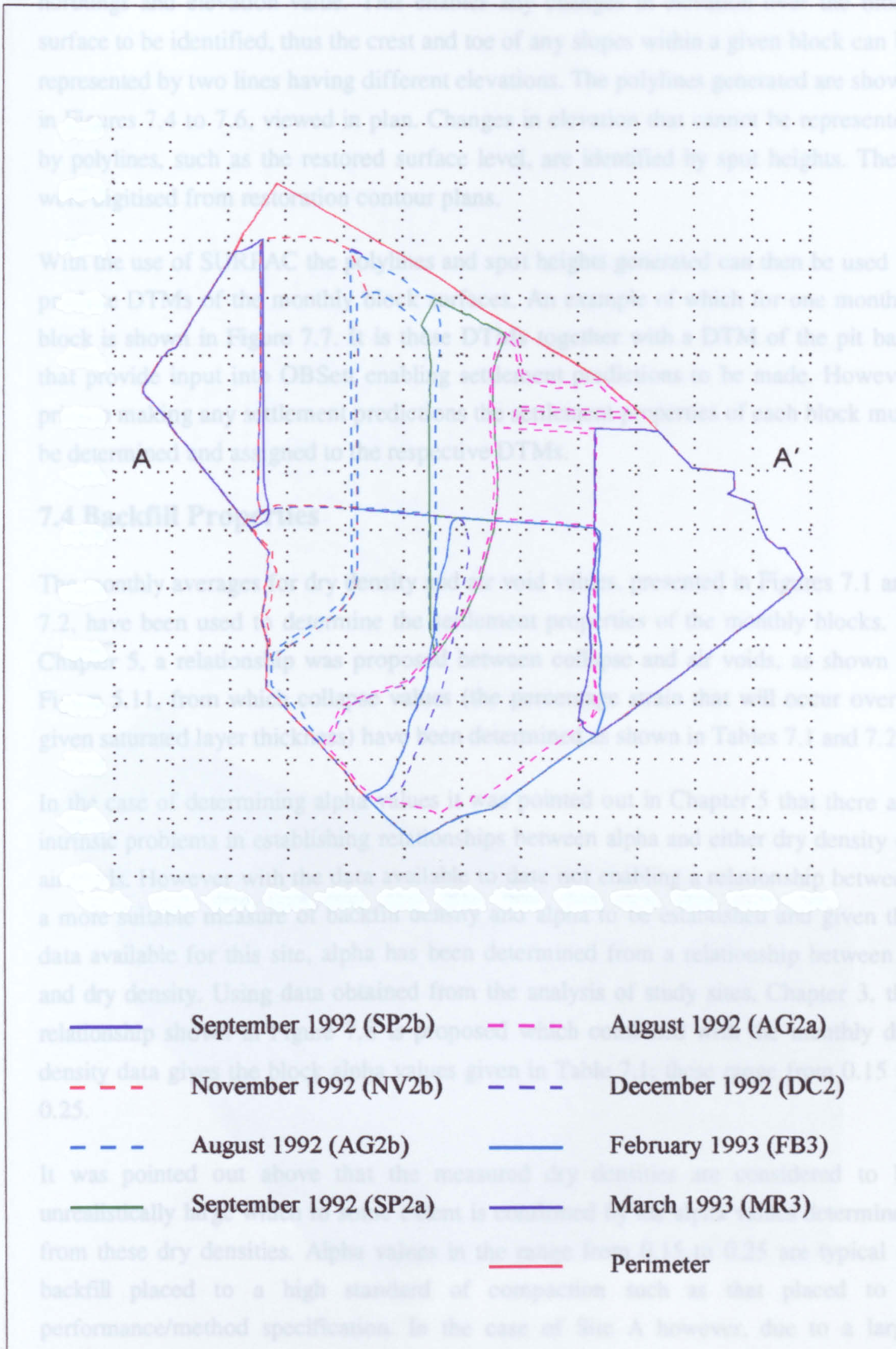


Figure 7.6. Polylines describing backfill blocks SP2a to MR3, viewed in plan.

northings and elevation value. This enables any changes in elevation over the block surface to be identified, thus the crest and toe of any slopes within a given block can be represented by two lines having different elevations. The polylines generated are shown in Figures 7.4 to 7.6, viewed in plan. Changes in elevation that cannot be represented by polylines, such as the restored surface level, are identified by spot heights. These were digitised from restoration contour plans.

With the use of SURPAC the polylines and spot heights generated can then be used to produce DTMs of the monthly block surfaces. An example of which for one monthly block is shown in Figure 7.7. It is these DTMs together with a DTM of the pit base that provide input into OBSett enabling settlement predictions to be made. However prior to making any settlement predictions the settlement properties of each block must be determined and assigned to the respective DTMs.

7.4 Backfill Properties

The monthly averages for dry density and air void values, presented in Figures 7.1 and 7.2, have been used to determine the settlement properties of the monthly blocks. In Chapter 5, a relationship was proposed between collapse and air voids, as shown in Figure 5.11, from which collapse values (the percentage strain that will occur over a given saturated layer thickness) have been determined as shown in Tables 7.1 and 7.2.

In the case of determining alpha values it was pointed out in Chapter 5 that there are intrinsic problems in establishing relationships between alpha and either dry density or air voids. However with the data available to date not enabling a relationship between a more suitable measure of backfill density and alpha to be established and given the data available for this site, alpha has been determined from a relationship between it and dry density. Using data obtained from the analysis of study sites, Chapter 3, the relationship shown in Figure 7.8 is proposed which combined with the monthly dry density data gives the block alpha values given in Table 7.1; these range from 0.15 to 0.25.

It was pointed out above that the measured dry densities are considered to be unrealistically large which to some extent is confirmed by the alpha values determined from these dry densities. Alpha values in the range from 0.15 to 0.25 are typical of backfill placed to a high standard of compaction such as that placed to a performance/method specification. In the case of Site A however, due to a large proportion of the backfill consisting of large rock fragments placed in thick layers, such a high level of compaction, as indicated by these alpha values, is considered unlikely. This is borne out by the air voids values measured which are generally in the region of

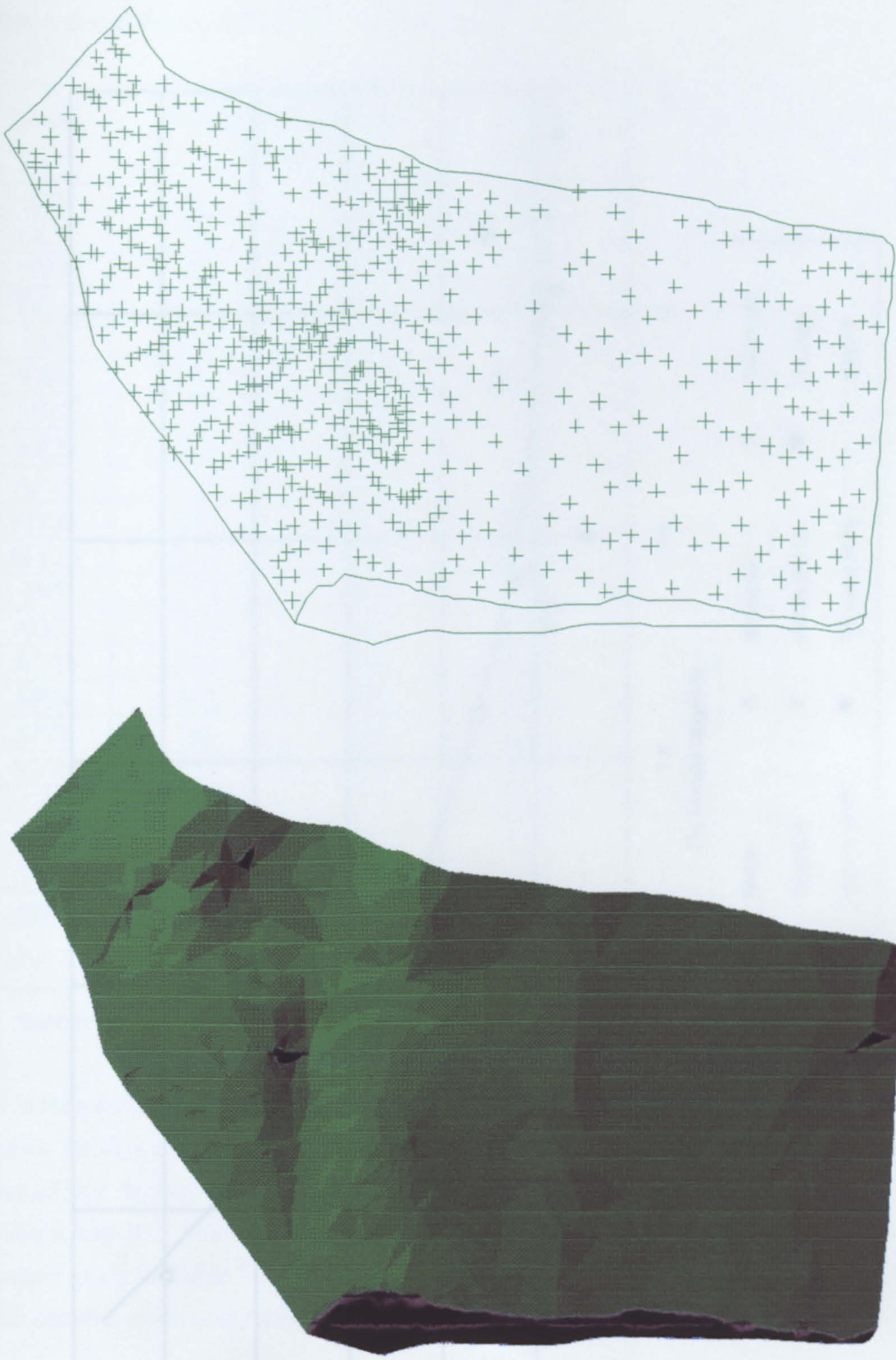


Figure 7.7. The generation of a block surface DTM from polylines and spot heights.

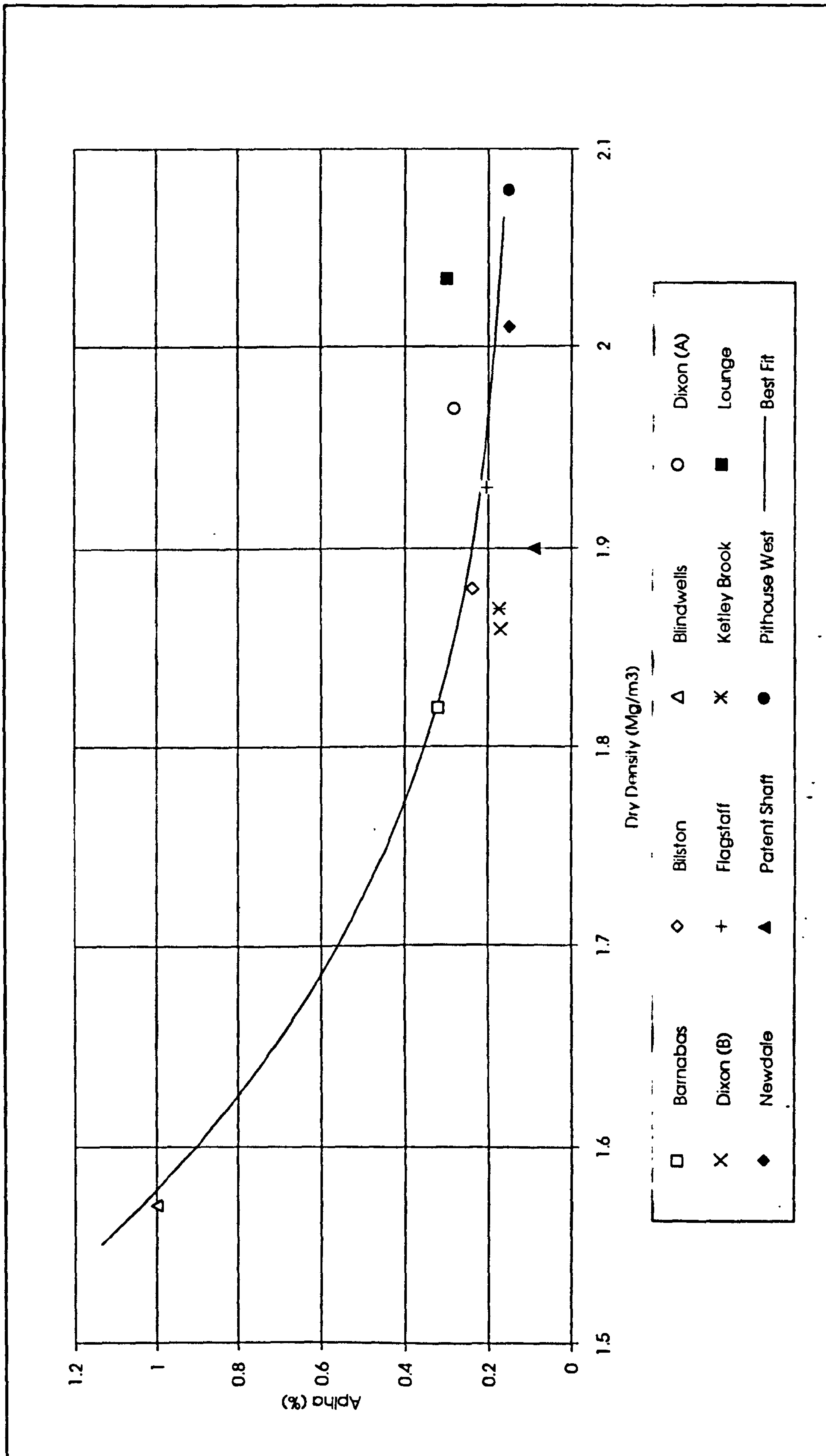


Figure 7.8. Relationship between backfill dry density and the creep compression parameter (alpha).

10% plus, values more typical of backfill compacted to a lesser degree than either performance or method compaction i.e. more typical of the compaction carried out at Site A due to the modifications made to the specification.

Block	Period of Placement (days) [1]		Alpha (%)	Collapse (%)	Unit Weight (kN/m³)
SP1	0	29	0.25	2.05	20
OC1	30	60	0.21	1.01	20
NV1	61	90	0.23	1.15	20
DC1	91	121	0.22	1.18	20
JA2	122	152	0.20	0.93	20
FB2	153	181	0.22	0.90	20
MR2	182	212	0.20	0.77	20
AP2a	213	242	0.21	1.00	20
AP2b	213	242	0.21	1.00	20
MY2a	243	273	0.20	0.68	20
MY2b	243	273	0.20	0.68	20
JN2	274	303	0.19	0.37	20
AG2a	335	365	0.23	0.32	20
AG2b	335	365	0.23	0.32	20
SP2a	366	395	0.21	0.26	20
SP2b	366	395	0.21	0.26	20
NV2a	427	456	0.18	0.27	20
NV2b	427	456	0.18	0.27	20
DC2	457	487	0.17	0.37	20
FB3	519	546	0.18	0.56	20
MR3	547	577	0.16	0.38	20

Table 7.1. Block placement periods and properties for prediction 1.

[1] Number of days from the start of backfilling operations

It is therefore considered that to get a better estimate of alpha for each monthly backfill block the dry density measurements require adjustment to bring them in line with the actual dry density of the blocks as a whole. This is rather crudely carried out by subtracting 0.2 from the dry density measurements which gives the adjusted alpha values given in Table 7.2. These are considered more typical considering the nature of the backfill when compared to the study site data.

The values given in Table 7.1 are used in prediction 1, those in Table 7.2 for predictions 2 and 3.

Block	Period of Placement (days)[1]		Alpha (%)	Collapse (%)	Unit Weight (kN/m³)
SP1	0	29	0.60	2.05	20
OC1	30	60	0.43	1.01	20
NV1	61	90	0.53	1.15	20
DC1	91	121	0.49	1.18	20
JA2[2]	122	152	0.43	0.93 / 1.10[3]	20
FB2[2]	153	181	0.47	0.90 / 1.30[3]	20
MR2	182	212	0.42	0.77	20
AP2a[2]	213	242	0.43	1.00 / 1.25[3]	20
AP2b	213	242	0.43	1.00	20
MY2a	243	273	0.40	0.68	20
MY2b	243	273	0.40	0.68	20
JN2	274	303	0.38	0.37	20
AG2a	335	365	0.51	0.32	20
AG2b	335	365	0.51	0.32	20
SP2a	366	395	0.47	0.26	20
SP2b	366	395	0.47	0.26	20
NV2a	427	456	0.33	0.27	20
NV2b	427	456	0.33	0.27	20
DC2	457	487	0.29	0.37	20
FB3	519	546	0.34	0.56	20
MR3	547	577	0.26	0.38	20

Table 7.2. Block placement periods and properties for predictions 2 and 3 (see note 3).

[1] Number of days from the start of backfilling operations

[2] Blocks that become saturated by the recovery of the groundwater table during the prediction period.

[3] Prediction 3 collapse strain values.

7.5 Groundwater

Having defined the timing of placement and the properties of the blocks making up the backfill, the next stage is to define the rise in the groundwater table such that collapse strains can be calculated. During, prior and after the mining and backfilling operation, the groundwater table has been monitored at piezometers GT1A, GT1B, GT1C, GT2A, GT2B, GT2C and GT2D, the results of which are shown in Figure 7.9 with locations given in Figure 7.10. The results indicate that upon completion of the majority of the backfill placement, November 1992, the water table was initially drawn down then rose over a period of some 12 months to within approximately 2 metres of its original level. Piezometers BS01 to BS08, installed at a later date (June 1993),

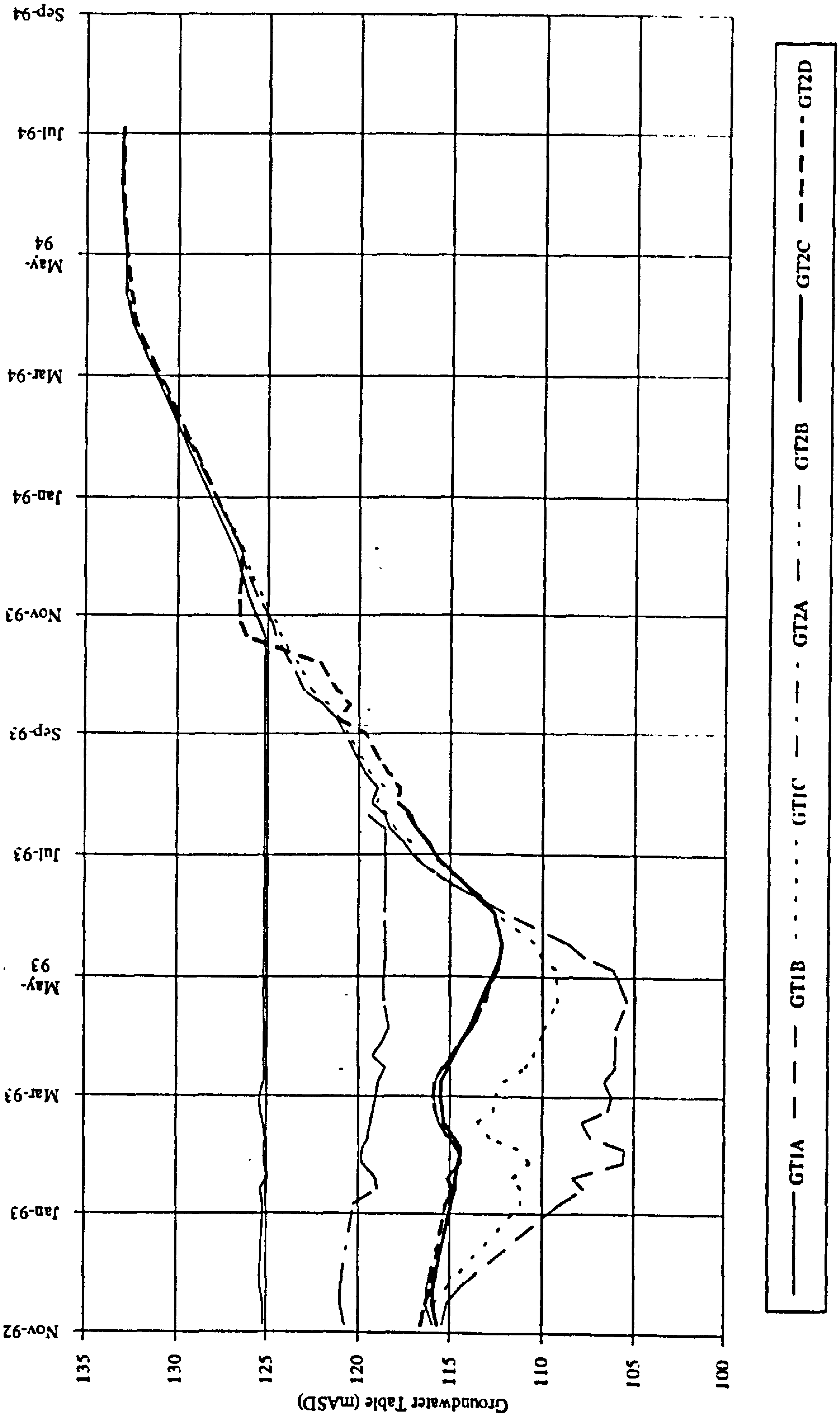


Figure 7.9. Site A groundwater table monitoring results.

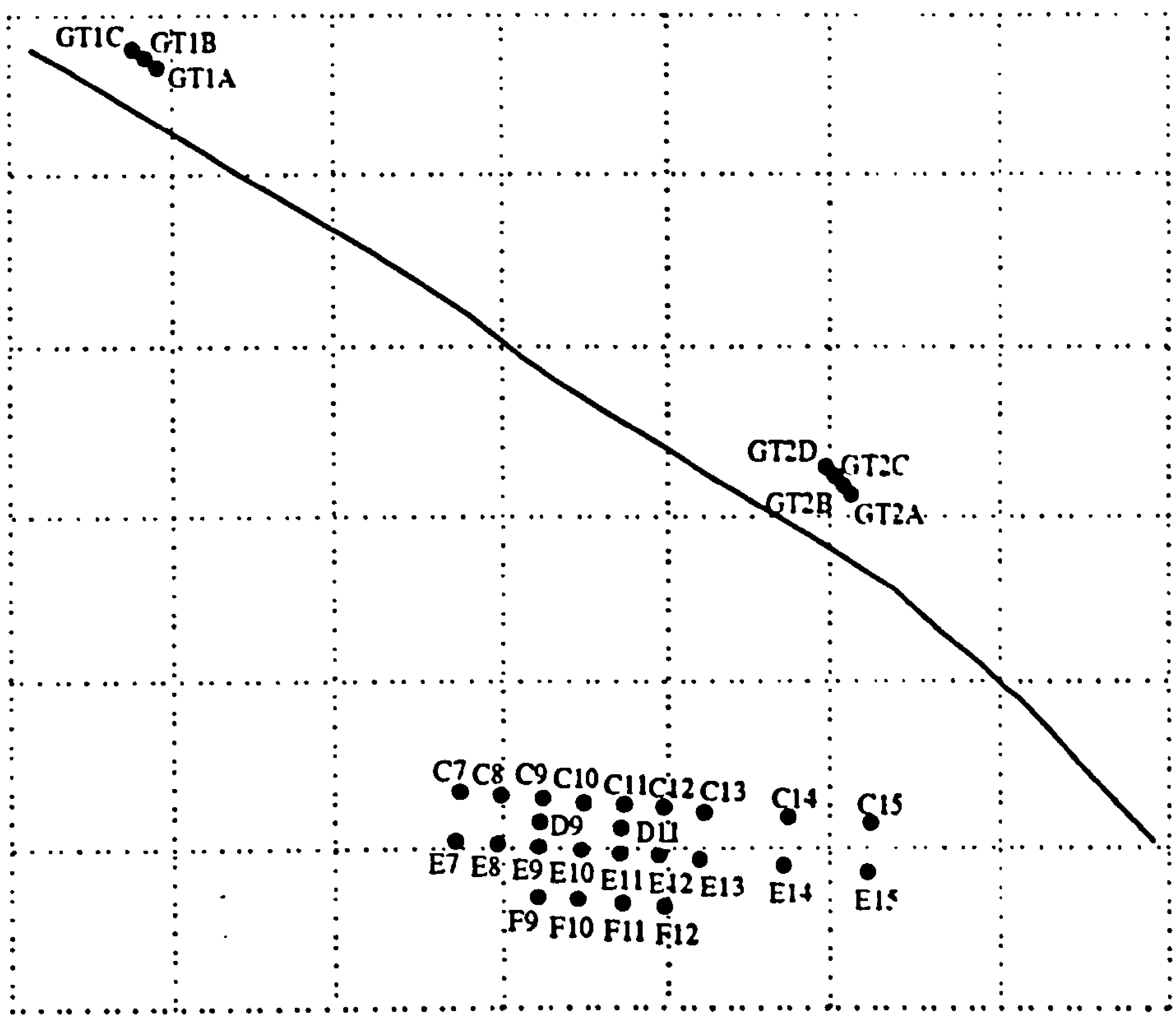


Figure 7.10. Site A piezometer and settlement marker locations.

indicate a similar behaviour with the water table having re-established itself close to its original level by June 1994.

These monitored water levels were however all obtained from instrumentation placed along the northern periphery of the site and as such only give a possible indication of how the groundwater table rose through the backfill; to determine collapse strains it is this information that is required. Piezometers in the main body of the backfill have only been relatively recently installed thus insufficient data is available from them to determine the rise through the fill. They do however indicate a similar level once monitoring commenced as those along the site periphery.

The groundwater table at any time will be defined within OBSett, by a DTM representing its upper surface. As insufficient data is available within the main body of the fill it will be assumed that the level of this surface will be the same across the whole of the site and that it will follow closely those measured at the periphery. Figure 7.11

shows two groundwater tables for which collapse strain will be calculated. GWT1 is the same as that which was measured at GT1B whilst GWT2 is an assumed water table based upon measurements from the BS series of piezometers. GWT1 is used for predictions 1 and 2, whilst GWT2 is used for prediction 3.

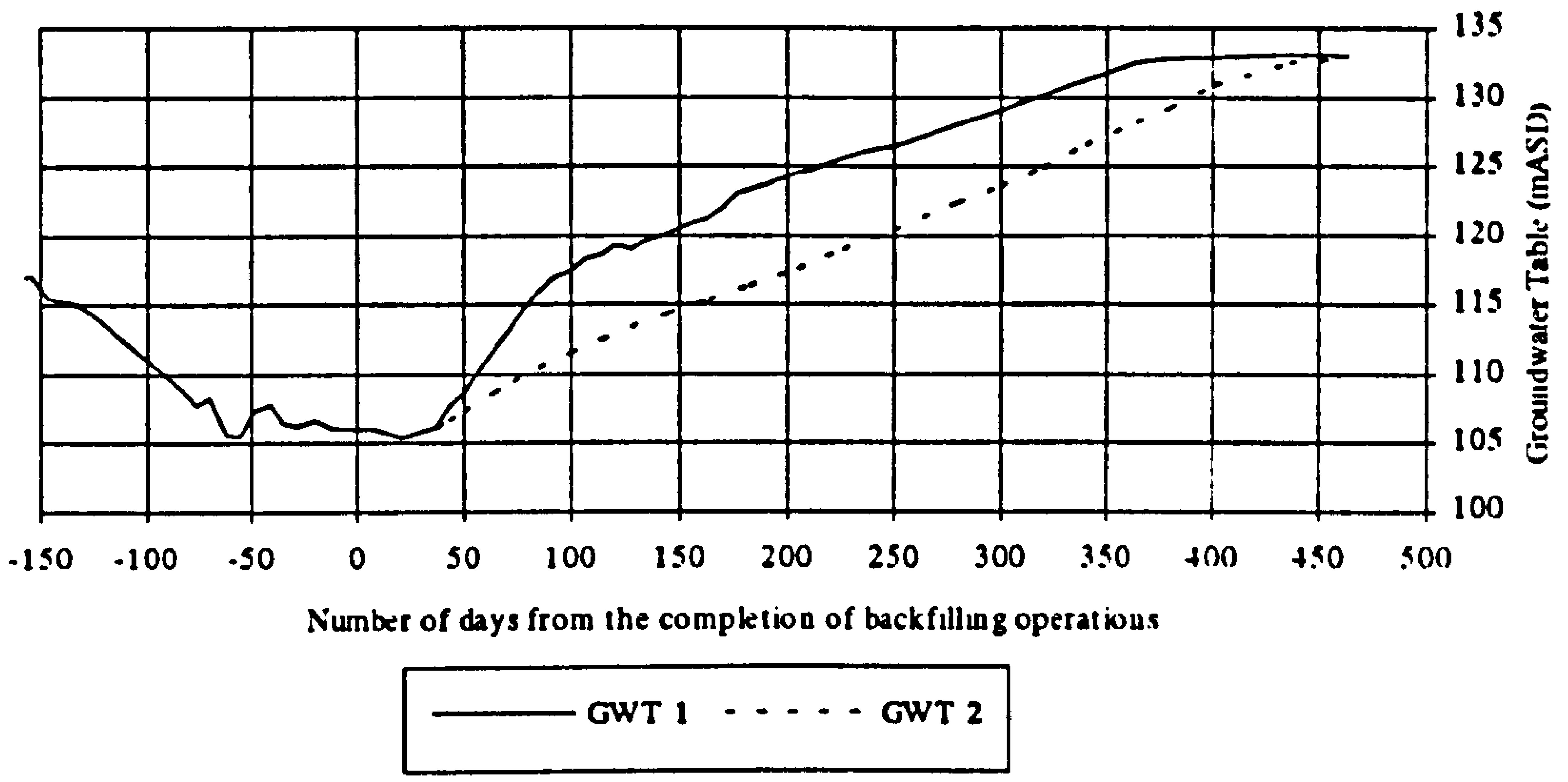


Figure 7.11. Groundwater tables GWT1 and GWT2.

7.6 Settlement Prediction

Settlement monitoring has been carried out at Site A since 12th November 1992 at locations C7 to C15, D9 and D11, E7 to E15 and F9 to F12 (Figure 7.10). It is at these same locations and over the same period that settlement predictions using OBSett have been carried out, enabling comparisons to be made between predicted and actual settlements. The results of 3 different predictions together with the actual settlements monitored are shown in Figures 7.12 to 7.19.

The comparison between prediction 1 and the actual settlements indicate that the alpha values based upon actual dry density measurements were too low as shown by the predicted creep component of the settlement resulting in an average under estimate, at the end of the prediction period, of some 35%. This was to be expected for the reasons pointed out above.

Prediction 2 which uses the alpha values based upon modified dry density measurements gives a much better estimate of settlement. Estimates are on average in the region of 10% of measured settlements.

In a large proportion of the opencast backfilling operations a period of settlement/ groundwater table monitoring is carried out upon completion of backfilling. This enables the settlement behaviour of the backfill to be examined, leading to the ability to modify any initial settlement predictions in the light of actual results. Prediction 3 represents such a modification to the prediction. It is felt that the magnitude of collapse was slightly under estimated by prediction 2 thus the collapse strain properties of the material that became saturated were slightly increased (Table 7.2). Also the timing of collapse has been modified for prediction 3 by using GWT2 as opposed to GWT1. This does however only slightly increase the average accuracy of the predictions made over those of prediction 2

7.7 Conclusion

This case study has demonstrated that given the amount and level of detail of compaction and groundwater table monitoring data, made available from Site A, predictions for backfill settlement can be made with the use of OBSett that lie within the region of 10% of those actually recorded. Where such data is only partially or even not available, clearly such levels of accuracy would not be obtainable due to the greater number of assumptions made in backfill modelling and assigning appropriate settlement properties to the backfill. However it is considered that the amount and level of detail of data made available from Site A is not atypical of that which would be available from any controlled opencast backfilling operation.

Where detail may be lacking a scheme of settlement and groundwater monitoring, carried out immediately after backfilling, can be used to modify the backfill model used by OBSett to produce better estimates more in line with those being monitored. This is demonstrated by prediction 3 which due to the level of detail of the compaction and groundwater table monitoring data makes only a slight improvement on the settlements estimated by prediction 2.

The settlement contours in Figures 7.20 and 7.21 demonstrate the type of output available from OBSett which can be used to provide valuable information in the design of after site development.

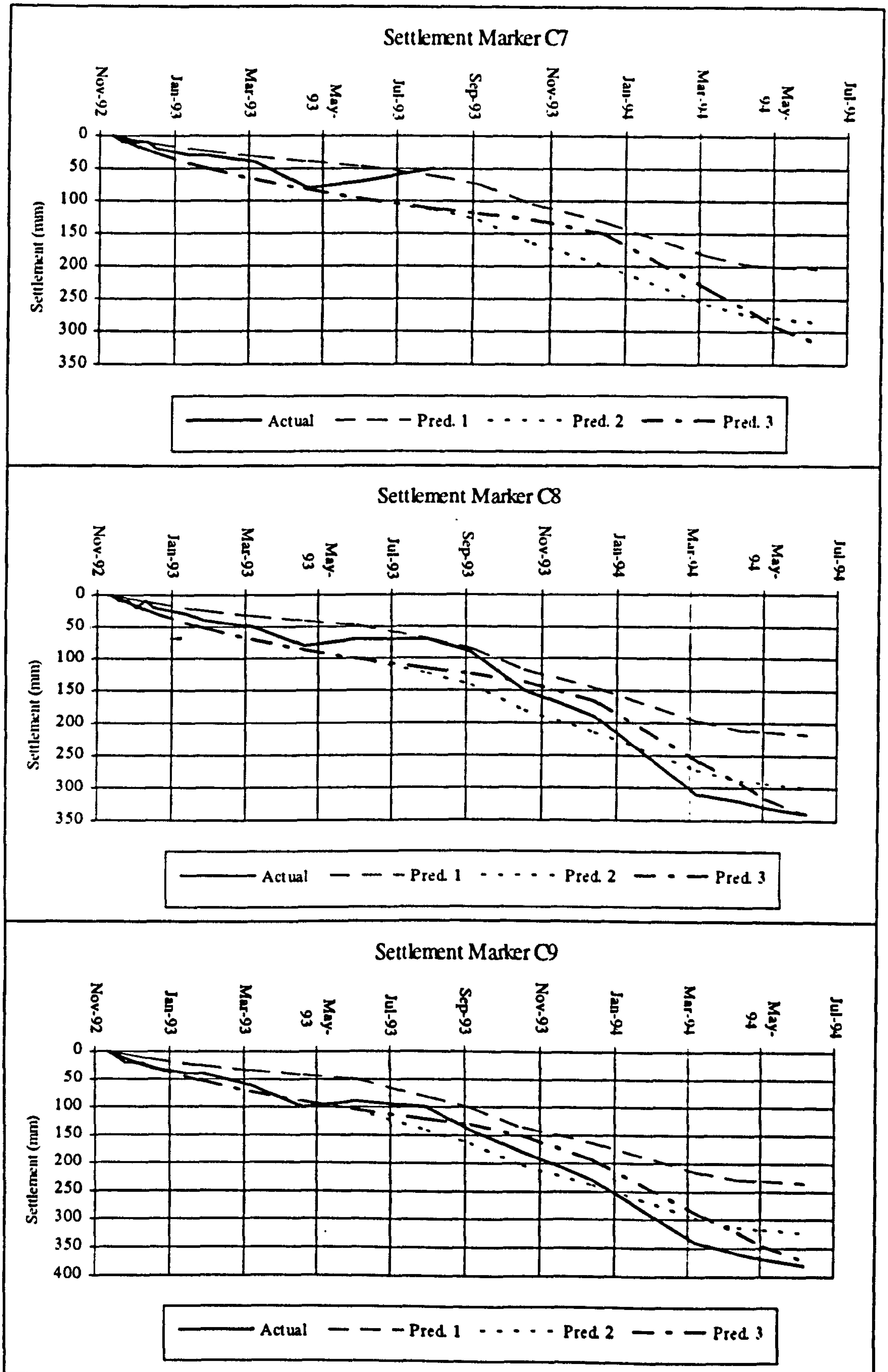


Figure 7.12. Actual and predicted settlements at markers C7, C8 and C9

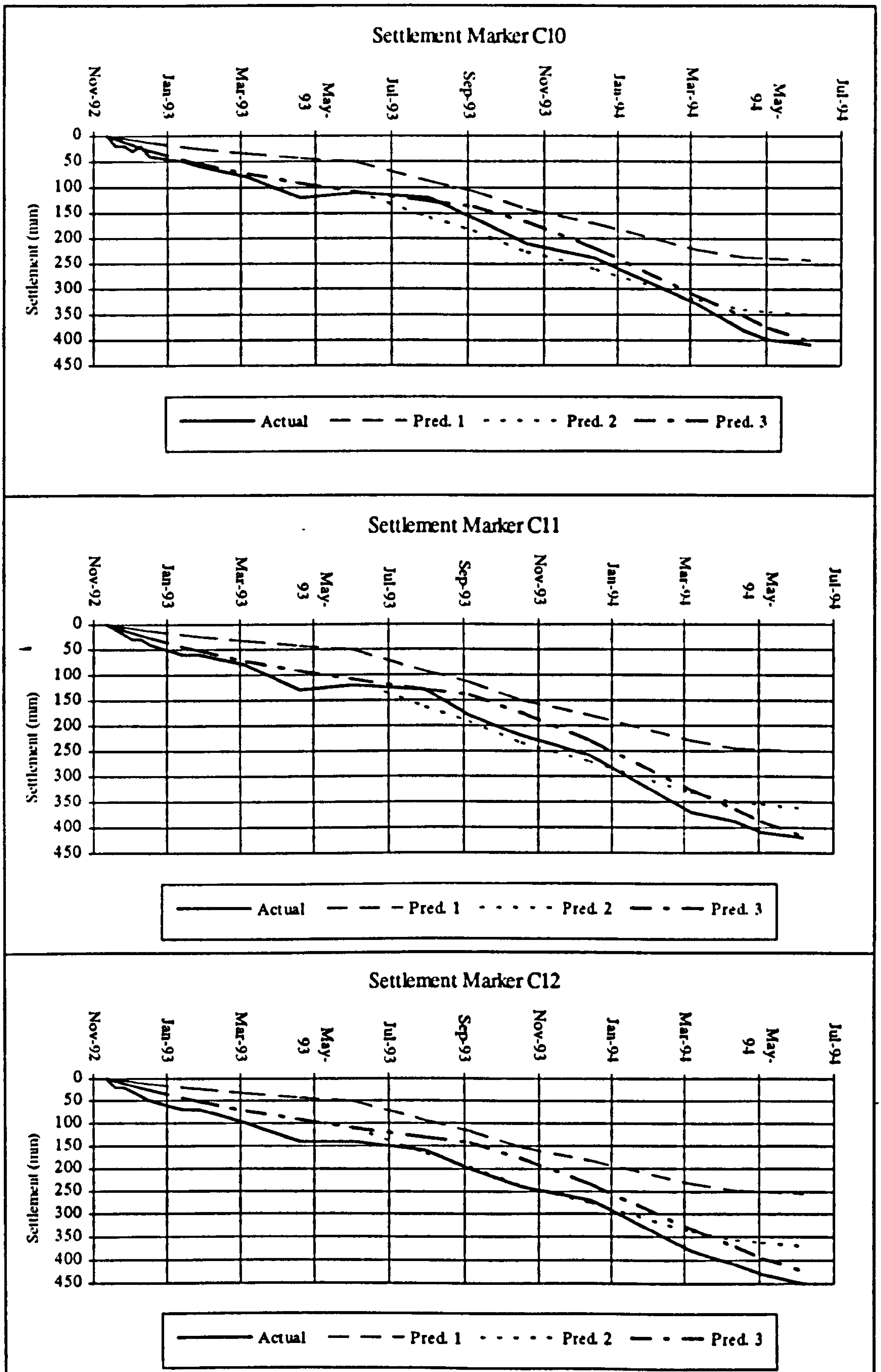


Figure 7.13. Actual and predicted settlements at markers C10, C11 and C12

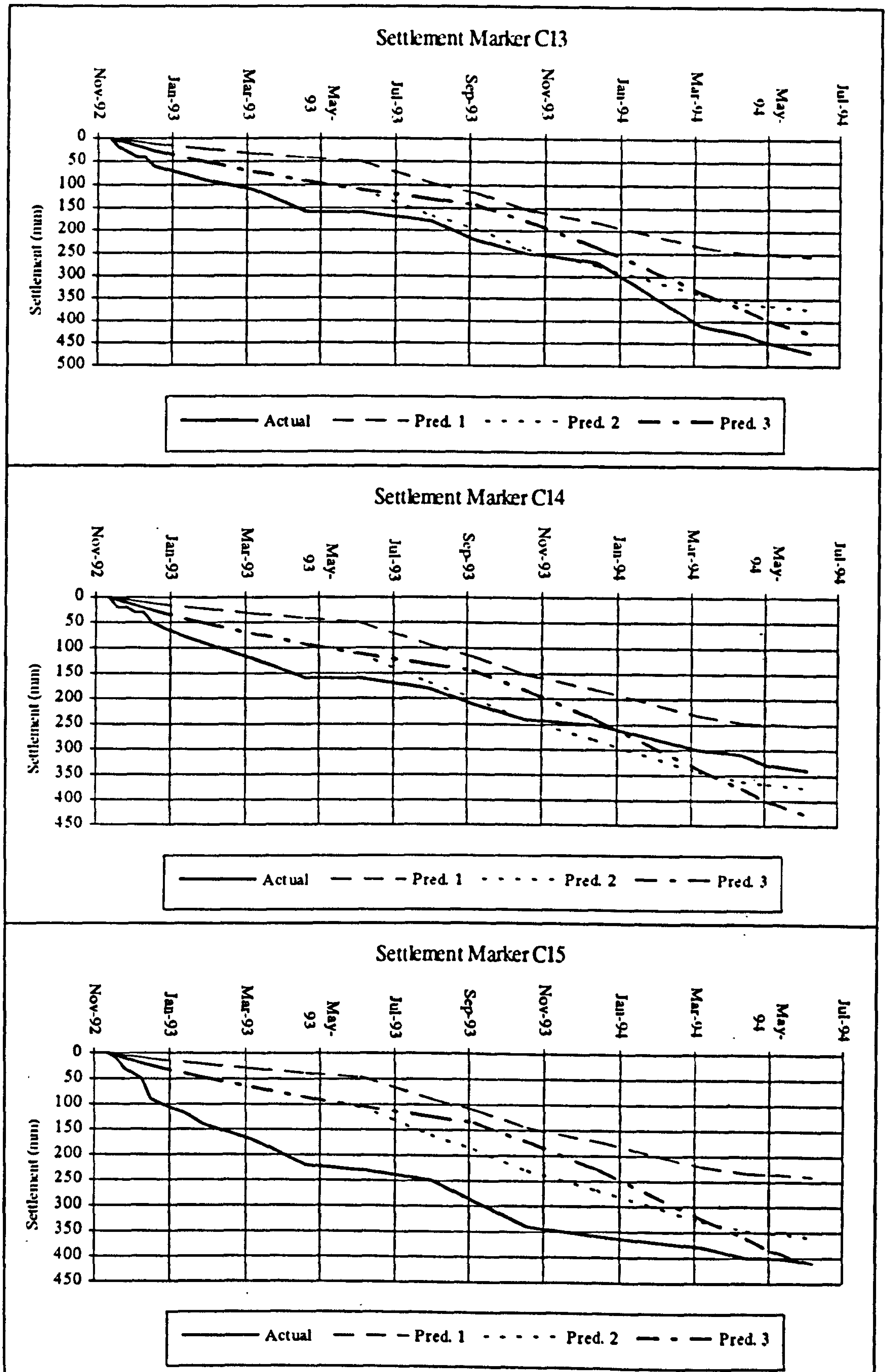


Figure 7.14. Actual and predicted settlements at markers C13, C14 and C15

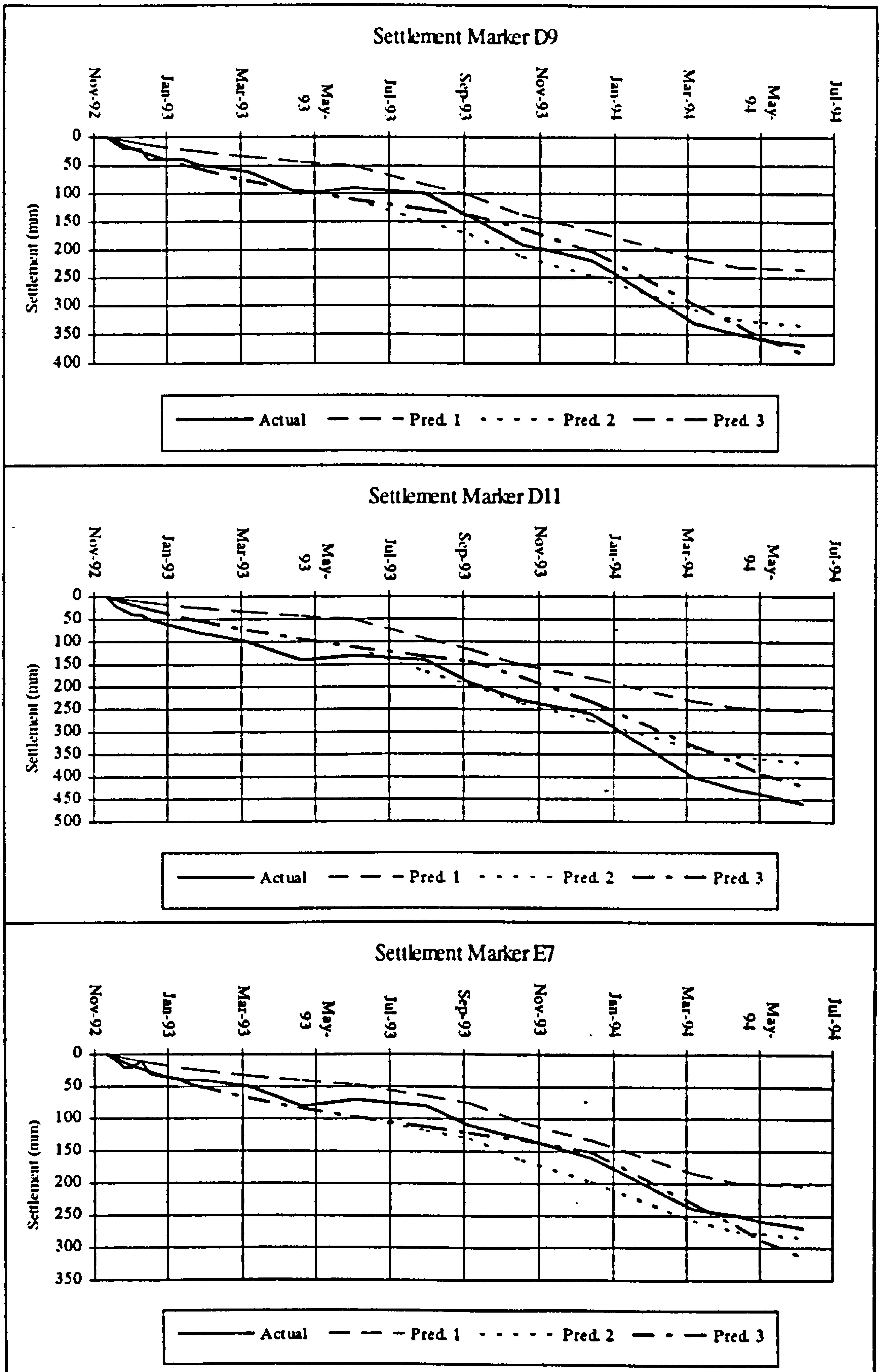


Figure 7.15. Actual and predicted settlements at markers D9, D11 and E7

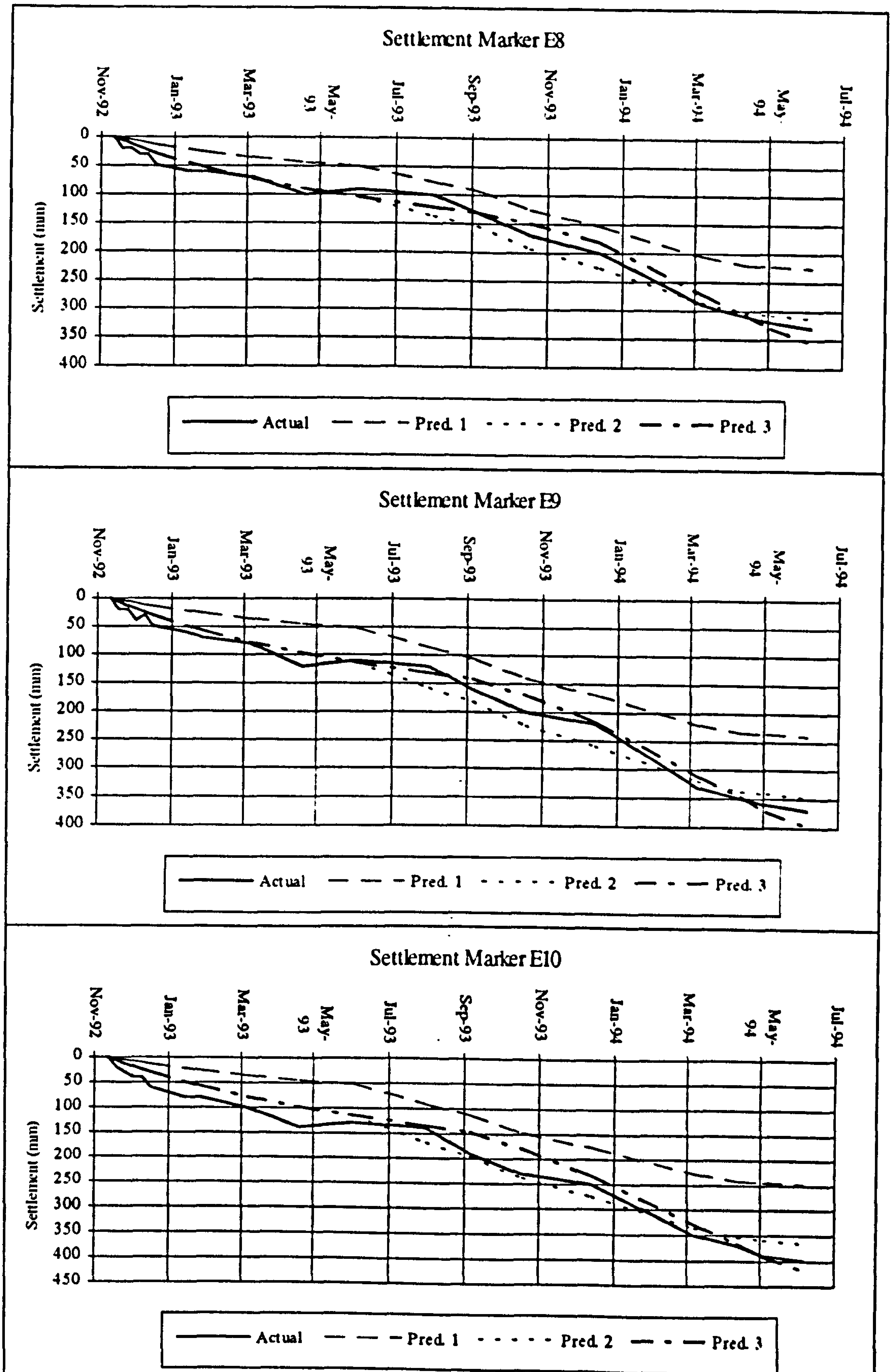


Figure 7.16. Actual and predicted settlements at markers E8, E9 and E10

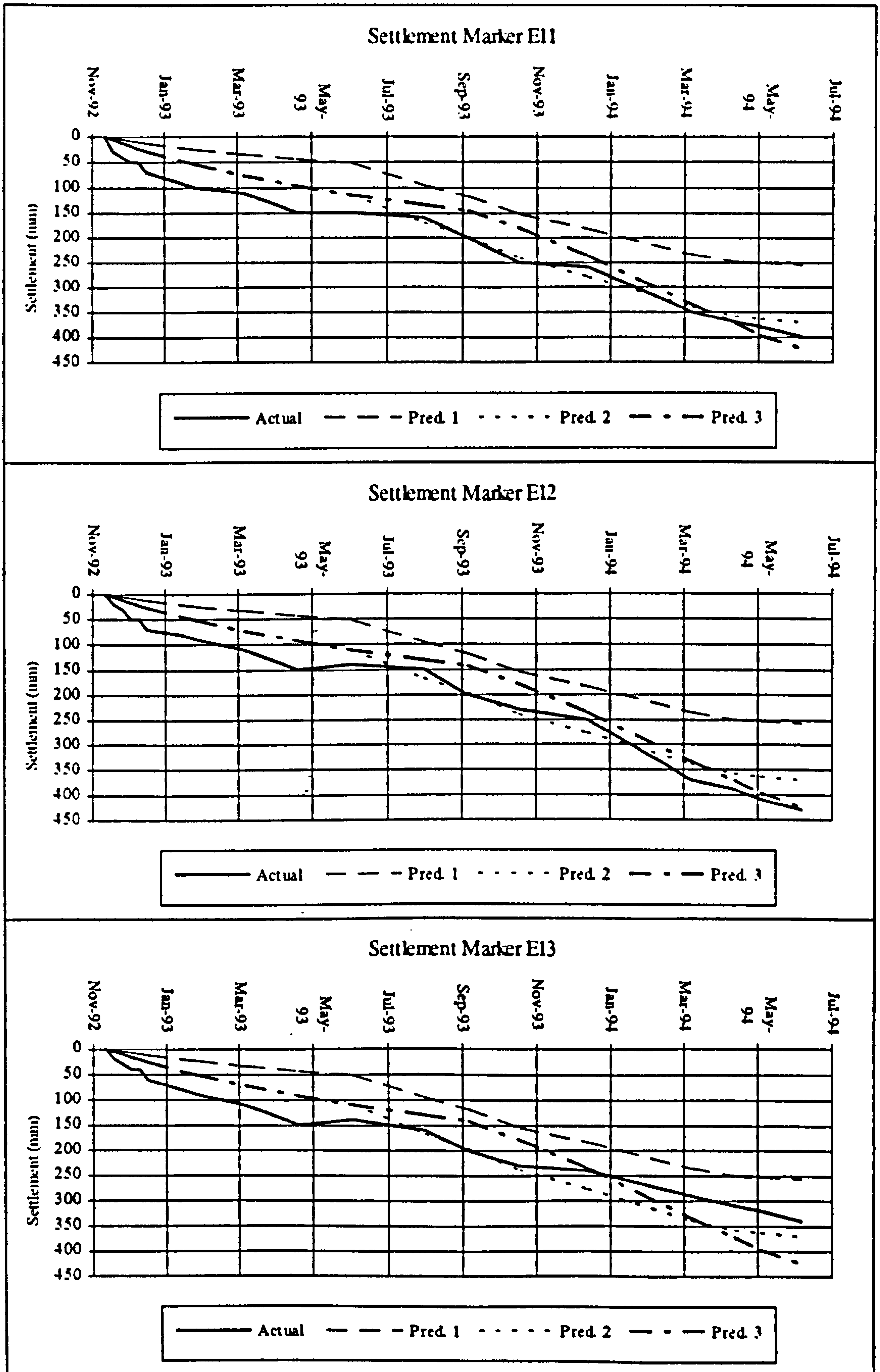


Figure 7.17. Actual and predicted settlements at markers E11, E12 and E13

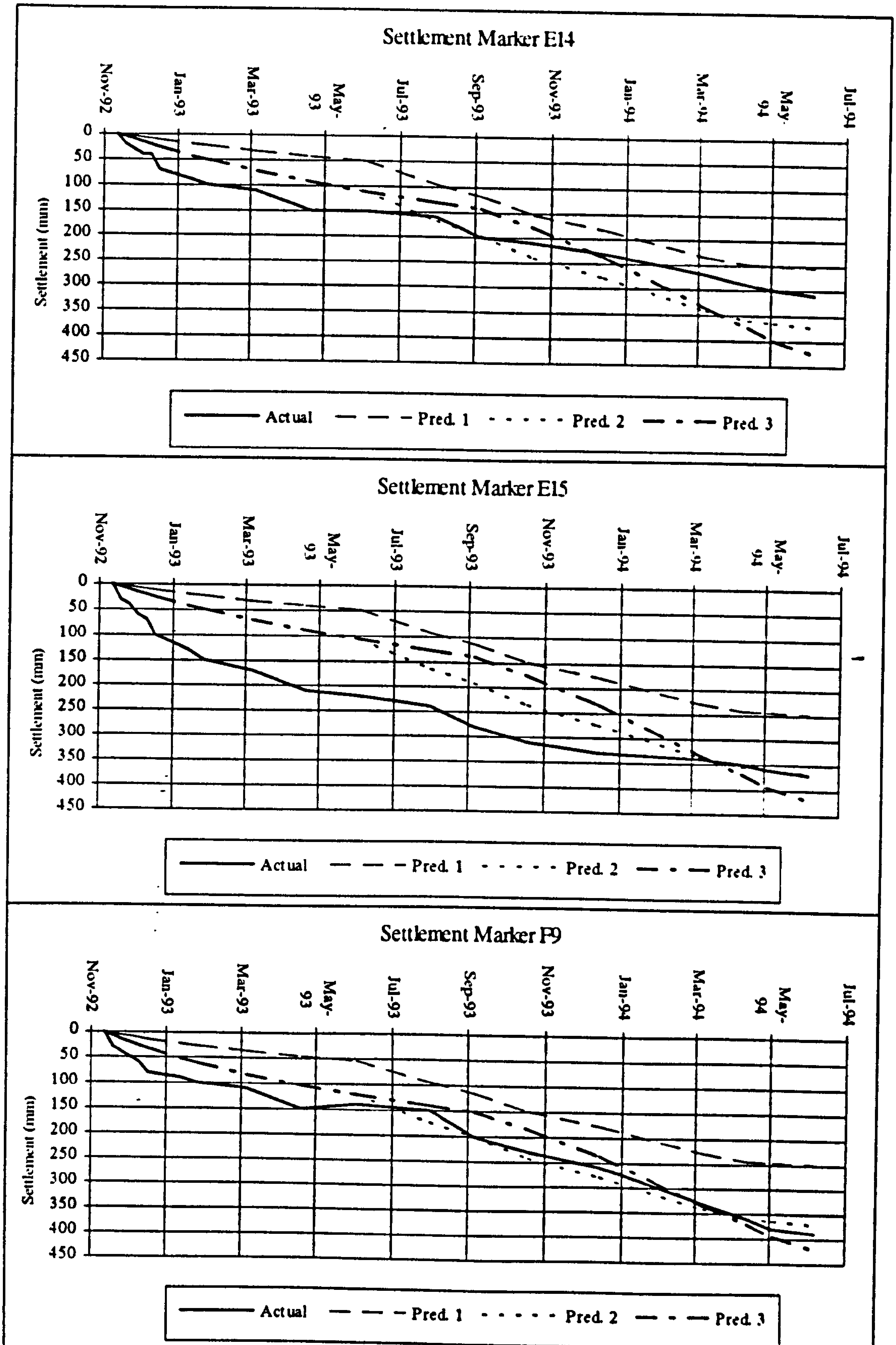


Figure 7.18. Actual and predicted settlements at markers E14, E15 and F9

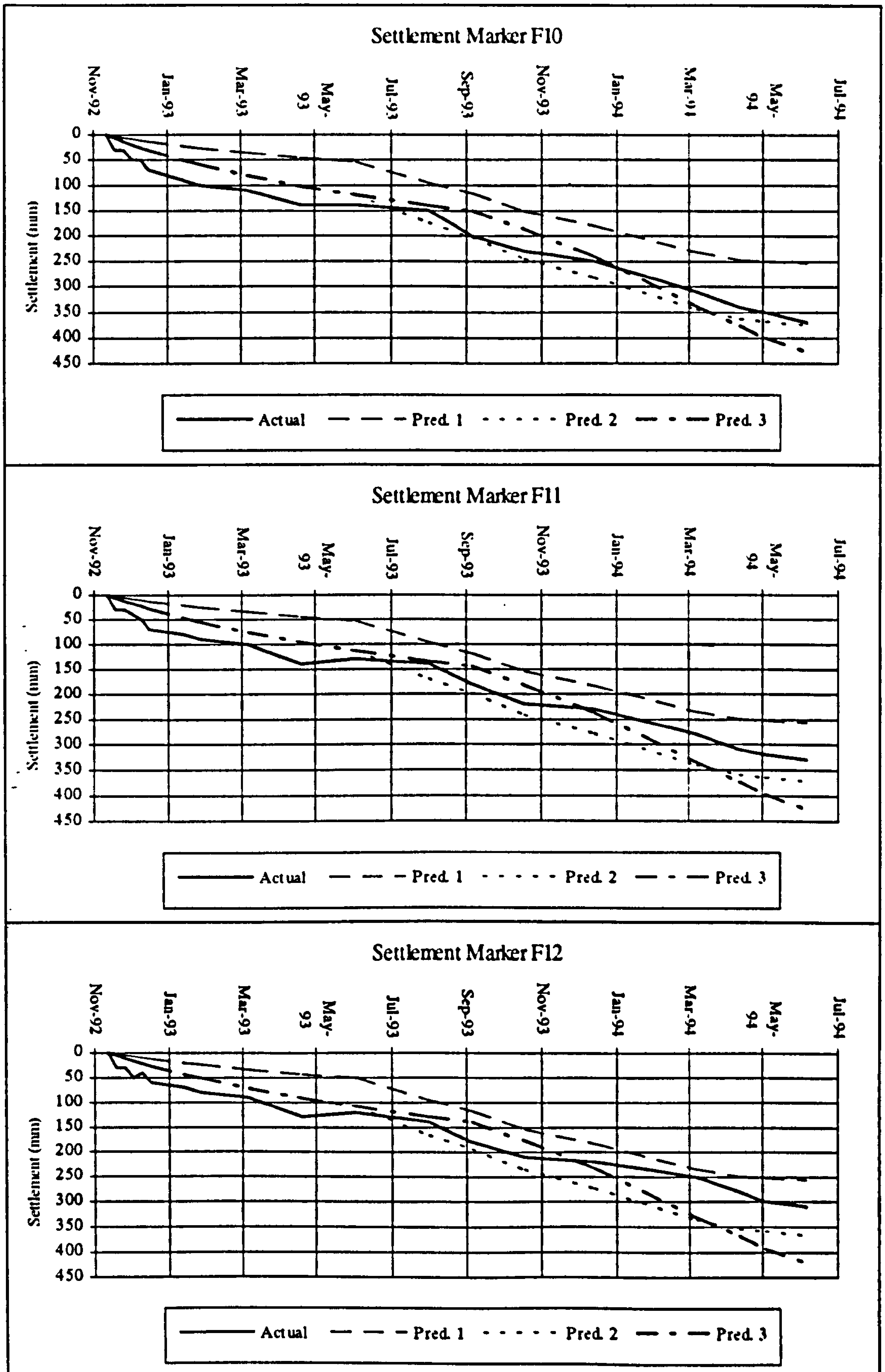


Figure 7.19. Actual and predicted settlements at markers F10, F11 and F12

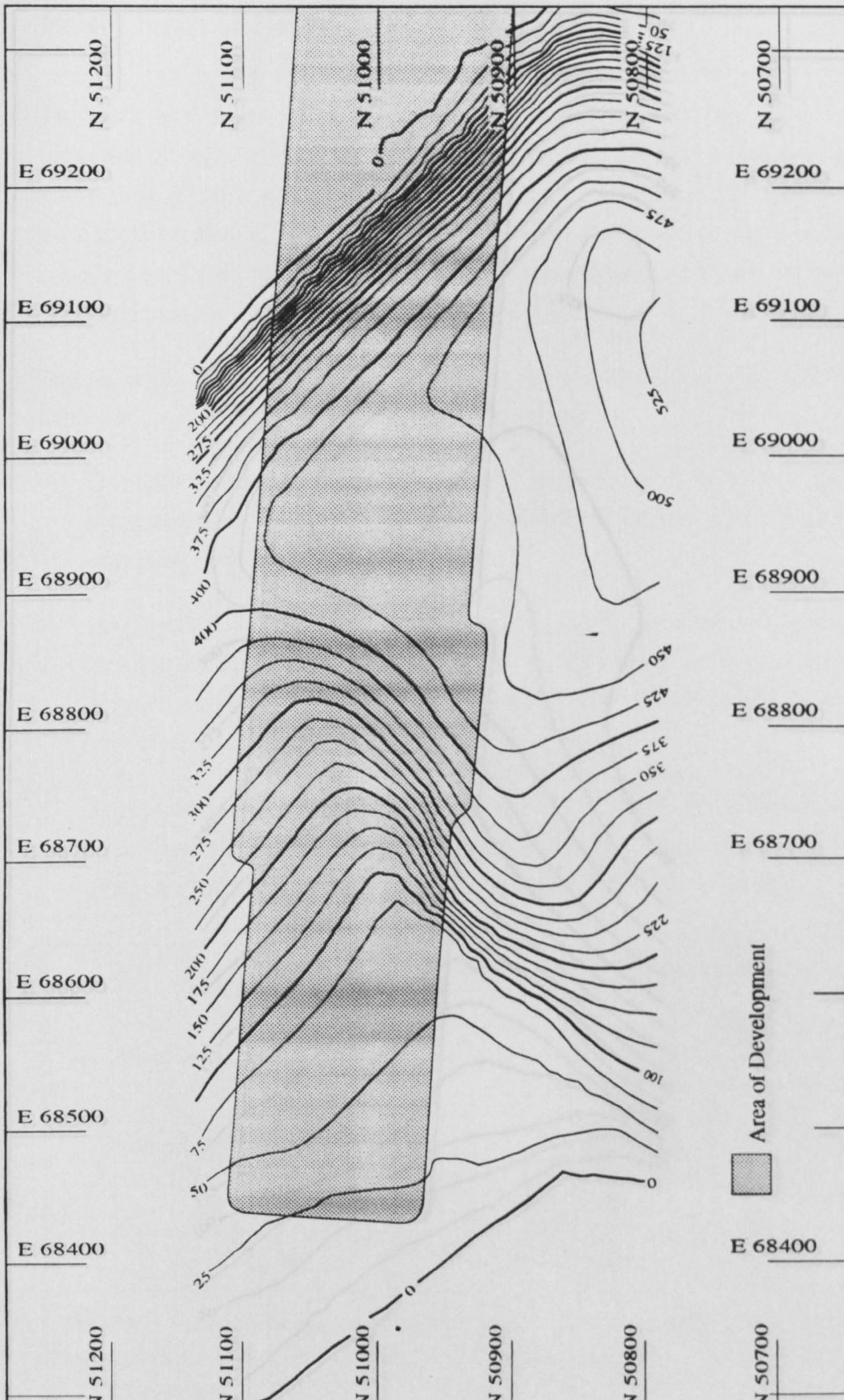


Figure 7.20. Site A, predicted settlement contours (mm) for the prediction period 21 to 435 days from the completion of backfilling operations.

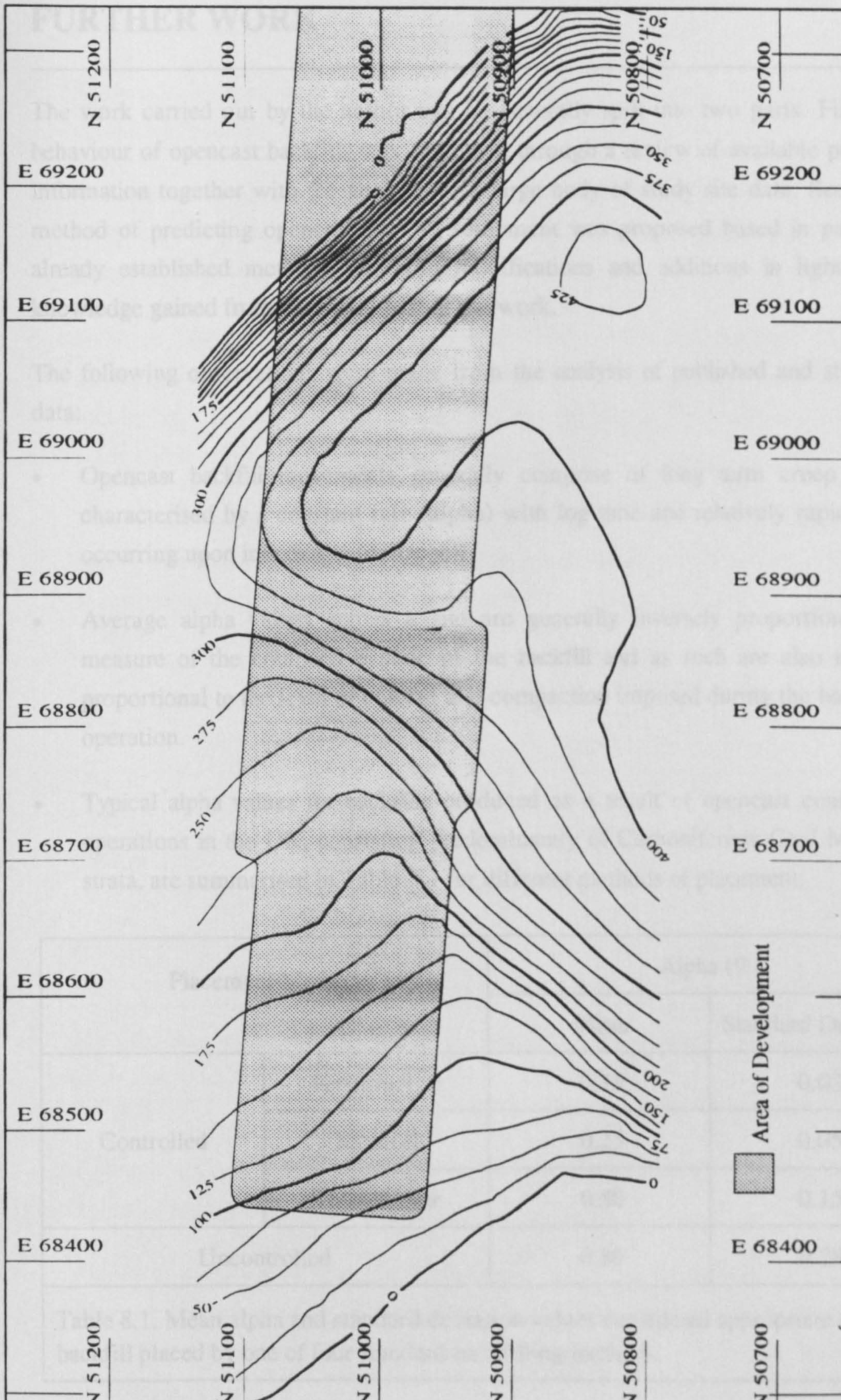


Figure 7.21. Site A, predicted settlement contours (mm) for the prediction period 436 to 9565 days from the completion of backfilling operations.

CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER WORK

The work carried out by the author can be generally spilt into two parts. Firstly the behaviour of opencast backfills was examined through a review of available published information together with the analysis of a large body of study site data. Secondly a method of predicting opencast backfill settlement was proposed based in part upon already established methods but with modifications and additions in light of the knowledge gained from the initial part of the work.

The following observations were made from the analysis of published and study site data:

- Opencast backfill movements generally comprise of long term creep strains, characterised by a constant rate (alpha) with log time and relatively rapid strains occurring upon inundation (collapse).
- Average alpha values across a site are generally inversely proportional to a measure of the compactive state of the backfill and as such are also inversely proportional to the level of control and compaction imposed during the backfilling operation.
- Typical alpha values for backfills produced as a result of opencast coal mining operations in the UK, consisting predominantly of Carboniferous Coal Measures strata, are summarised in Table 8.1 for different methods of placement.

Placement Method		Alpha (%)	
		Mean	Standard Deviation
Controlled	Performance	0.15	0.03
	Method	0.25	0.05
	Thick Layer	0.50	0.15
Uncontrolled		0.80	0.28

Table 8.1. Mean alpha and standard deviation values considered appropriate to backfill placed by one of four standard backfilling methods.

- Strain as measured within the main body of the backfill through the use of extensometers, shows no apparent relationship with fill depth.

- Settlement as measured at the surface, even with relatively homogenous, well compacted backfill, is not simply proportional to fill depth.

- Collapse settlements as a result of inundation have been noted at the majority of study sites even those where a full scheme of controlled backfill placement and compaction had been carried out.

- Average collapse strains, as measured at depth within the inundated layer, are generally inversely proportional to a measure of the compactive state of the backfill and as such are also inversely proportional to the level of control and compaction imposed during the backfilling operation.

- Typical collapse strain values for backfills produced as a result of opencast coal mining operations in the UK, consisting predominantly of Carboniferous Coal Measures strata, are summarised in Table 8.2 for different methods of placement.

Placement Method		Collapse (%)	
		Mean	Standard Deviation
Controlled	Performance	0.25	0.04
	Method	0.40	0.08
	Thick Layer	0.90	0.28
Uncontrolled		1.20	0.41

Table 8.2. Mean collapse and standard deviation values considered appropriate to backfill placed by one of four standard backfilling methods.

- Collapse settlements, as measured at depth within the inundated layer, show no apparent relationship with fill depth.

- Levels of compaction that can be achieved for different methods of placement, expressed in terms of approximate dry density and air void values, for backfills consisting predominantly of Carboniferous Coal Measures strata are summarised in Table 8.3.

Placement Method		Dry Density (Mg/m^3)	Air Voids (%)
Controlled	Performance/Method	1.85 - 2.10	< 10
	Thick Layer	1.70 - 1.85	10 - 15
Uncontrolled		< 1.70	> 15

Table 8.3. Approximate dry density and air void values considered appropriate to backfill placed by one of four standard backfilling methods.

- An equilibrium groundwater table is attained close to or equal to the pre-excavation level generally within a year of completion of backfilling operations and the cessation of any pumping operations affecting the local groundwater table. However, such a rebound can be influenced by adjacent mining activities, past and present.

It was through these observations together with a study of presently available techniques that a method of predicting the settlement of opencast backfills was developed. The method proposed is based upon the property that the rate of the creep component of the backfill settlement is constant over log time (Sowers *et al* 1965). The salient points of the method are as summarised below:

- Factors important to the determination of backfill settlement are the timing of the backfilling operation, the constituent rock types making up the backfill and the level of control and compaction during placement. Thus the initial stage of the proposed method is to divide the backfill into separate blocks delineated by period of placement, material type and compactive state.

- The properties relevant to the prediction of settlement, are assigned to each of the backfill blocks.

- The settlement at a given point on the surface is determined from the cumulative settlement of the many layers beneath that point the backfill is split into.

- Dividing the backfill into individual blocks and layers enables a better estimate to be made for the time at which creep strain is considered to have commenced within a given backfill layer.

- The settlement determined for any given layer is simply a combination of the creep component of the settlement and the collapse component if the layer becomes inundated.

- Creep settlement for a given layer is determined over any prediction period by relating this period back to the time origin creep is deemed to have commenced; Having adjusted the prediction period in relation to the commencement of creep, settlement can be easily calculated from the alpha value assigned to the backfill.

- Collapse settlement is determined over the same prediction period from an estimate of the amount and type of backfill that becomes saturated together with relevant collapse strain values.

- The recovery of the groundwater table is expressed in terms of a series of surfaces representing how the surface of the groundwater table changes with time from the completion of backfilling operations. This enables the amount of backfill that becomes saturated and within which blocks this occurs to be determined.

- The settlement at a given point is not taken in isolation and a method of accounting for the influence of surrounding backfill material has been devised.

- The heterogeneous nature of the backfill can be taken into consideration by predicting an average and standard deviation settlement value for any point based upon observations of the typical variation of alpha and collapse strain values for given methods of backfill placement.

- A largely theoretical approach is devised to take into consideration the influence the ground improvement technique of surcharging has upon predicted settlements.

The method therefore relies on the ability to be able to divide the backfill into distinct blocks delineated by period of placement, material type and compactive state. The information required to achieve this will be different dependant upon whether predictions are to be made at the planning stage or upon completion of operations. In practice it will probably be a combination of the two with initial modelling and predictions being modified in the light of in situ testing data and monitoring during the backfilling operation.

At the planning stage information regarding the type of strata and volumes that are to be excavated and replaced as well as pre-excavation groundwater levels, would be obtained from pre-excavation drilling operations. The timing of the backfilling operation and zones and degrees of control and compaction would be determined during the planning of the mining operation. This information would then enable a model of the backfill to be constructed from which settlement predictions could be made. Thus different methods of backfill placement could be examined in terms of post backfilling settlements.

Upon completion of the backfilling operation a more accurate model of the backfilling operation could be made. Any modifications to the original specifications and differences between predicted strata types and volumes and those actually excavated and replaced, could be accounted for. In situ monitoring of the backfilling operation would also enable a more accurate estimate of the settlement characteristics of the backfill to be made. Backfill scheduling data would provide information on the advance of the backfill surface thus the backfill blocks, making up the backfill as a whole, could be more accurately defined.

It may be considered that the data necessary to build up an accurate picture of the backfill would not be available at the majority of opencast coal sites. It is in the opinion of the author however, that this is not the case. With ever increasing demands on land use, development upon completion of backfilling is becoming a necessity for mining permission to be granted. Therefore at a significant proportion of the opencast coal sites presently in operation, development of the restored backfilled land is a requirement. Thus a degree of controlled placement and compaction will be necessary during backfilling. This will result in the accumulation, through the necessary monitoring of the backfilling operation, of the data required to model the backfill. Site A, of the case study above, gives a good example of this and it is considered that the data made available from Site A is largely typical of that which would be available from any controlled opencast backfilling operation.

In the cases where backfill monitoring is limited a scheme of settlement monitoring carried out upon completion of the backfilling operation, could be used to supplement available data. This would provide a better picture of the backfill enabling modification to the original model to be made resulting in more accurate future settlement predictions.

The proposed method of settlement prediction was implemented through the development of a PC based computer program known as OBSett (Opencast Backfill Settlement Prediction Package), written in the C programming language to run under the Windows operating system thus providing a user friendly graphical interface. The modelling of the backfilling operation was carried out with the use of conventional mine design and CAD software, SURPAC and AutoCAD respectively. The backfill model is provided as input to OBSett enabling settlement predictions to be made for any prediction period at a point, along a section, over an area or across the whole site.

To test the validity of OBSett, extensive program testing was carried out together with a comparison between predicted and actual monitored settlements taken at the case study site, Site A. Program testing demonstrated that OBSett responded within realistic limits to different backfilling scenarios and the comparison demonstrated that predictions could be made in the region of 10% of those actually monitored.

The work carried out has therefore provided a means of predicting the settlement of opencast backfill within reasonable levels of accuracy, through the examination of a large quantity of backfilling and settlement monitoring data from a number of sites within the UK. Settlement predictions can be made at both the planning stage of the mining operation or upon completion of backfilling thus providing useful information when designing a backfilling operation to suit a particular development or design a development to suit a particular backfill.

The work has also identified a number of areas in which further work is considered necessary to better understand the behaviour of opencast backfill. The settlement characteristics of opencast backfills have been defined in terms of the creep compression rate parameter (alpha) and the magnitude of collapse strain upon inundation. In the case of alpha a number of questions arise when it is used in predicting creep settlement. Does creep settlement (and hence alpha) have a relationship with effective stress? What is the relationship between alpha and the material properties of the backfill? At what point in time is it considered that creep commences? Similar questions arise in the use of collapse strain values for the prediction of settlement.

In this work it is assumed that alpha remains constant regardless of the effective stress, that the time at which creep commences is defined in terms of the ratio between the stress at a given time and the total stress and that an alpha value can be assigned to a given backfill in terms of its method of placement.

The assumptions made, are in light of presently available data and it is considered that these points could be better understood through further examination. Linear relationships between alpha and effective stress have been proposed by Charles (1993) for different material types under similar methods of compaction. Thus the first steps have been made in establishing relationships between alpha, effective stress, material type and method of placement.

To further establish such relationship experimental methods could be employed, the difficulties of which would be ensuring that the samples tested where representative of opencast backfills. This would entail using large samples thus specialised testing equipment would have to be built.

Another approach would be the examination of measurements made in situ, most significantly those made from extensometer installations which enable creep to be measured over a given layer at depth within the backfill. Extensometer measurements would have to be analysed in conjunction with detailed measurements made during the placement of backfill such that for a given layer over which strain is being measured the material type and method of placement could be identified. Also as the determination of alpha requires an estimate of the point at which creep is considered to have commenced detail about the backfilling schedule would be required such that the timing of backfill placement, throughout the depth of an extensometer placement, could be determined.

Thus given enough monitoring data, relationships could be determined between effective vertical stress and alpha for a range of backfill materials and methods of placement. Backfill material could be classified in terms of its compaction properties as in Classes 1A to 1C and 2A to 2D used to classify material in both the performance and method backfill compaction specifications (chapter 4). Thus it would be possible to assign a typical alpha value to any backfill material, at any depth, for a given standard method of backfill placement. This information could be easily incorporated into OBSett potentially leading to more accurate estimates of creep settlements.

The determination of the point at which creep settlement commences is an important part of the proposed method of settlement prediction. Creep settlement occurs under conditions of constant stress and moisture content. Therefore it can be considered that the point at which creep settlement is deemed to have commenced is the point at which stress becomes constant. In an opencast backfilling operation this can be considered equal to the time that the last layer of backfill is placed for a given column of backfill material. However, unless backfill placement is very rapid a considerable delay can occur between the placement of the first layer and that of the last layer. Thus the assumption that the time at which creep settlement commences within the bottom layer, is equal to the time at which the last layer was placed can lead to a significant over estimate of the creep settlement within that layer.

To accommodate this the time at which creep settlement is deemed to have commenced is taken as the time at which the stress due to the placement of material above a given layer is equal to a certain proportion of the total stress once all the material is placed. This work has defined the point at which creep occurs in terms of a constant value which defines the proportion of the total stress creep is deemed to commence. This constant is known as the Percentage Stress Increase Cut-Off Value (x) and its influence on predicted settlements was examined in chapter 6.

Study site data was insufficiently detailed to be able to define a suitable value for x. The value taken by this work is such that the commencement of creep within a layer is set equal to the time taken to construct half the full thickness of backfill above that layer given backfill of uniform density.

It is considered that to better define the value of x, analysis of more detailed site data than was made available to this work would be required. If the site data was sufficiently detailed to enable the timing of backfill placement to be accurately known, the comparison between predicted settlements and actual monitored settlements could then be used to determine an appropriate value for x.

The above discussion therefore outlines ways in which the creep component of backfill settlement could be better understood and defined. Similar methods could be employed to further examine the collapse component of the settlement. Examination of further study site data combined with large scale laboratory testing could be used to identify the relationships between collapse, effective vertical stress, material type and levels of compaction. Levels of compaction could be expressed in terms of air void values such that presently proposed relationships between air voids and collapse could be further established.

A point that hasn't been addressed by this work, concerning collapse, is that of post-inundation creep strain. It can be considered that backfill material that has undergone collapse will be in a denser state thus subsequent creep strains will be reduced. Evidence has been found for such behaviour within the literature but not in sufficient detail for quantification. It is believed that the most recent settlement monitoring carried out at the case study site, Site A, (too recent for inclusion in this work) indicates reduced creep strains following collapse. Such behaviour could have a significant influence on the prediction of settlements and as such is an area that requires further examination.

Another important point with regard to collapse is that of collapse as a result of surface water infiltration. Its magnitude is difficult to access in relation to the level of saturation that occurs within the backfill as the amount of surface water infiltration is difficult to measure. It is considered however that collapse as a result of surface water infiltration will be less than that as a result of groundwater inundation. Settlement monitoring in conjunction with some measure of infiltration would be required to better understand this type of collapse. The approach taken by this work, without the benefit of relevant monitoring data, is the recommendation that adequate drainage be installed during site development to ensure surface infiltration is kept to a minimum thus collapse as a result of surface water infiltration could be considered negligible.

The techniques incorporated into the settlement prediction method to account for differential settlements would also benefit from the examination of further study site data. Two points are of issue: the build up of shear stresses developed within the body of the backfill due to differential movements and the heterogeneous nature of the backfill.

In the first case an empirical approach has been taken with the settlement at any point being adjusted by the settlement of surrounding points (chapter 5). This has the affect of 'smoothing out' rapid changes in fill depth or material properties on the predicted settlement. By considering a greater area within which the settlement of surrounding points is taken as being influential, for increasingly compacted backfill, the 'smoothing out' affect increases with increasing backfill compaction. It is considered that this is the behaviour that would be observed as a result of the build up of shear stresses between backfill fragments as the fill moves differentially between rapid changes in fill depth or material properties. Further study site settlement observations would provide useful information to better establish the technique employed or indicate alternative methods.

The means by which the heterogeneous nature of the backfill is incorporated is to generate a population (assuming a normal distribution) of settlement values for a given point. This relies on the assumption that a range of alpha and collapse strain values exist for a given backfill following a normal distribution that can be defined by a mean and standard deviation value. It is considered that just as alpha and collapse strain values can be assigned to backfills dependant upon methods of placement so to can measures of the heterogeneous nature of backfills. In this case the heterogeneous nature is expressed in terms of a standard deviation value for both alpha and collapse strain. The greater control carried out during backfill placement the less heterogeneous the backfill will be hence the smaller the differential settlements as a result of backfill heterogeneity. Standard deviation values appropriate to different methods of placement could be further examined in conjunction with the establishment of the alpha and collapse strain relationships discussed above.

Where the above methods of accounting for differential settlements may prove inadequate is in the case where the backfilling operation incorporates the placement of a near surface layer of highly compacted backfill over relatively uncompacted material. This backfilling approach, whilst having little affect upon total settlements, can lead to considerable improvements upon differential settlements. Only through the examination of measured differential settlements compared with predicted results, for a site incorporating such a backfilling approach, can the validity of the methods proposed be further examined and if necessary modified.

An area of importance to the design of opencast backfilling operations, that has been left largely un-addressed by this and other published work, is that of backfill improvement methods. These will most notably consist of surcharging, dynamic compaction and artificial inundation. A method of determining the beneficial affect of surcharging has been proposed (chapter 4) but this is largely theoretical and requires further monitoring data to be better established. Methods for determining the beneficial affects of dynamic compaction and artificial inundation have not been proposed due to the limited amount of data available concerning there application to opencast backfills; this is therefore another area in which further work would be beneficial.

Finally, backfill movements only briefly examined by this work are those of heave and lateral movements which may in some cases be sufficiently large to be destructive to post restoration developments. It is considered that heave will generally only be of an issue as a result of the rebound affects following surcharging especially where a significant proportion of the backfill beneath the surcharge has been affected. As a general rule the affect of surcharging is negligible within backfill at depths approximately equal to and greater than the height of the surcharge (chapter 4). Lateral movements will be associated with large differential settlements and as such will be mainly observed above side walls; their magnitude will be inversely proportional to the level of control and compaction carried out during backfill placement.

COLLAPSE SETTLEMENT CASE STUDIES

Horsley, Northumberland

In 1973 the Building Research Establishment had the opportunity to investigate the effect of a rising ground water table on the settlement of an opencast coal mining backfill (Charles et al 1984a). The opencast workings covered an area approximately 1500m by 600m and were up to 70m deep in places. Backfilling took place between 1961 and 1970 with the restoration completed in 1973. The excavated strata belong to the Middle and Lower Coal Measures of the Carboniferous system. The backfill is composed largely of mudstone and sandstone fragments. In the upper part of the workings excavation of the overburden was carried out by face shovels and backfilling by end tipping from dump trucks. In the lower part of the workings excavation was by dragline.

It was necessary to de-water the site during the period of mining and pumping was continued after completion of backfilling up until early 1974. Magnetic extensometers and piezometers were installed in boreholes in 1973 to monitor settlement at different depths some four months prior to pumping being stopped.

When examining the settlement behaviour of the backfill it is helpful to consider three periods of monitoring :

Period (i) the four month period of monitoring prior to cessation of pumping when pumping kept the water table below the backfill.

Period (ii) the three year period when the water table rose some 34m reaching a new equilibrium in April 1977.

Period (iii) six and a half years subsequent to April 1977 when the water table showed only minor changes.

Settlements recorded in the deepest part of the backfill at gauge B2 are plotted in Figure A1. Here 8mm of settlement were recorded at ground level (magnet 13) during period (i), 338mm during period (ii) and 142mm during period (iii).

The vertical compressions measured between adjacent magnets at gauge B2 are plotted against time in Figure A2. Also shown is the rate of water level rise expressed in terms of magnets 3 to 9 . As the water rose from the level of magnet 5 to the level of magnet 6 a vertical compression of just over 1% occurred over the depth of fill between these two magnets. As the water table continued to rise large compressions occurred successively between magnets 7 and 8, 8 and 9 and 9 and 10. Most of the settlement occurred between magnets 11 to 5 (8 m to 43 m below ground level). As the water table rose compressions were locally as large as 2% but the average compression over the 34m of saturated backfill was less than 1%. The effect of saturation in producing collapse compression within the backfill was thus clearly demonstrated.

[Charles *et al* (1984a), Charles (1984), Charles, Burford (1985), Charles Burford (1987)]

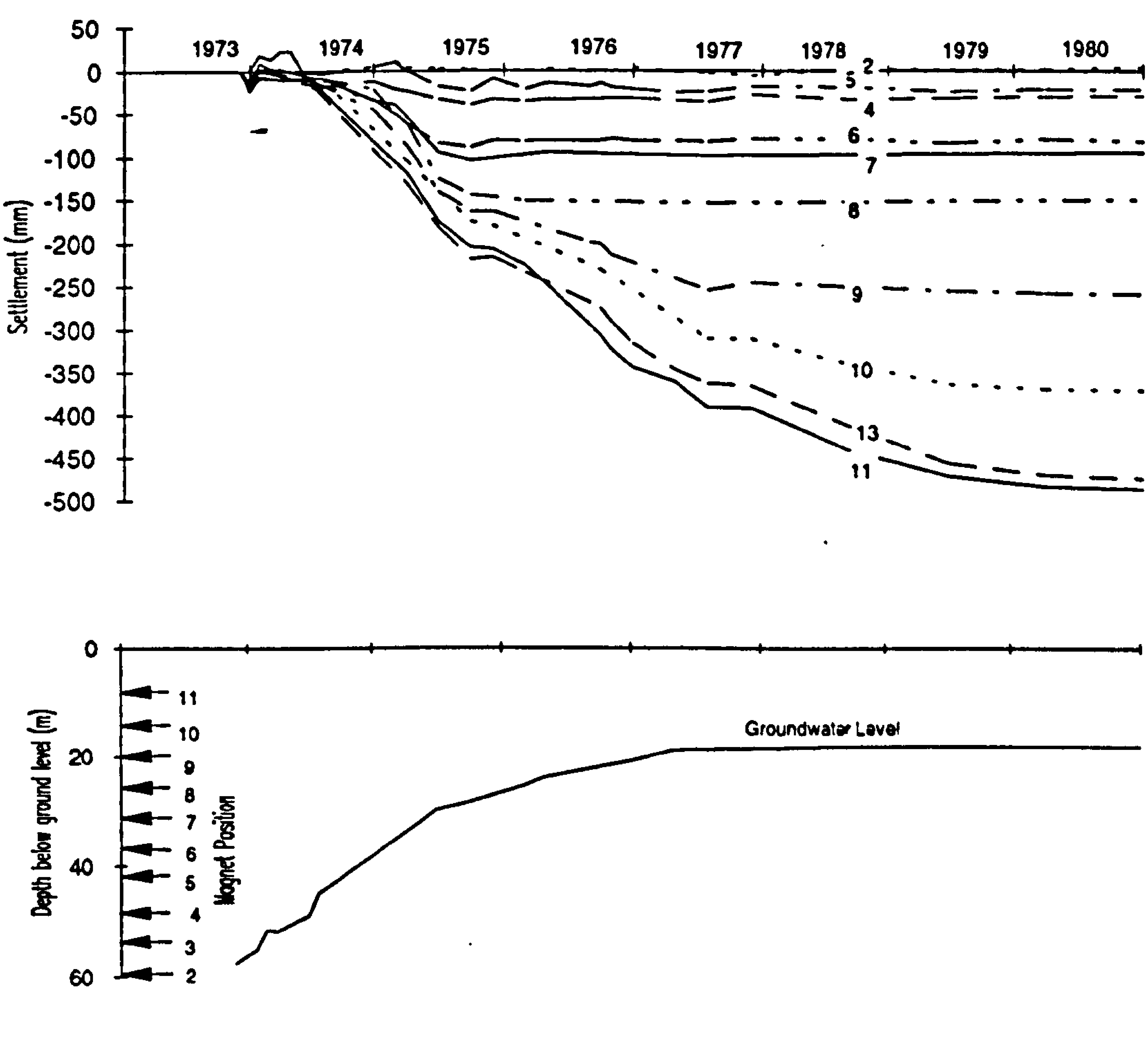


Figure A1. Settlement monitored at gauge B2 in deepest backfill, and change in water table (Charles Burford 1985)

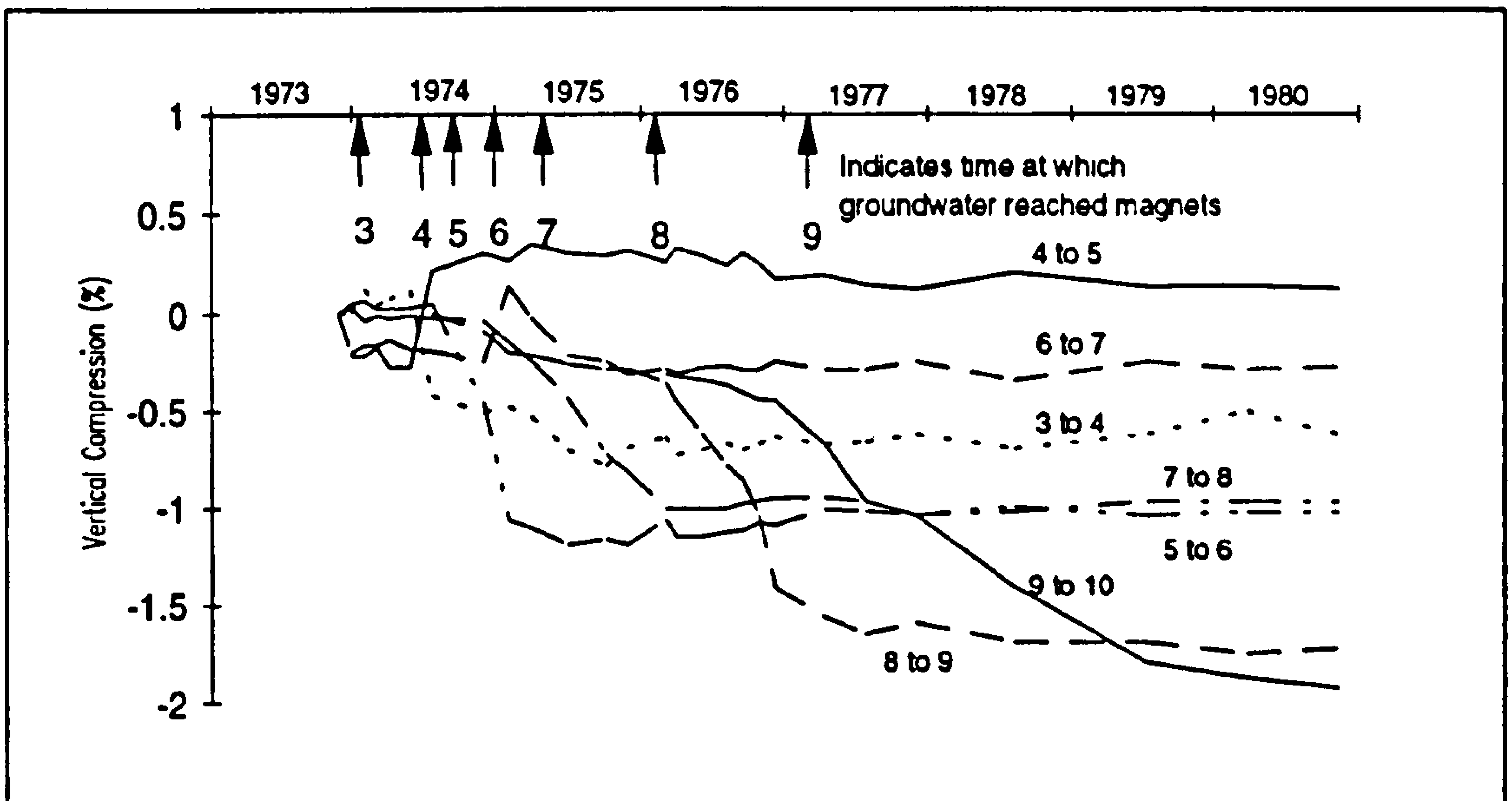


Figure A2. Vertical compression at gauge B2 (Charles Burford 1985)

Ilkeston

A block of terrace houses were built during 1972 on a site where opencast mining, completed in 1959, had left some 10 metres of backfill. Early in May 1973 excavation for drains began close to the north gable end of the terrace and a cracked floor and beam were observed. Following heavy rain in June, movement took place in the centre of the row of houses. In July it was reported that all the houses in the block were affected. Underpinning and pressure grouting were carried out but movements continued. In 1982 after having never been occupied the block was demolished following an estimated 0.3 metres of settlement (approximately 3% compression).

It was suggested that water penetrating into the fill through drain trenches was the major cause of the settlement. An inundation test was carried out in 1975 to test this hypothesis. Three extensometers were installed adjacent to trenches of 2m by 1m by 3 metres deep. The trenches were filled with water, the rate at which water passed into the backfill varied from 0.08 m^3/hour to 2 m^3/hour in the different trenches. Within 24 hours of filling the trenches, additional settlements of up to 50mm had been recorded confirming that water penetrating into the opencast backfill via surface trenches could cause significant collapse compression.

[Charles (1984), Charles, Burford (1987)]

Snatchill Experimental Housing Site, Corby

To investigate the problems that could arise with the construction on opencast backfill and to assess the effectiveness of ground improvement methods an experimental housing site was designed in Corby. Previous mining activity during 1963 to 1970 had involved the opencast extraction of iron ore in 20m wide strips using a walking dragline to excavate overburden. This was carried out in two stages, excavation of the upper boulder clay followed by the blasting and excavation of Oolitic Limestone overlying the ironstone beds. Excavated material was dumped directly into the working void left behind once the iron ore was removed, the limestone being placed at the bottom, the boulder clay on top. The upper part of the restored ground is therefore predominantly cohesive. The fill had been left in a characteristic hill and dale formation, and then levelled by scrapers at a later date. Groundwater level had remained below the base of the fill (24 m deep) since drainage had been left in place following backfilling.

A laboratory investigation carried out on backfill material obtained from instrumentation drilling indicated a mean dry density of 1.70Mg/m^3 and an average moisture content of 18 %. Test 12, max. dry density was 1.92Mg/m^3 at an optimum mc of 14%. A particle size analysis indicated that 54% of the material was finer than 0.075mm and that the clay fraction was 19%.

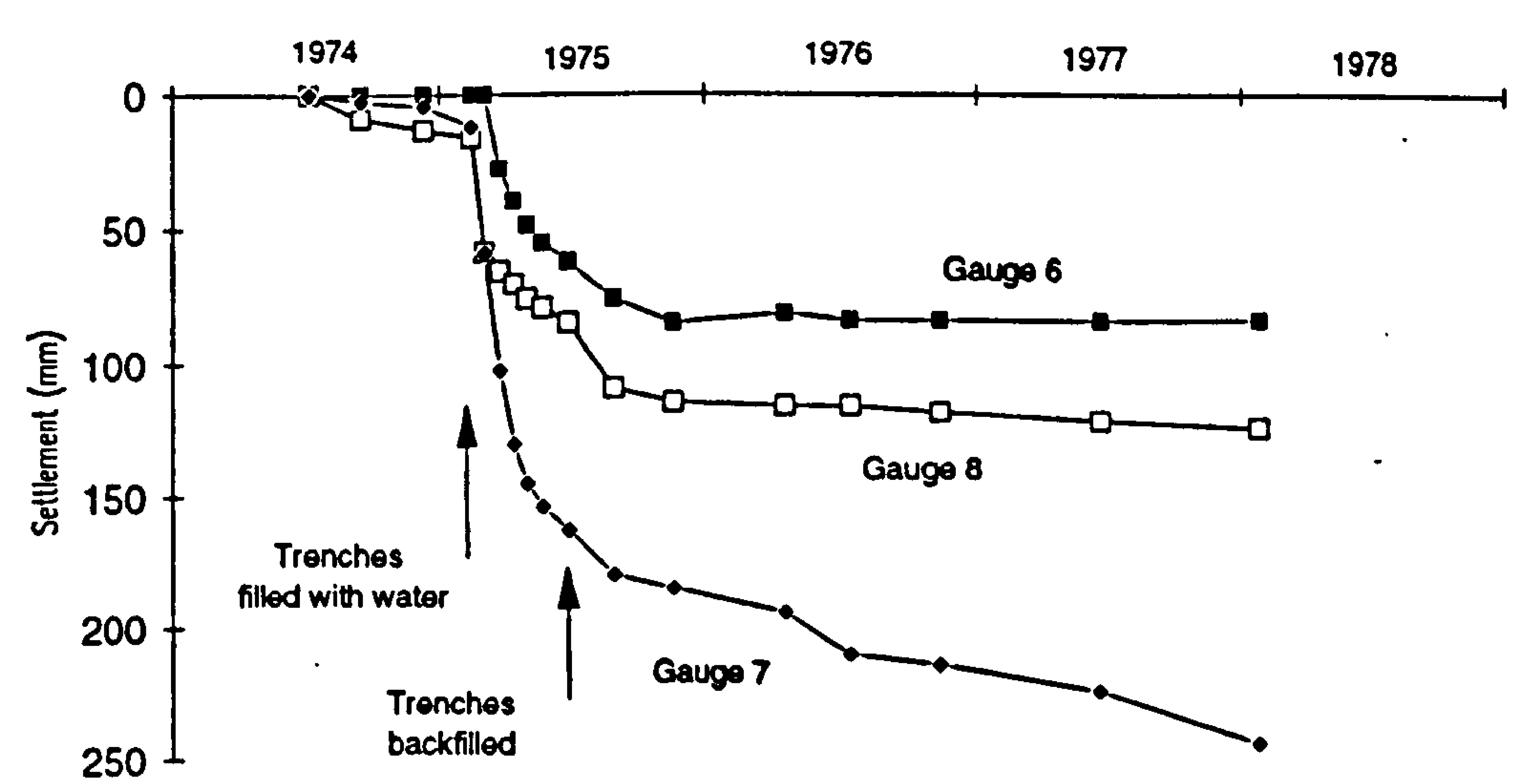


Figure A3. Surface settlement vs time (Charles, Burford 1978)

In an attempt to improve ground conditions at one of four experimental areas within the site, five 1m deep trenches at 10m centres were dug and kept full with water for four months after which they were backfilled. It was estimated that during the first ten days of the experiment some 90 m^3 of water were absorbed by the fill. Comparatively little water was absorbed subsequently. Settlements were measured by 5 extensometers, some results of which are shown in Figure A3 which clearly shows the increased settlement as a result of inundation. Settlements measured at extensometer 7 are considerably larger than those measured at the other extensometers. Extensometer 7 was within 2 metres of the trench through which the largest water penetration occurred, a volume approximately equal to the volume lost from all the other four trenches added together. The settlement caused by inundation seemed to be confined to the upper 5m of the fill.

[Charles Burford (1978), Charles Burford Watts (1986), Charles Burford (1987)]

West Auckland

In 1971 a single storey light industrial factory was built on Greenfields Estate, West Auckland. In 1977 serious differential settlement was experienced at the factory. An investigation showed the backfill to consist generally of a firm to stiff silty clay with fragments of shale. Extensometers and piezometers were installed to monitor further settlements and revealed a correlation between a rising water table and increased compression in the part of the backfill newly saturated. In 1978 the magnitude of surface settlement was equal to 4.5% of the depth of the saturated backfill. It appeared that the settlement of the factory had resulted from the water table rising up from below the base of the backfill. This was confirmed by inspection of coal mining records which revealed the cessation of local deep mining in 1967 and all associated pumping by 1972. The water then rose, eventually inundating the lower part of the opencast backfill.

[Leigh Rainbow (1979), Charles (1984), Smyth-Osbourne, Mizon (1984), Charles Burford (1987)]

Dibdale West, Dudley

Dibdale West was one of many sites in the Dudley and Stourbridge areas of the West Midlands mined for coal, fireclay and ironstone worked from Middle and Lower Coal Measures strata. Opencast mining began at Dibdale West in 1968 and was completed in 1971, backfilling was carried out by self propelled box scrapers. The backfill consists predominantly of mudstone fragments in a stiff clay matrix with a smaller proportion of sandstone, siltstone, carbonaceous shale, sideritic clay ironstone and coal fragments.

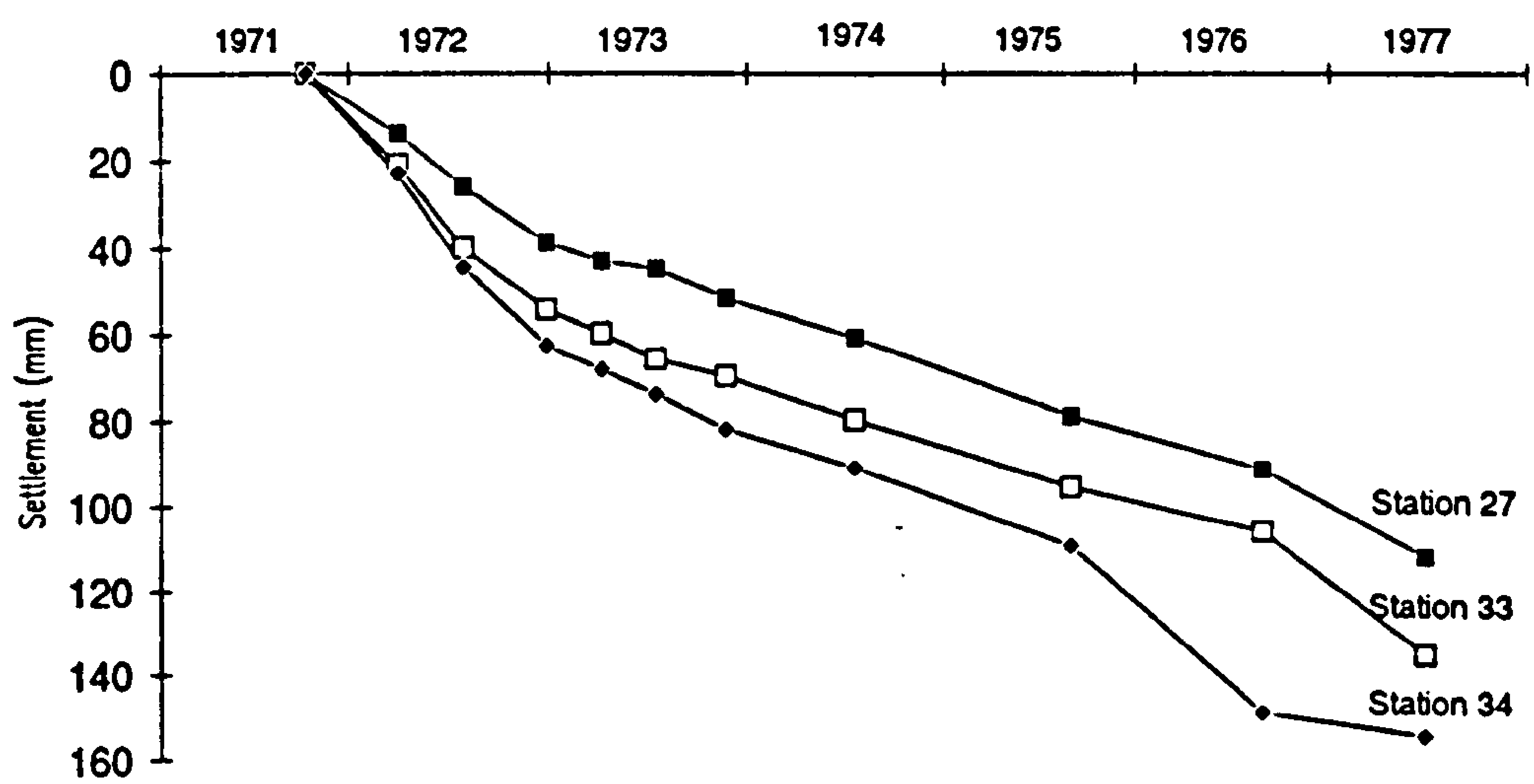


Figure A4. Dibdale West settlement observations (Knipe1979)

Systematic levelling surveys were made across the Dibdale West site for almost 6 years from 1971. Figure A4 presents time/settlement curves for 3 of the surviving most active levelling stations. The settlement at station 34 represents the greatest measured movement corresponding to 0.46% of fill depth. It can be seen that from mid to late 1975 an acceleration in settlement occurred which can be related to the cessation of pumping in an adjoining opencast area resulting in a rapid rise in water levels by an estimated 15 metres.

[Knipe (1979)]

Cogswell Dam

A dumped rockfill dam constructed in the 1930's having an average height of 38 metres. All the rock, according to the specification, was to be sound, hard, durable, angular quarried rock weighing not less than 2.5 tonnes per cubic metre, to be unaffected by air and moisture and of such toughness as to withstand dumping without shattering or breakdown, and to have a minimum compressive strength of 5000 psi.

On December 31st 1933, when 80% of the rockfill had been placed a major storm swept over the Pacific Ocean which by noon January 1st 1934 had yielded 38 cm's of rain at the dam. The rainstorm caused an immediate settlement of the crest of the fill by 4% of its height. Subsequent watering through infiltration wells increased the strain from 4% to 6%.

[Terzaghi (1960)]

Highvale, Alberta

To examine the post mining settlement characteristics of an opencast coal mine in Alberta, Canada, extensometers, piezometers and surface settlement stations were installed. The mining operation was carried out using draglines which cast the excavated overburden directly into adjacent mined-out areas. Thus producing a heterogeneous backfill consisting of the original strata which comprised of up to 5 metres of glacial lake sediments and till, overlying interfingering strata of mudstones, siltstones and sandstones. On completion of backfilling operations, due to cessation of pumping, the groundwater table rose up through the backfill. Figure A5 shows this rise together with the associated collapse settlement as measured at an extensometer located within the backfill. It can be seen that collapse is in the region of 1% strain between the magnets at 14 and 18 metres depth and the magnets between 9 and 14 metres depth. It is of interest to note that the collapse strain appears to proceed complete saturation. This can be explained by capillarity which will cause the backfill material above the groundwater table to become partly saturated which as this example implies is sufficient to cause collapse. Backfill already saturated and backfill considerably higher than the water table and thus assumed dry, display similar settlement behaviour.

[Thomson Sonnenberg (1987)]

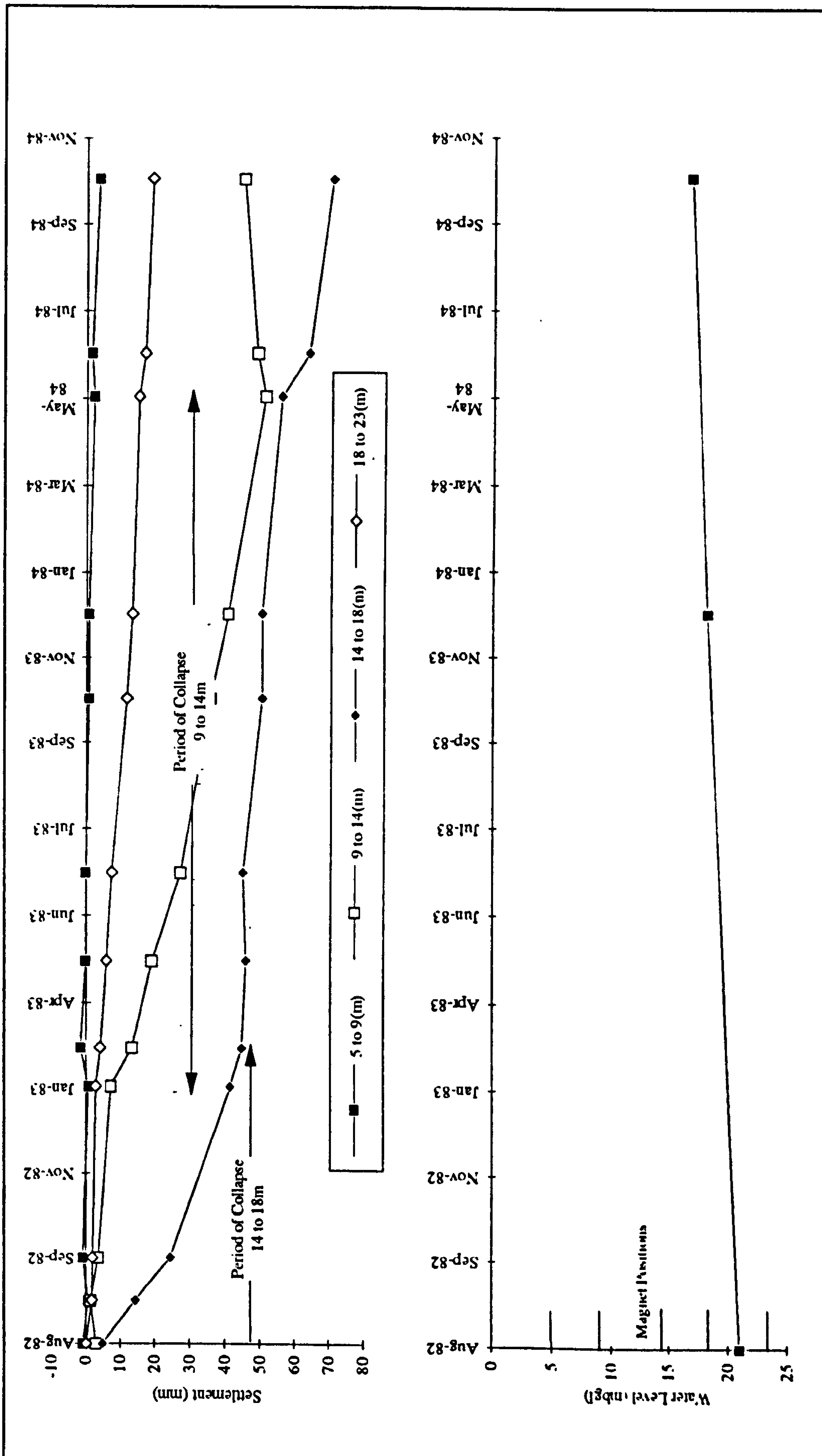


Figure A5. Settlement with time shown in relationship with the groundwater table rise - High Vale mine (Thomson Sonnenberg 1987)

Site plans and instrument locations

Barnabas
Bilston
Blindwells
Dixon
Flagstaff
Ketley Brook
Lounge
Newdale
Patent Shaft
Pithouse West

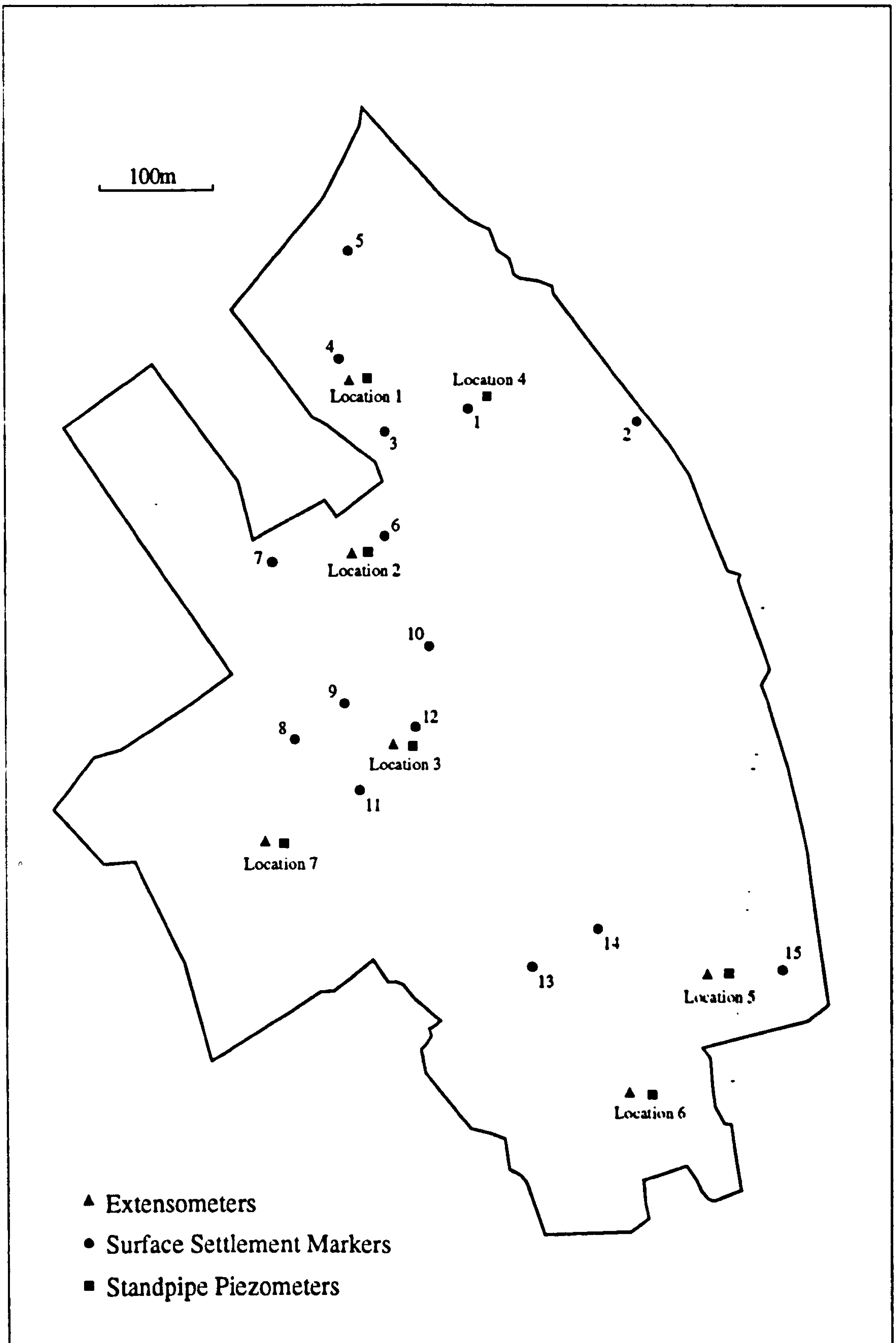


Figure B1. Barnabas site plan and instrument locations

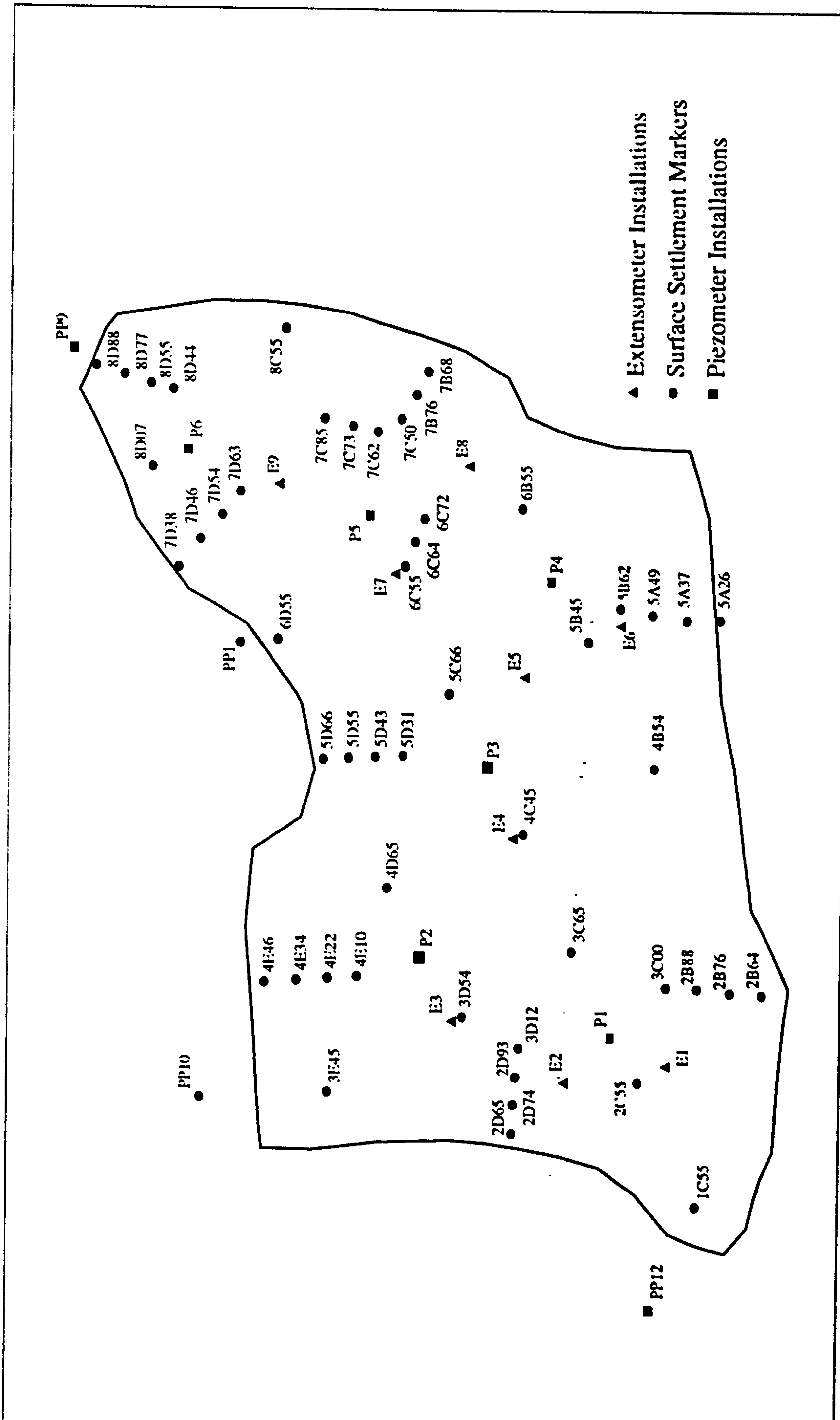


Figure B2. Bilston site plan and instrument locations

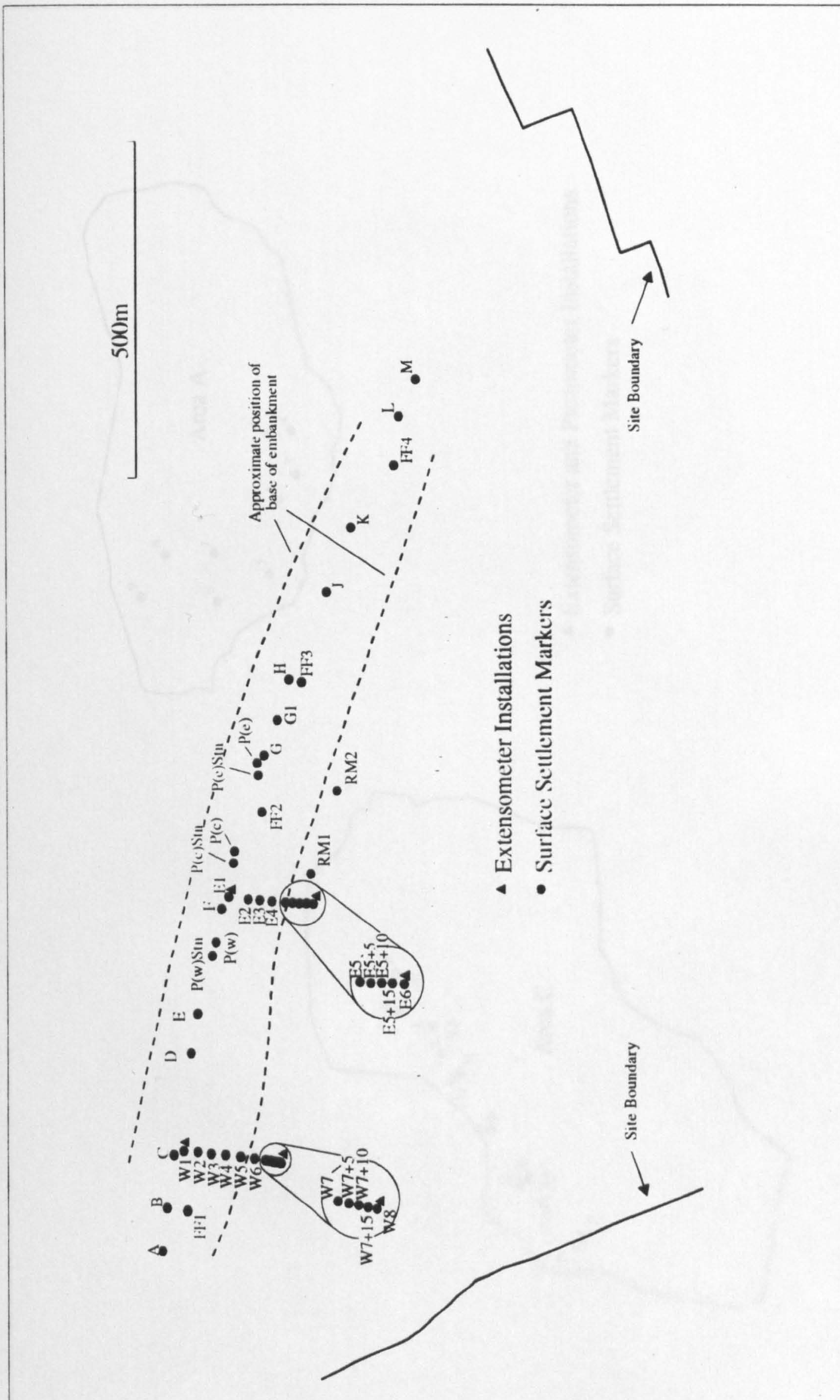


Figure B3. Blindwells site plan and instrument locations

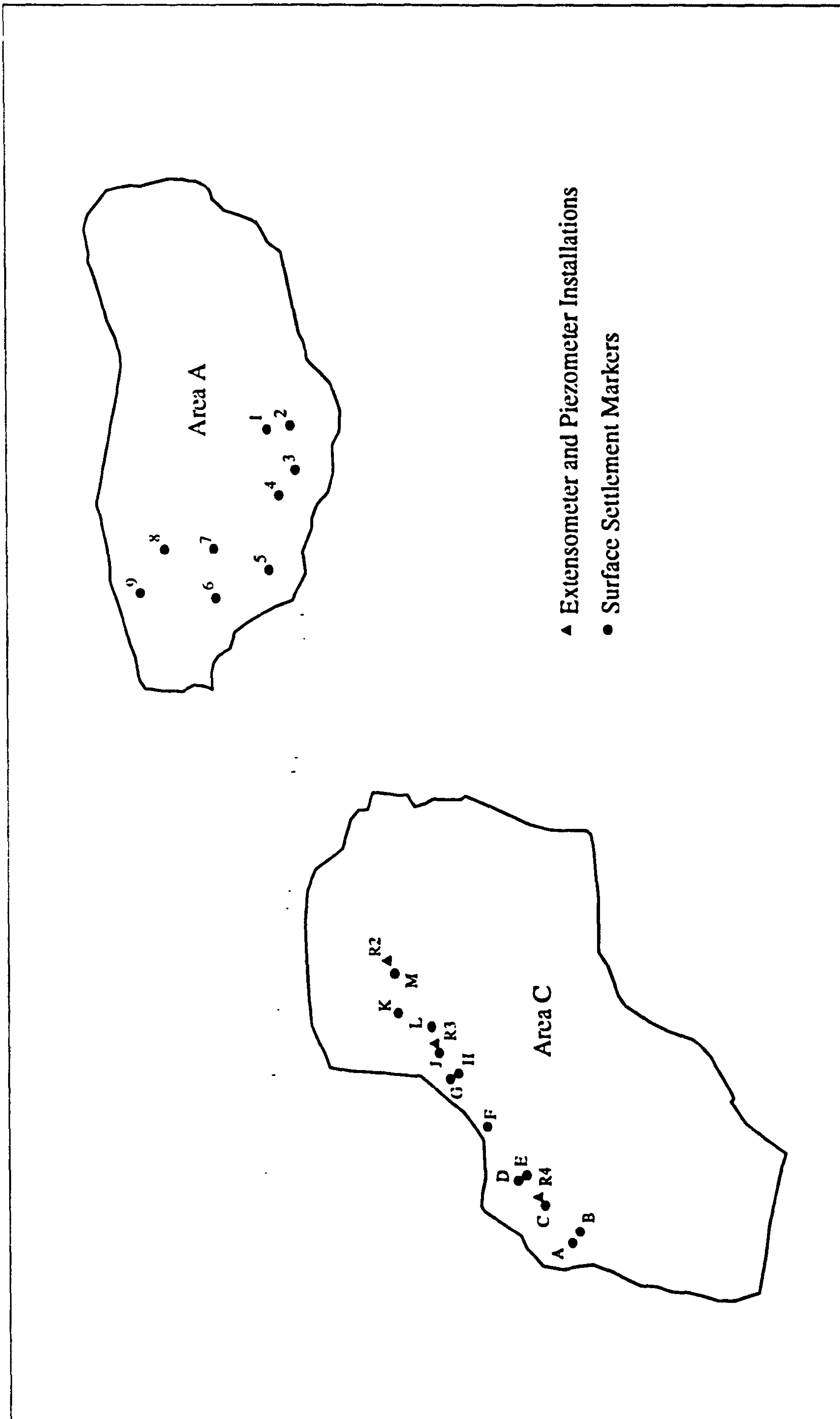


Figure B4. Dixon site plan and instrument locations

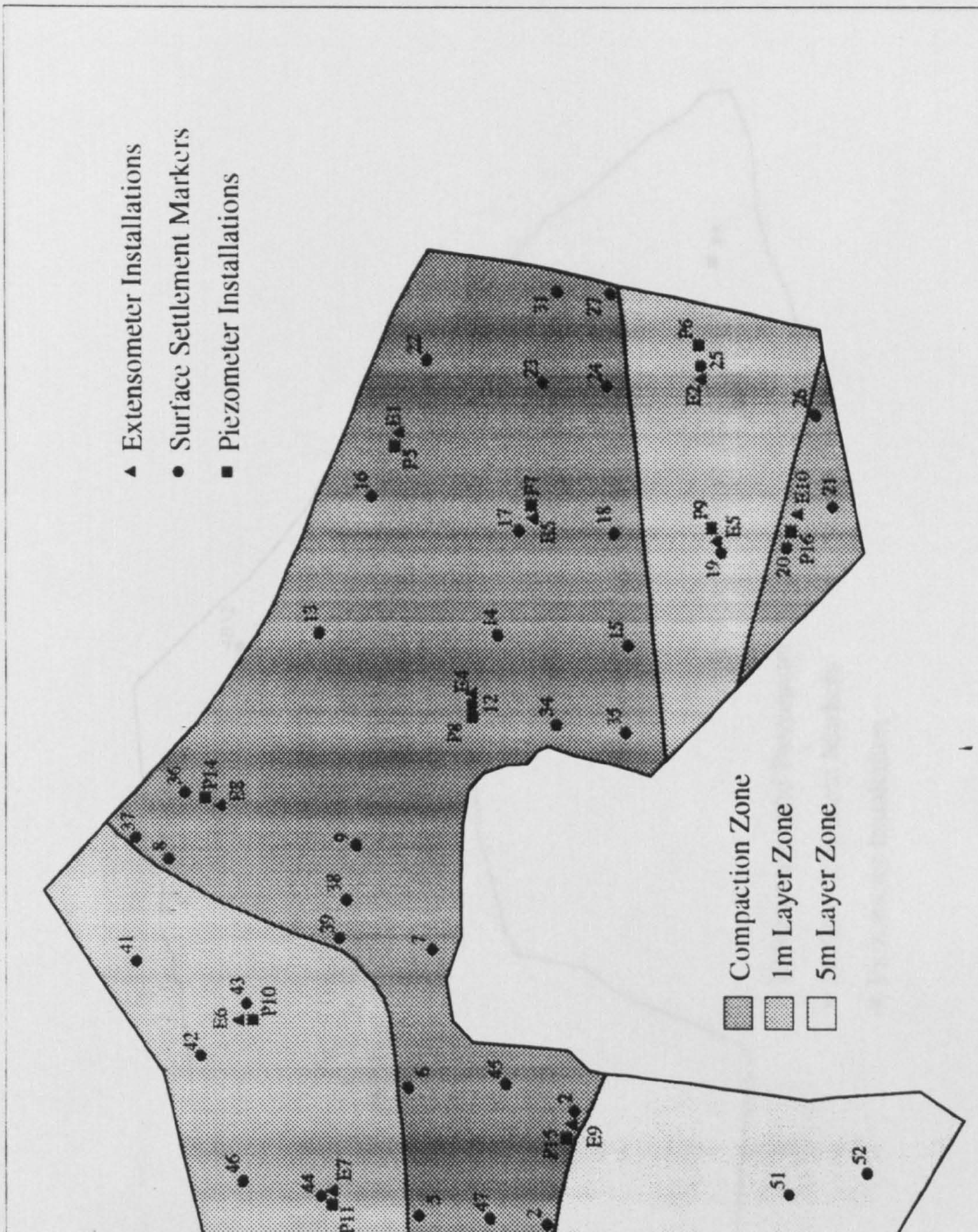


Figure B5. Flagstaff site plan and instrument locations

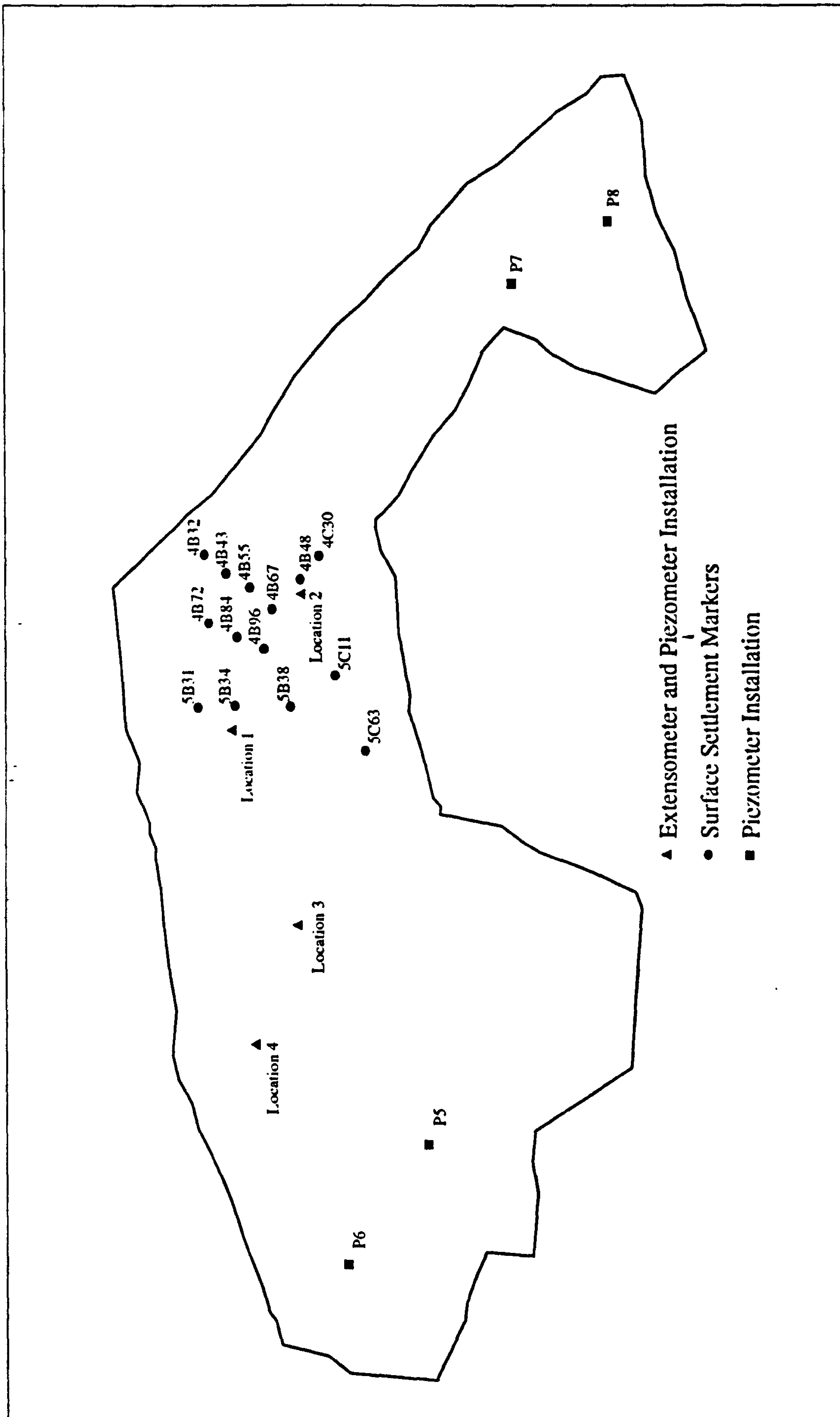


Figure B6. Ketley Brook site plan and instrument locations

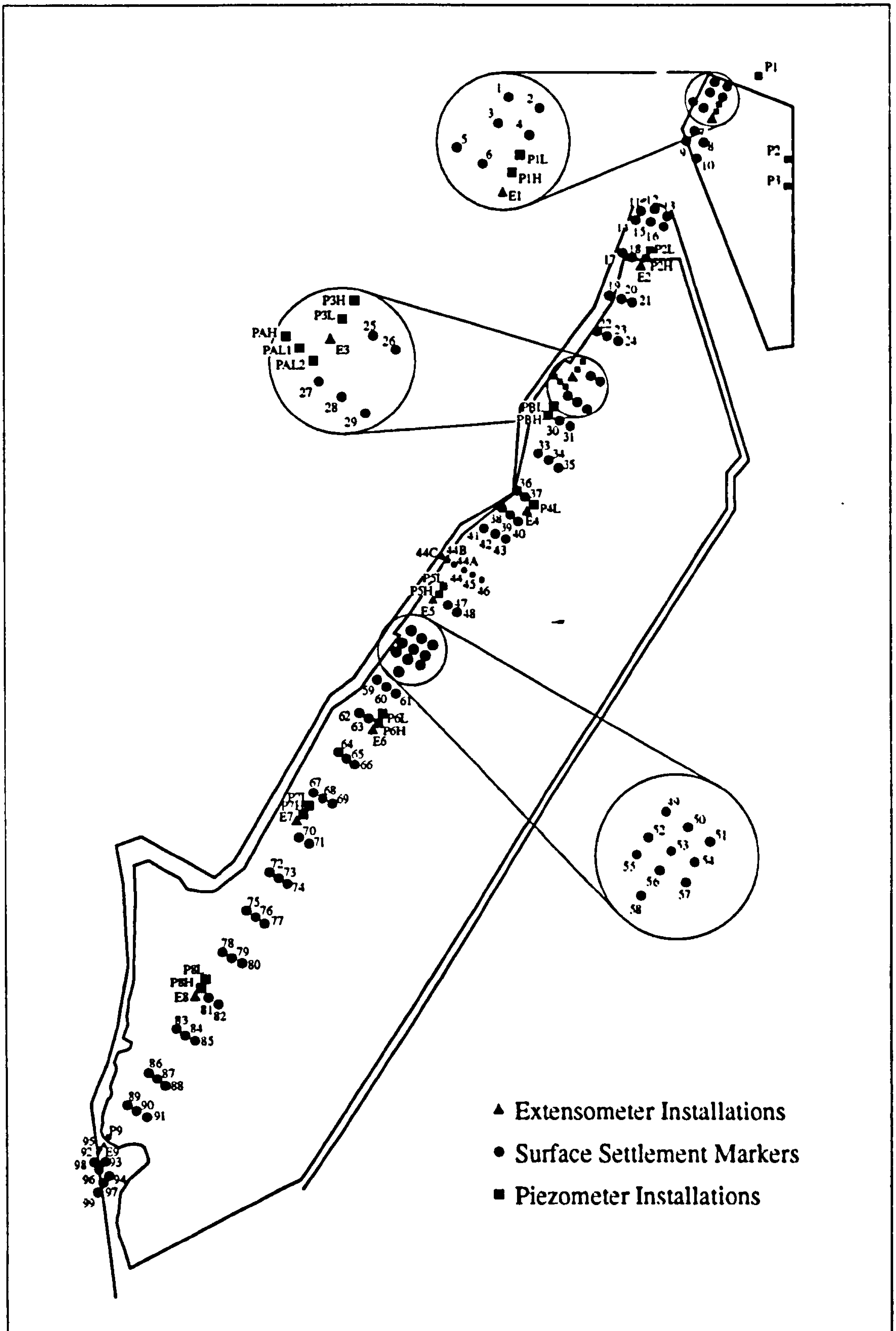


Figure B7. Lounge site plan and instrument locations

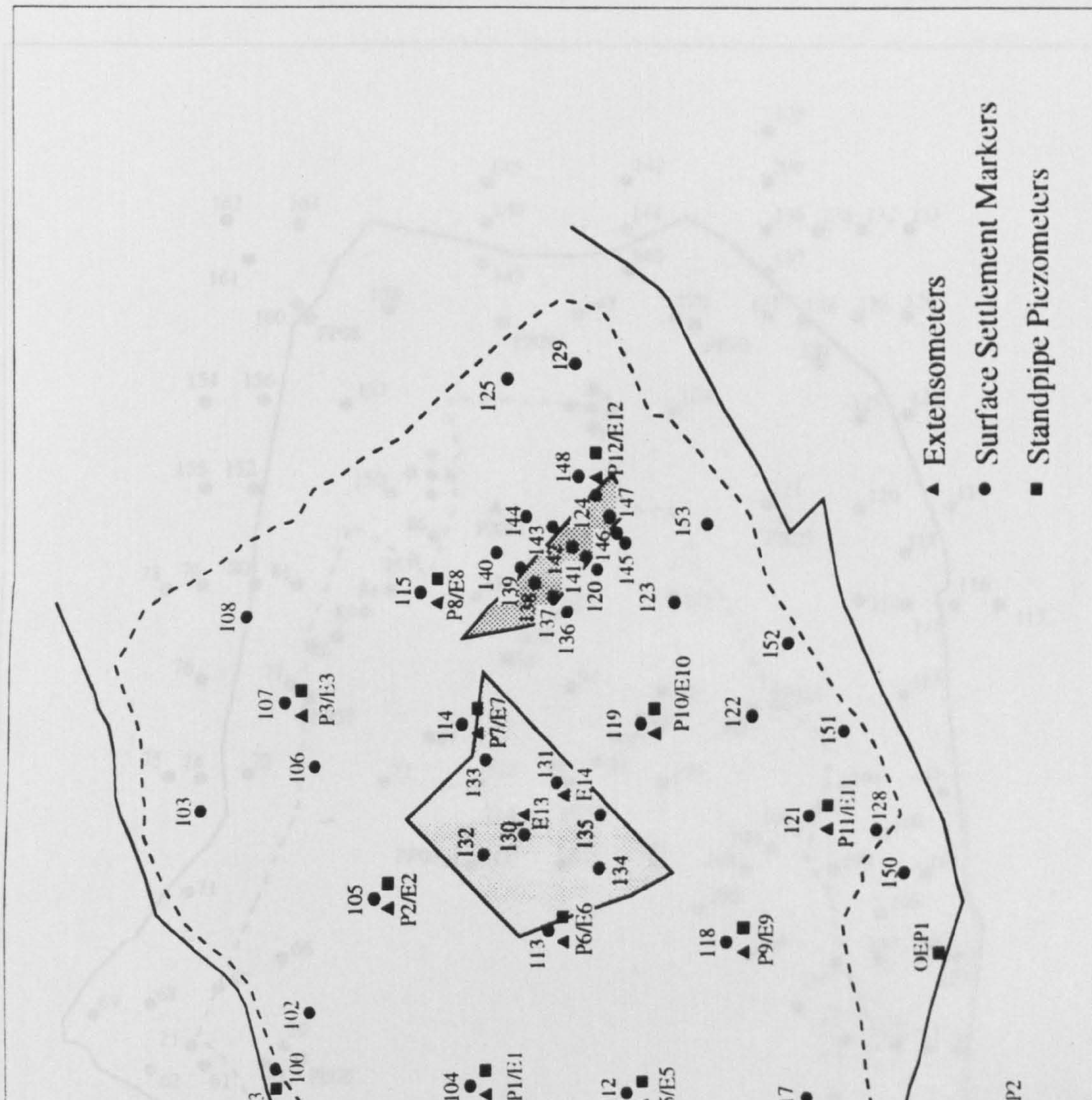


Figure B8. Newdale site plan and instrument locations

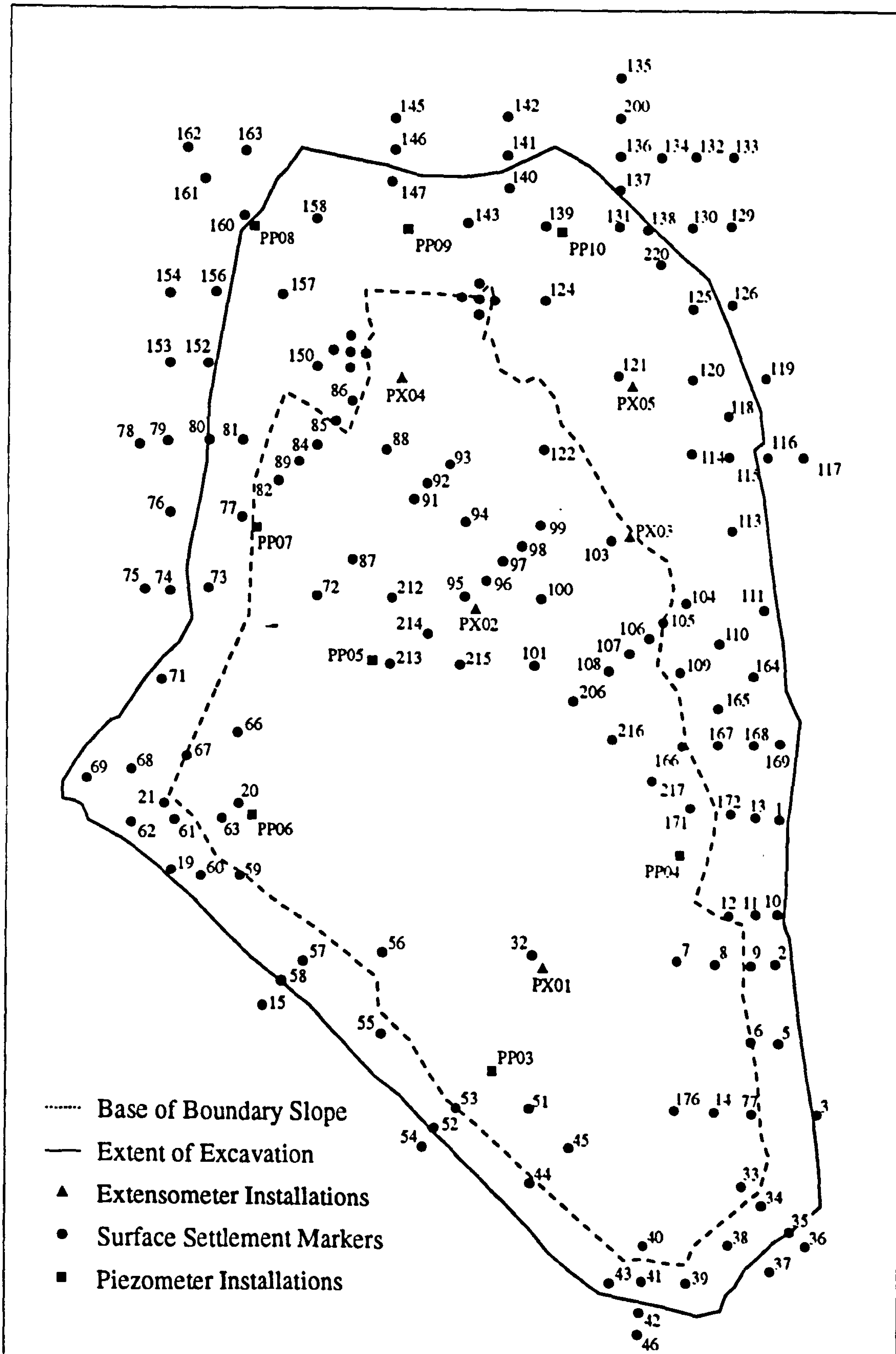


Figure B9. Patent Shaft site plan and instrument locations

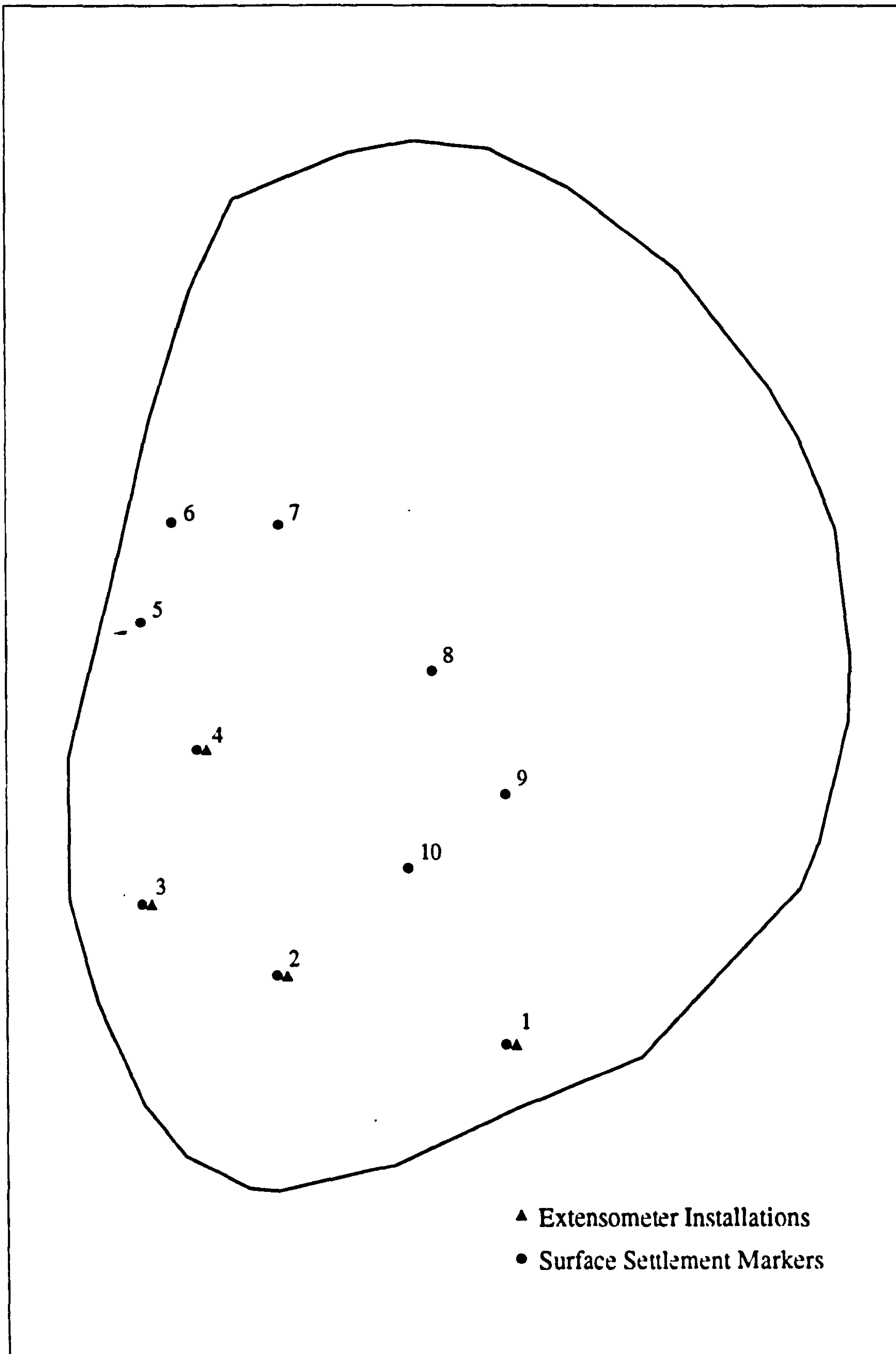


Figure B10. Pithouse West site plan and instrument locations

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