The Behaviour of Jacked Concrete Pipes during Site Installation

by

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Thesis submitted for the degree of Doctor of Philosophy at the University of Oxford

Pembroke College

Trinity Term 1992

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ABSTRACT

While much money and effort has been spent by manufacturers and users of pipe jacking equipment to develop suitable techniques, this work appears to be the first to study the method at full scale, in a scientific research programme. It has involved monitoring a series of five pipe jacks during construction. In each case a heavily instrumented pipe was incorporated into the pipe string to measure pipe joint stresses, pipe and joint compressions and contact stresses between pipe and ground. Total jacking loads and movements of the pipe string were also measured and all results correlated with a detailed site log, full tunnel alignment surveys, and observed ground conditions. The success of the site monitoring has been highly dependent upon the development of a suitable instrumentation and data acquisition system in conjunction with appropriate site procedures for working in the restricted and physically demanding pipe jack environment without undue disruption to normal site operations.

The build up of total jacking force is the result of highly complex soil-pipe interaction. The local interface stresses are essentially frictional in most ground conditions, and can be related to the shear strength of the ground. The problem is in determining the effective radial stresses which are affected by soil insitu stresses, stiffness and strength; groundwater conditions; rate of progress; pipeline misalignment and use of lubricants.

Relations between pressure distributions at pipe joints and measured tunnel alignments are presented. That small angular deviations between successive pipes cause severe localisation of stresses on their ends is clearly demonstrated. Careful back analysis shows that the linear stress approach of the Concrete Pipe Association of Australia can adequately match the measured stresses and could be used by pipe manufacturers to provide design data on allowable jacking forces for pipes on the basis of pipe size, packer properties, concrete strength and angular alignment. It is also clear from the small pipe barrel stresses that improved packing materials would allow more of the potential strength of pipes to be achieved.

Since relative angular rather than absolute deviations control transfer mechanisms between pipes, uncritical adherence to specifications based on absolute line and level is counter-productive.

"When you can measure what you are speaking about, and express it in numbers, you know something about it; but when you cannot measure it, when you cannot express it in numbers, your knowledge is of a meagre and unsatisfactory kind; it may be the beginning of knowledge, but you have scarcely, in your thoughts, advanced to the stage of science, whatever the matter may be."

- Lord Kelvin -

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Acknowledgements

The author is indebted to Dr G.W.E. Milligan for his much appreciated guidance and advice during the course of this study; it has been a pleasure working for such a stimulating and receptive engineer.

The financial support of SERC, the Pipe Jacking Association, and the five water companies, Northumbrian, North West, Severn Trent, Thames and Yorkshire is gratefully acknowledged, as are the contributions of the various contractors and pipe manufacturers during the various schemes.

In the development of the instrumentation, considerable assistance and support was provided by Mr Roy Smurthwaite, head of the department workshops, and his technicians. Mr Ron Morton, electronics technician, helped greatly with the design, manufacture, site installation and repair of the data collection system. The contact stress cells were designed jointly with Mr Clive Dalton of Cambridge Insitu Ltd. Thanks are also due to Dr Andrew Bond (formerly of Imperial College) for providing details and advice on the pore pressure probe design and the many commercial organisations who have supplied equipment to the project and provided excellent after sales support.

Special thanks go to Mr Chris Eggleton, research student, who provided invaluable assistance on site and during routine calibration of the instruments, Miss Nicola Andrews for part typing the thesis and Mrs Judith Takacs for preparation of the drawings. The whole programme was only possible with the involvement of individuals too numerous to mention, in the management group, on the various sites, at the pipe manufacturer's works and throughout the University.

On a personal note I wish to express my gratitude to past and present members of the Soil Mechanics research group for providing a stimulating work and social environment and Mott MacDonald for their support and leave of absence to pursue this project.

Finally, I wish to express sincere gratitude to my wife Andree, and son Matthew for providing encouragement, support and the necessary escape from the frustrations associated with research, and to them I dedicate this thesis.

Publications

The following conference papers and reports have been published as a result of this research.

Norris, P. and Milligan, G.W.E. (1991). Field instrumentation for monitoring the performance of jacked concrete pipes. FMGM 91, Proc. 3rd. Int. Symp. on Field Measurements in Geomechanics, Oslo.

Milligan, G.W.E. and Norris, P. (1991). Concrete jacking pipes, the Oxford research project. Proc. 1st. Int. Conf. on pipe Jacking and Microtunnelling, London.

Norris, P. (1992). Instrument design, manufacture and calibration for use in monitoring the field performance of jacked concrete pipes. Report No. OUEL 1919/92, Department of Engineering Science, Oxford University.

Norris, P. and Milligan, G.W.E. (1992) Pipe end load transfer mechanisms during pipe jacking. Proc. Int. Conf. on Trenchless Construction, No-Dig 92, Washington.

Norris, P. and Milligan, G.W.E. (1992) Frictional resistance of jacked concrete pipes at full scale. Proc. Int. Conf. on Trenchless Construction, No-Dig 92, Paris.

Nomenclature

a	Initial packing material thickness
b	elastic contact width between pipe and soil
c'	Cohesion of the soil
c_u, s_u	Undrained shear strength
E_c	Young's modulus of concrete
E_j	Equivalent joint elasticity coefficient
E_p	Young's modulus of packing material
f_{cu}	Concrete characteristic cube strength
h_1, h_2, h_3	Joint compression measured with the joint movement indicators
k	Terzaghi's coefficient of soil load
L	Pipe length
P_{TOTAL}	Total radial load on pipe
SIGMA	Total radial interface stress
SIGMA'	Effective radial interface stress
t_{95}	Time required for 95% response to a step load
TAU	Interface shear stress
T_γ	Dimensionless tunnel stability number
W	Weight of pipe per metre run
z	Diametrical contact width at pipe joint
α	Radial angle to the point of maximum compression
$\alpha.s_u$	"adhesion" between pipe and clay
β	Angular deflection at pipe joints
β_{em}	Pipe end squareness angle at socket end
β_{es}	Pipe end squareness angle at spigot end
γ	Unit weight of the soil
γ_w	Unit weight of water

δ	Angle of skin friction between pipe and soil (in total stress terms)
δ'	Angle of skin friction between pipe and soil (in effective stress terms)
Δa	Packing material compression
ΔL	Pipe deformation
ε_c	Pipe longitudinal strain
μ	Coefficient of friction between pipe and rock
ξ	Deviation angle between pipe invert and point of contact in rock
σ_c	Pipe longitudinal stress
σ_H	Horizontal soil stress
σ_j	Maximum stress at pipe joint
σ_{jo}	Joint stress for uniform load
σ_P	Radial soil stress
σ_T	Required tunnel support pressure
σ_V	Vertical soil stress
φ, φ_U	Undrained angle of internal friction of the soil
φ'	Drained angle of internal friction of the soil

CHAPTER 1:

INTRODUCTION

1.1 Pipe jacking with concrete pipes and the need for research

Pipe jacking is a technique for forming small diameter tunnels by pushing or jacking pipes through the ground from a thrust pit to a receiving pit. Pipes are advanced using hydraulic power packs located in the thrust pit as the ground in front of the pipeline is mined. Excavation is normally carried out within a shield using either pneumatic tools or a tunnel boring machine, with the spoil being transported along the pipeline to the surface. Steering and adjustments for line and level are made at the shield using jacks, in conjunction with frequent surveying to fixed reference points. The technique is depicted in Figure 1.1.

Pipe jacking is suited to short lengths of tunnel typically less than 150m between jacking points and is often used for sections of pipe under embankments, roads and railways, where open cut methods would be particularly uneconomic because of the need to keep traffic moving continuously. Likewise there have been many occasions when the method has been used for small sections of long tunnels that pass below buildings. One of the main advantages of using pipe jacking, particularly under existing structures, is that a rigid liner is provided immediately after excavation, in the shape of the pipe, with relatively little over excavation (typically less than 20mm around the circumference).

There is now a tendency for longer and larger contracts to be undertaken using pipe jacking as an alternative to open cut methods either for direct economy or to reduce disturbances at the surface. Wallis (1982) reports an 1800mm diameter pipejack, 460m long in London clay. Winfield (1986) records 690m of 1950mm diameter pipes being

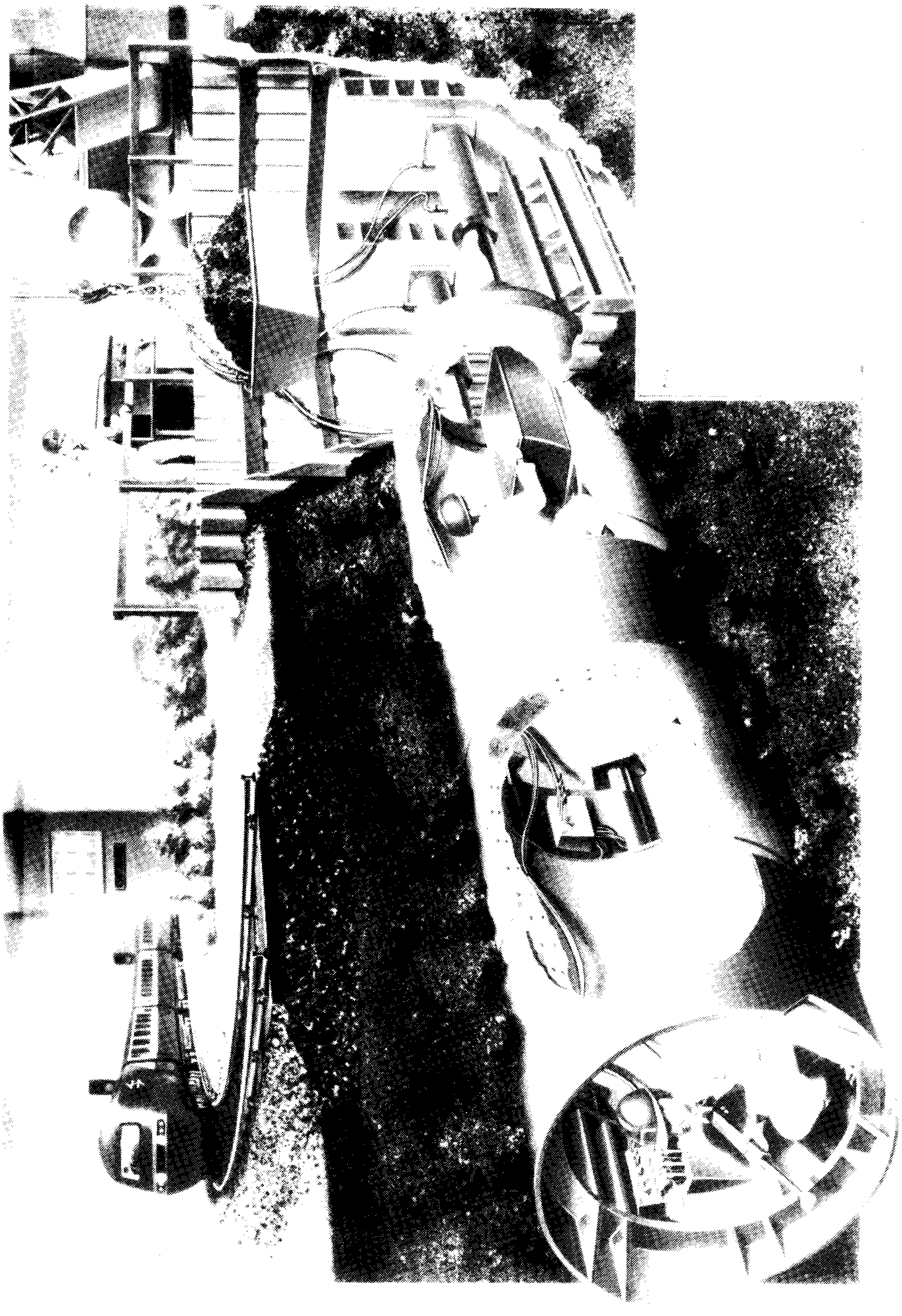


Figure 1.1 Typical arrangement of pipejacking equipment.

jacked through sandstones and siltstone. Perhaps one of the most impressive recent examples is the jacking of a 14.4m wide by 9.3m high by 35m long precast concrete box culvert through a clay embankment supporting a four track railway, New Civil Engineer (1989). For the larger diameter pipes and long lengths of thrust, either intermediate jacking stations, which are built into the lining and subsequently removed, or lubrication, usually bentonite, may be necessary to reduce the jacking forces, Durden (1982). The insertion of polythene sheeting between the ground and the pipeline has also been used successfully for a similar purpose, Tohyama (1985).

Although the method has many advantages over conventional methods of tunnelling its wider use is being held back by a lack of understanding of many factors affecting the installation and performance of such tunnels, and loss of confidence by some specifying authorities resulting from unexpected failures, Shullock (1982).

The need for research in several areas was first reported by Kirkland (1982) and then in a CIRIA report on the state of the art in pipe jacking (Craig 1983). Industry, represented by the Pipe Jacking Association (PJA), has actively promoted and supported the case for research, which resulted in the first stage of the Oxford pipejacking research; a laboratory based study of model pipes starting in February 1986 and finishing three years later, Ripley (1989). The most important findings from this work emphasised the need for suitable packing material in joints along with careful control of pipe alignment, and the superiority of steel banded butt joints to inwall spigot and socket joints for the transmission of large jacking forces.

1.2 Phase two of the Oxford research project

In April 1989, a second phase of the research was initiated to confirm under actual field conditions the findings from the laboratory work and investigate aspects of

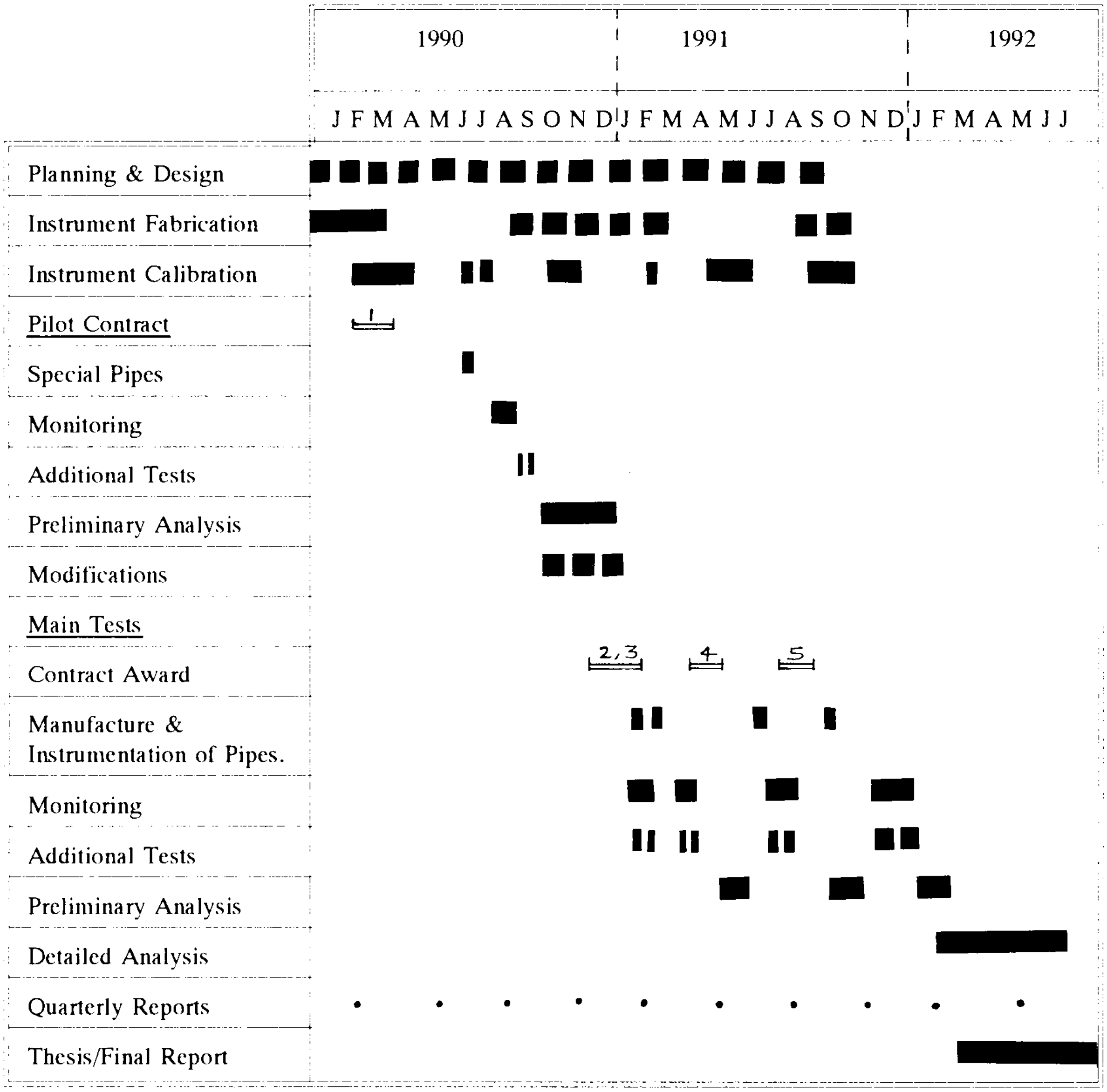


Figure 1.2 Research programme

performance that can only be studied realistically at full scale. This phase involved incorporating an instrumented pipe into pipejacked tunnels during construction on five sites. The work was supported by the Science and Engineering Research Council (SERC) in collaboration with the PJA and five water service companies, Northumbrian, North West, Severn Trent, Thames and Yorkshire. Close cooperation between clients, contractors and researchers has been essential to the success of the project.

The main uncertainties in pipe jacking arise because the alignment of pipes can never be perfect. The loads between pipes are not transmitted uniformly, and the interaction between soil and pipe is such that frictional forces resisting the forward movement of the pipe string may be greatly increased. These two effects interact with each other to increase the jacking loads and cause stress concentrations in the pipes. The main purpose of the research is therefore to investigate the load transfer between pipes and the contact pressures between pipes and soil. The measurements are related to detailed line and level surveys of the pipeline and to local ground conditions. The aim is to allow better prediction and control of pipe jacking operations in the future.

The progress of the overall programme of research is presented in Figure 1.2. The first year saw completion of the design, manufacture and calibration of the instrumentation. Minor contractual delays meant that the first drive was about three months late in starting, with the next year and a half required for the fieldwork on the five pipe jacks. To allow adequate time for analysis and writing up of the work the three year programme was extended by four months.

The contract cost for phase two was £219,500 as shown in Table 1.1 ; SERC carried about 20% of the total, the PJA about 40%, and the remainder was divided equally between the five water companies. In addition, each site operation involved additional costs of between £10,000 and £20,000, mainly for modification of a standard pipe to incorporate the instruments, provision of a liner inside the instrumented pipe to protect the instruments, some delay and loss of production by the contractor, and costs of retrieving the instruments at the end of the operation. By the end of the contract, each water company had carried these additional costs for one site.

The programme of research was overseen by a management group with two representatives from the PJA, one from Oxford, and one from each of the water companies. The group met four times a year to discuss progress. The funds were administered via a

Costs	£
Research assistant, including expenses	67,500
Oxford costs including overheads (supervision, technician support, reprographic, and secretarial assistance)	69,000
Instrumentation and data logging	63,000
Contingency	20,000
TOTAL	219,500
Funds	
Science and Engineering Research Council	44,500
Pipe Jacking Association	85,000
Water Authorities (Companies) 5 x 18,000 =	90,000
TOTAL	219,500

Table 1.1 Costs and Funding of research work.

special account with Thames Water, apart from the SERC contribution which was used to support the author directly through the University. Financially, the project, including the extension, was completed within the original budget.

This report on the site based research begins with Chapter 2 which provides a background to the pipe jacking method of construction, its uses and limitations. It continues to present the current methods for determining the magnitude of pipe jacking forces in different ground conditions and the effect of pipeline misalignment on load transfer at pipe joints. A review of previous site based tunnel, piling and pipe jacking instrumentation projects is then presented which highlights the necessity of minimising contractual pressures by thorough planning and clear communication at all stages of the work.

Chapter 3 sets out the overall concept of the instrumentation and presents details of the design, manufacture and calibration phases. All instruments have been designed to operate successfully in the tunnel environment, have minimal effect on the property to be measured, be sufficiently accurate, be simple to calibrate and to disrupt normal site operations as little as possible. The selection of sites and development of suitable site procedures is included as Chapter 4, which also provides an assessment of the effectiveness of the site operations.

The results of the fieldwork are covered in Chapters 5 to 7. Chapter 5 deals with pipe end load transfer mechanisms during jacking. Relationships between pressure distributions at pipe joints and measured tunnel alignments are presented and the effectiveness of packing material and the importance of good control of line and level assessed.

Measurements of total jacking resistance and localised contact pressures between pipes and soil are included as Chapter 6. Detailed research objectives of the soil-pipe interaction data include determination of the areas of contact between the pipe and ground in both granular and cohesive soils, the magnitude of the normal and shear stresses developed at these contacts and how these relate to the overall pipeline resistance. The effects of stoppages and lubrication on both the local and global measurements are evaluated.

The resulting stresses and strains in individual pipes are the subject of Chapter 7, while the conclusions from the research and recommendations for future work are drawn together in Chapter 8.

CHAPTER 2:

LITERATURE REVIEW

2.1 Introduction

Tunnels constructed in the UK may be grouped into three size categories:

- Small diameter - up to 3m internal diameter, for sewer, water and cable tunnels.
- Medium diameter - 3-6m internal diameter, for underground railways and associated tunnels.
- Large diameter - 6m upwards internal diameter, for road and main line railway tunnels, underground chambers.

All tunnels, except in sound unjointed rock, where the ground is self supporting, are lined with a primary lining which is designed to support the ground loads and temporary loads that may occur during installation and for the design life of the structure. Such linings should also exclude or control the ingress of water into the tunnel.

Pipe Jacking is a technique for providing a rigid primary lining in the form of a pipe which eliminates the need for temporary ground support or secondary linings. It is principally used for small diameter tunnels in soft ground. A summary of the conditions in which pipe jacking is a competitive alternative to other forms of primary lining is included as Table 2.1, Craig and Muir Wood (1978).

The earliest recorded use of the Pipe Jacking method was in America about 1910, Richardson and Mayo (1941). The basic principles of the technique have been presented in detail by the American Concrete Pipe Association (1960), Richardson (1970), Hough (1974), Drennon (1979) and Clarkson and Thomson (1983). The British Pipe Jacking Association published notes giving guidance on design and practice (Pipe Jacking Association 1981 and 1986). The increasing use of modern technology and a competitive

Tunnel use	Sewer	Water	Cable	Underground railway	High speed railway	Road	Pedestrian Subways, Passages, Concourses etc.
INTERNAL FINISH	Smooth	Smooth	As primary lining or special profile	As primary lining	Smooth	Aesthetic, waterproof	Aesthetic waterproof
GROUND CONDITIONS SOFT GROUND							
(a) Drift above watertable	Bolted Concrete Smooth Bore concrete Pipe Jacking	Bolted Concrete Smooth Bore concrete Pipe Jacking	Bolted Concrete Smooth Bore concrete Pipe Jacking	Bolted Cast Iron (Bolted Concrete)	Bolted Cast Iron (Bolted Concrete)	Bolted Cast Iron (Bolted Concrete)	Bolted Concrete Bolted Cast Iron
(b) Drift below watertable	Bolted Concrete Pipe Jacking (Smooth Bore concrete)	Bolted Concrete Pipe Jacking (Smooth Bore concrete)	Bolted Concrete Pipe Jacking (Smooth Bore concrete)	Bolted Cast Iron (Bolted Concrete)	Bolted Cast Iron (Bolted Concrete)	Bolted Cast Iron (Bolted Concrete)	Bolted Cast Iron (Bolted Concrete)
(c) Silts and clays	Bolted Concrete Smooth Bore concrete Pipe Jacking	Bolted Concrete Smooth Bore concrete Pipe Jacking	Bolted Concrete Smooth Bore concrete Pipe Jacking	Bolted Cast Iron (Bolted Concrete)	Bolted Cast Iron (Bolted Concrete)	Bolted Cast Iron (Bolted Concrete)	Bolted Cast Iron (Bolted Concrete)
(d) Very soft clays	Pipe Jacking (Bolted Concrete)	Pipe Jacking (Bolted Concrete)	Pipe Jacking (Bolted Concrete)	Bolted Cast Iron Bolted Steel	Bolted Cast Iron Bolted Steel	Bolted Cast Iron Bolted Steel	Bolted Cast Iron Bolted Steel
(e) Stiff fissured clays	Bolted Concrete Smooth Bore concrete Expanded Concrete	Expanded Concrete Bolted Concrete Smooth Bore concrete	Expanded Concrete Bolted Concrete Smooth Bore concrete	Expanded Concrete or Cast Iron Bolted Concrete (Bolted Cast Iron)	Expanded Concrete or Cast Iron	Expanded Concrete	Expanded Concrete Bolted Concrete
ROCK							
(a) Very weak to moderately strong	Bolted Concrete Smooth Bore concrete (Cast in-situ concrete)	Bolted Concrete Smooth Bore concrete (Cast in-situ concrete)	Bolted Concrete Smooth Bore concrete (Cast in-situ concrete)	Expanded grouted Concrete lining Cast in-situ concrete	Expanded grouted Concrete lining Cast in-situ concrete	Expanded grouted Concrete lining Cast in-situ concrete	Bolted Concrete Cast in-situ concrete
(b) Strong	Cast in-situ concrete (Bolted Concrete) (Sprayed Concrete)	Cast in-situ concrete (Bolted Concrete) (Sprayed Concrete)	Cast in-situ concrete (Bolted Concrete) (Sprayed Concrete)	Expanded grouted Concrete lining Cast in-situ concrete (Bolted Concrete) (Sprayed Concrete)	Expanded grouted Concrete lining Cast in-situ concrete (Bolted Concrete) (Sprayed Concrete)	Expanded grouted Concrete lining Cast in-situ concrete (Sprayed Concrete)	Cast in-situ concrete (Bolted Concrete)
(c) Very strong and extremely strong	unlined (Cast in-situ concrete)	unlined Sprayed Concrete (Cast in-situ concrete)	unlined Sprayed Concrete (Cast in-situ concrete)	unlined Cast in-situ concrete Sprayed Concrete Rock bolting	unlined Cast in-situ concrete Sprayed Concrete Rock bolting	unlined Cast in-situ concrete Sprayed Concrete Rock bolting	Cast in-situ concrete Rock bolting

Notes: 1. Linings shown in brackets may be used for the type of ground but other forms may be preferred on economic grounds.
2. For water tunnels a steel lining may be necessary where for particular conditions, the ratio of the overburden pressure to the water pressure is below an acceptable factor.

Table 2.1 Suggested methods of lining tunnels (Craig and Muir Wood (1978)).

tendering market have led to many innovative methods being introduced in recent years.

Richardson and Scruby (1981) reported the development of the Uni-Tunnel system in which pipes are jacked forward by inflatable bladders positioned between successive pipes. A new jointing profile is reported by Cole (1986) and White et al (1988) presented details of the use of new joint packing materials and microtunnel technology using unreinforced concrete pipes. More diverse applications of the technique include a novel tunnel forming method using multiple pipejacks to form an arch roof to an underground station in Milan, New Civil Engineer (1990) and the replacement of a single brick arch rail bridge by a series of precast box tunnels, Ghosh and Madhusudhan (1991).

2.2 Concrete jacking pipes

Precast concrete jacking pipes are generally cast in manufacturers' works and transported to site by road. Good quality control and adequate precautions for the curing of the concrete and regular checking of the moulds can produce accurately cast pipes of consistent concrete strength. The pipes are manufactured by centrifugal spinning or vertical casting using concrete with a 28 day characteristic cube strength greater than 60 N/mm^2. Spirally wound reinforcement spot welded to longitudinal steel to form internal and external cages are often used to prevent damage during the temporary handling and installation stages.

British pipe manufacturers currently assess compliance of a pipe's strength to withstand ground loading imposed on an installed pipe. Class H pipes are used for jacking and are designed to meet Marston (1930) loading conditions. They are designed to higher standards and strengths if ground conditions or client specifications impose more onerous design requirements. Allowance for installation loads receives scant attention, possibly due to the uncertainty in predicting loads, stresses and their distribution throughout the jacking cycle. The lack of data required to establish a suitable test to assess a pipe's ability to sustain installation end loads is acknowledged in the recent British Standard, BS5911 : Part 120 (1989). The forward to the standard states "The joint face test is included in the absence of a suitable jacking strength test. Research is presently being undertaken in order to devise such a test and this may be incorporated into a future revision".

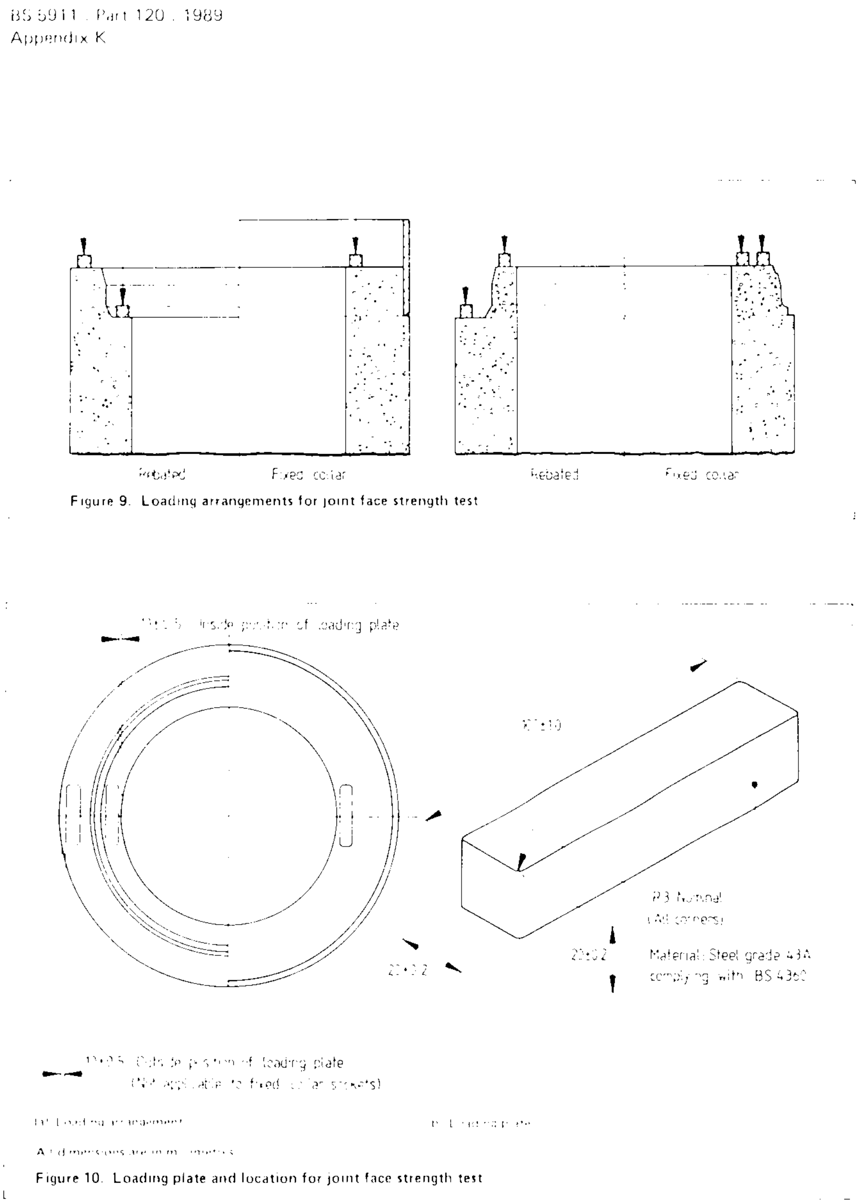


Figure 2.1 Arrangement for joint face test (BS5911: Part 120: 1989).

The drafting committee settled on the joint face strength test as a way of determining the "quality" of the concrete in the jacking face. The test is illustrated in

Figure 2.1 and consists of loading areas at both ends of a pipe through a 100mm x 20mm x 20mm steel plate. The average ultimate strength of the concrete face must not be less than 100 N/mm^2 and no individual result can fall below 70 N/mm^2. The standard points out that the resulting strengths should not be used to calculate permissible jacking loads.

For the purposes of installation it is usual to assume that ideal jacking conditions exist in which the pipes are subjected to an evenly distributed axial load around the pipe circumference. The worst case is assumed to be in the pipe nearest the thrust pit which will be subjected to the maximum jacking force from the rams. In a typical design example for an 1800mm internal diameter pipe, a jacking load of 1000 tonnes produces an average direct compression on a full pipe section of 9N/mm^2. British pipe manufacturers specify their pipes as being capable of sustaining a uniformly distributed end stress of between 10 and 15 N/mm^2 (dependent upon manufacturer and pipe diameter). These figures appear conservative if loads are applied to the full end area of the pipe, but allow for end tolerances and jacking misalignment in an arbitrary fashion. It is one of the principal aims of the Oxford research to enable manufacturers and contractors to offer clients a theoretically based method for assessing the axial load capacity of jacking pipes.

Precast concrete is a relatively impermeable material and therefore problems with leakage of water will normally be at pipe joints or at any cracks in the concrete. The functional requirements of a joint on a jacked pipe are adapted from Clarke (1968):

- It should be designed to permit angular and axial movements large enough to tolerate the maximum displacements likely to occur, without damage or loss of water tightness
- It should withstand the force applied during installation without damage
- It should remain efficient throughout its working life
- It should be simple to make and dismantle in the limited space of the thrust pit.

Traditionally, jacked pipe joints in the UK have been the in wall spigot and socket type as shown in Figure 2.2a. Disquiet in the industry about the performance of the in wall

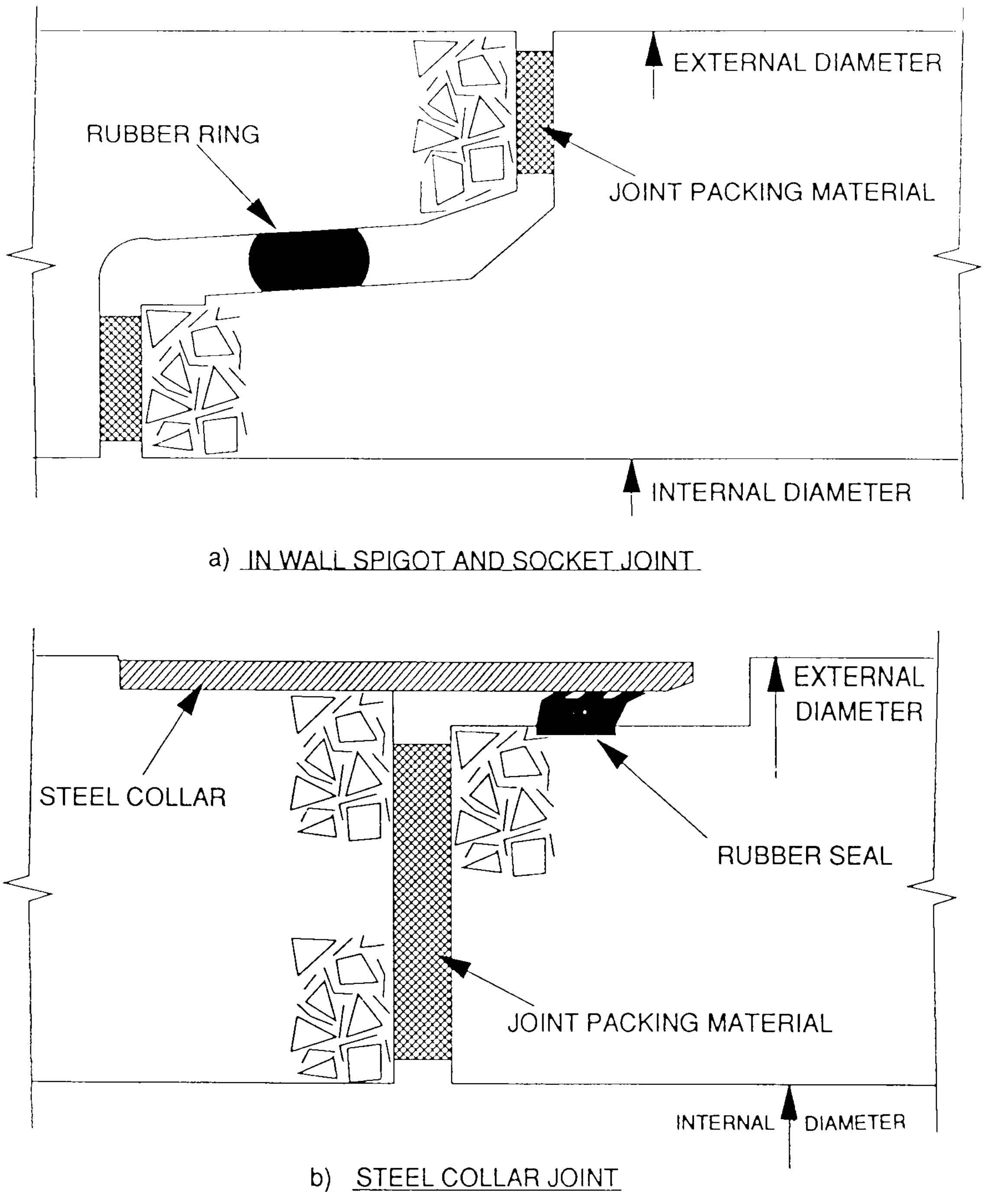


Figure 2.2 Common UK pipe joints.

joint and its ability to transmit longitudinal loads efficiently has led to the introduction of the steel collar joint, Figure 2.2b. The main reason for the new joint detail is the belief that jacking loads are better transmitted through the centre of the pipe wall rather than along its edges. The CIRIA technical reports 112 (1983) and 127 (1987) provide further details of these joints and examples of other less common jointing arrangements.

The functional requirements of the joints are in part contradictory. The flexibility offered by the joint enables the pipeline to accommodate future ground movements without imposing substantial stresses into the pipes, while conversely misalignment of pipes relative to each other during the jacking process can lead to local stress concentrations in both the ground and joints. These have traditionally been minimised by seeking to limit the maximum errors in line and level to typically ±75mm. Compressible joint packing materials are recommended by all pipe manufacturers as an aid to distributing end stresses over larger areas during jacking. However, not all are prepared to specify the type of material or advise on the details of its use. This aspect has not been covered in the recent BS5911 : Part 120.

2.3 Pipe jacking forces

Accurate predictions of the force required to jack a pipeline are not only structurally important but also enable the contractor to ensure that his equipment and site set up (including provision of interjacks and thrust wall arrangements) are capable of producing the required thrust to complete the drive without damage to pipes and joints from excessive stress concentrations. The total jacking load depends upon both the force required to push the shield into the excavation, commonly referred to as face resistance, and the frictional resistance along the pipe length.

The amount of resistance encountered at the face depends upon ground conditions and the measures required to support the face. In hand drives it is predominantly related to the edge cutting resistance of the jacking shield and overall size is important as larger dimensions produce larger face resistances. Typical estimates vary from 100 to 400 tonnes, for diameters from 900mm to 2550mm, Auld (1982). In machine drives the face pressure required to support the ground must be taken into account.

Currently UK pipe jacking contractors use empirical methods to predict friction forces. These express the required thrust in terms of frictional resistance per unit surface area of pipe. Typical values of friction resistance for different ground conditions are given in Table 2.2.

Ground Type	External load (kPa)
Rock	2-3
Boulder clay	5-18
Firm clay	5-20
Wet sand	10-15
Silt	5-20
Dry loose sand	25-45
Fill	-45

Table 2.2 Typical values of friction load on pipes (After Craig (1983)).

It will be appreciated that the method is no more than an "experienced guess" which can lead to problems with pipe failures and abandoned boring machines. Injection of a lubricant such as bentonite slurry into the overbreak can result in reduced frictional resistance. Research carried out in Japan indicates that a reduction of 30-50% for clayey soils and about 20% for sandy soils can be achieved (Ishibashi, 1988). However no attempts have been made to standardise the use of lubricants.

In order to provide a comprehensive model for jacking force predictions it is necessary to review the parameters which influence the jacking loads. These must be rationalised in relation to theoretical methods for evaluating ground pressures on buried structures.

The principal factors include:

- Primary load from the ground including surcharge, transient live loads and internal and external fluid pressures
- Resistance at the excavation face
- Amount of overcut during excavation
- Variation in ground conditions along the pipeline.
- Misalignment of pipes
- Injection of a lubricant into the overbreak void
- Use of intermediate jacking stations

Auld (1982), Haslem (1986), O'Reilly and Rogers (1987), Ripley (1989) have proposed various approaches to predicting jacking forces based on the analysis of field data from selected pipejacks.

Auld presents an analysis for a pipe driven through a cohesionless material for which the soil is assumed to collapse onto the pipe and exert radial pressure around its circumference. The method is notably based on work by Terzaghi (1943) in which the analysis of pressure distribution on rectangular buried structures in active failure zones is suitably modified for the circular pipe jack situation. The method takes account of arching in the soil above the pipe redistributing ambient stress away from the pipe. The frictional resistance is then determined by using coefficients of friction in conjunction with the calculated ground pressure. Unfortunately Auld fails to resolve the horizontal and vertical stresses into a correct radial stress component resulting in a 25% overestimate of jacking resistance. Evaluation of suitable friction coefficients is also difficult to define accurately and can be further modified by the use of lubricants. Ripley (1989) states that pipejackers usually adopt a value of 0.7 times the internal angle of friction in cohesionless ground, however the value is likely to be stress level dependent.

The ability of an unlined tunnel to support itself in non-cohesive and cohesive ground has been studied using laboratory and centrifuge models, Atkinson, Brown and Potts (1975) Davies, Gunn, Mair and Seneviratne (1980) and Atkinson and Mair (1981).

In non-cohesive ground it was found that the support pressure σ_T was independent of tunnel depth. Collapse was initiated by a small triangular wedge of material loosening at the crown of the tunnel. For the case with no surcharge loading

$$\sigma_T = \gamma D \; T_\gamma$$

where T_γ is a stability number and is only a function of ϕ'. (Figure 2.3)
D is the diameter of the bore.
γ is the density of the soil.

If water pressure is present it is added to σ_T and $(\gamma - \gamma_w)$ is used for that part of the soil below the water table.

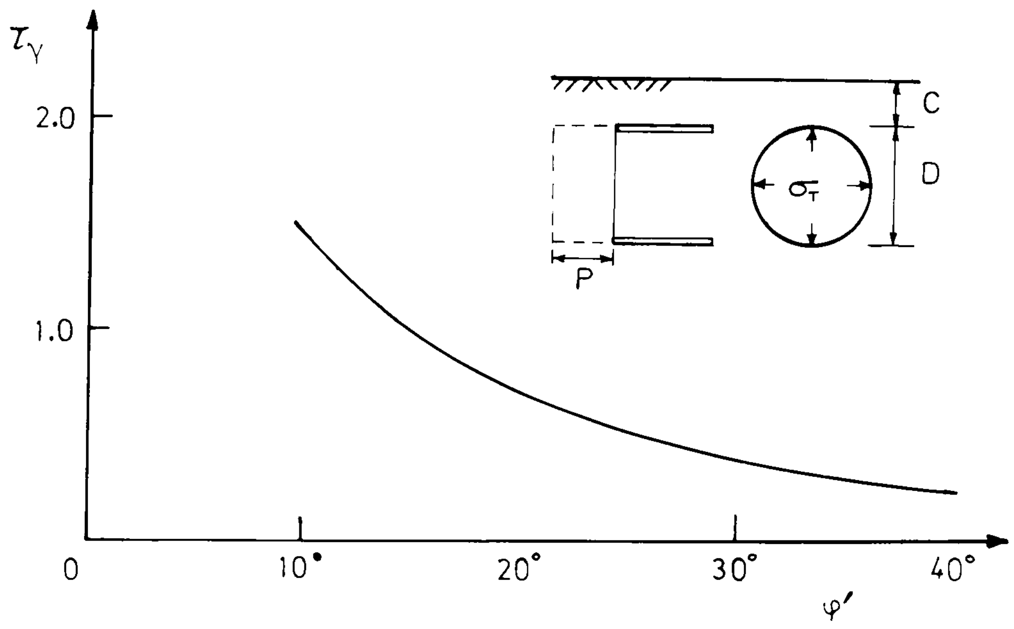


Figure 2.3 Relationship between tunnel stability number T_γ and drained angle of internal friction ϕ'.

In cohesive ground, stability is dependent upon the unsupported length of tunnel advance P. For two dimensional tunnels ((ie) P/D large) and $\sigma_T = 0$, the short term stability of a tunnel can be assessed from the relationship between $\gamma D/C_u$ and C/D in Figure 2.4.

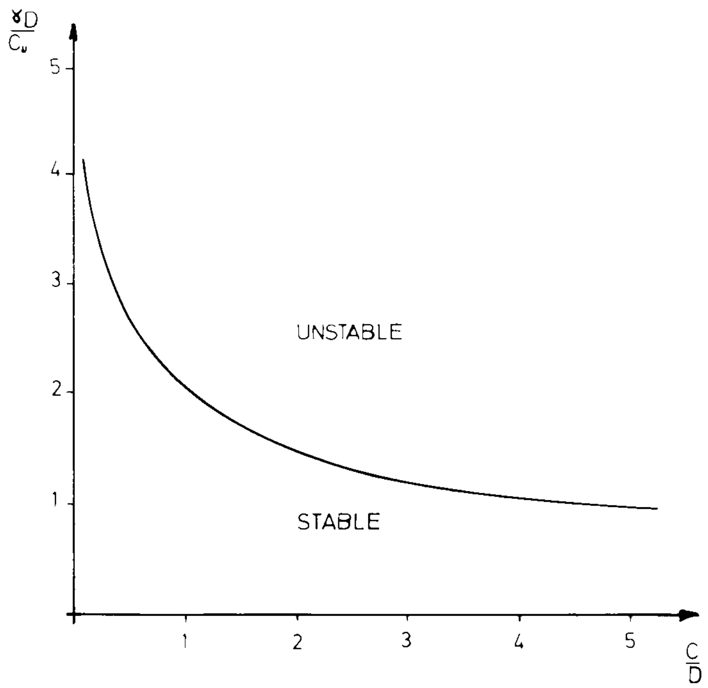


Figure 2.4 Criterion for assessment of the immediate stability of an unsupported circular plane section tunnel in clay.

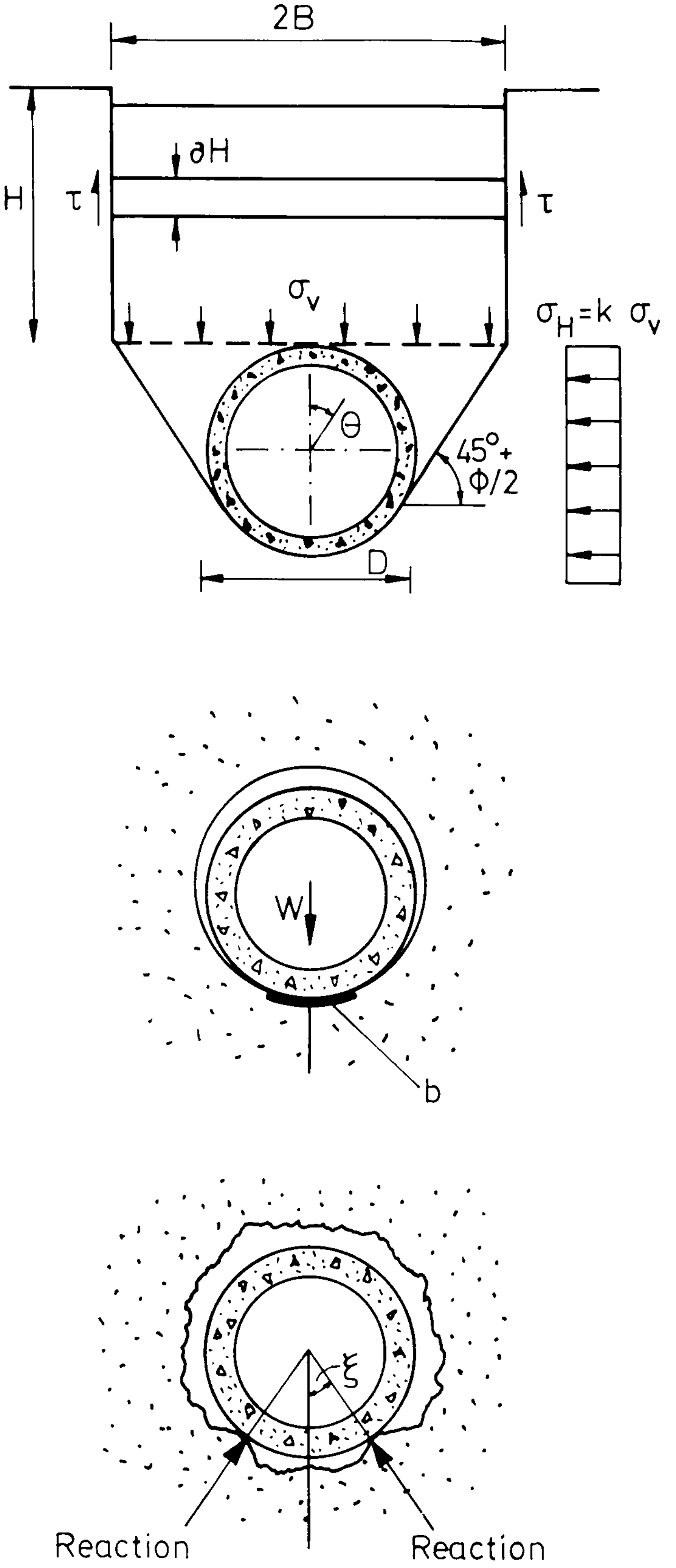
Haslem studied the behaviour of pipe jacks in London Clay, with stable bores. The model he adopted was based on the behaviour of an elastic cylinder resting in a cylindrical void in an elastic continuum, Roark and Young (1976). The shear surface for the cohesive material was assumed to be within the soil, rather than at the pipe soil interface. Comparisons made between the jacking forces and those measured in actual pipe jacks were found to provide an underestimate of the total jacking force. Adjustments were made to allow for misalignment of pipes by considering an element of pipe length subject to a local radius of curvature, but this was found to have negligible effect.

O'Reilly and Rogers (1987) used the same technique to compare predicted and field measurements in clay and rock (sandstone). Laboratory tests carried out by the authors to assess the contact area between pipes and reconstituted clay indicated that time dependent plastic deformations occur under the pipes resulting in larger contact areas. Replotting the data published by Haslem and increasing the predicted values in accordance with the experimental data produced a better fit. The analyses for rock were based on either single or multiple point contact relative to the pipe invert. The model produced a good fit to the available data provided the site conditions were relatively uniform.

A summary of the models is presented in Figure 2.5. All are based on the assumptions of a perfectly straight pipeline and plane strain conditions for the lining and the ground. This is never the case in practice. Some degree of misalignment is inevitable and this will introduce a series of forces normal to the line of the drive increasing the total jacking force. The types of force resulting from misalignment, assuming elastic deformations, are shown schematically in Figure 2.6. Stevens (1989) and Ripley (1989) have conducted laboratory based tests on jacked pipes maintained in deflected positions as the axial load is increased . Examination of the induced loads in the lateral support systems provides an indication of the resistance the ground around a pipe jack must provide if realignment is prevented. The data substantiate the fact that less interaction occurs between pipe and restraint with smaller deflection angles. The work also enabled the resulting strain distribution throughout differently misaligned pipes to be recorded and associated failure modes to be observed.

2.4 Pipe end load transfer

The problems associated with misalignment, which generally manifest themselves as local spalling of concrete at the joints, have led to the introduction of low stiffness packing material between the joints to help distribute the axial force. Ripley (1989) has tested the most commonly used wood based packing materials obtaining information on the effects of angular deviations and packer positioning in the joint. A summary of the results



Non Cohesive Auld (1982)

$$\sigma_v = \frac{\gamma B}{k \tan\phi} \left(1 - e^{-k \tan\phi H/B}\right)$$

$$\sigma_H = 0.3\gamma\ (0.5D + D\sigma_v)$$

$$\sigma_p = \frac{\sigma_v + \sigma_H}{2} + \frac{(\sigma_v - \sigma_H)}{2} \cos 2\theta$$

$$P_{TOTAL} = \frac{\pi D}{2} (\sigma_v + \sigma_H)$$

$$F = \tan \delta .\ P_{TOTAL}$$

---

where ϕ is the angle of internal friction of the soil.
δ is the angle of friction between the pipe and the soil.

Cohesive with a stable bore Haslem (1986)

$$F = \alpha . s_u\ b$$

---

where $\alpha . s_u$ is the "adhesion" between the pipe and clay.
b is the contact width.

Rock O'Reilly & Rogers (1987)

$$F = W\mu / \cos \xi$$

---

where W is the weight of pipe per unit length.
μ is the coefficient of friction.

Figure 2.5 Summary of ground loading models.

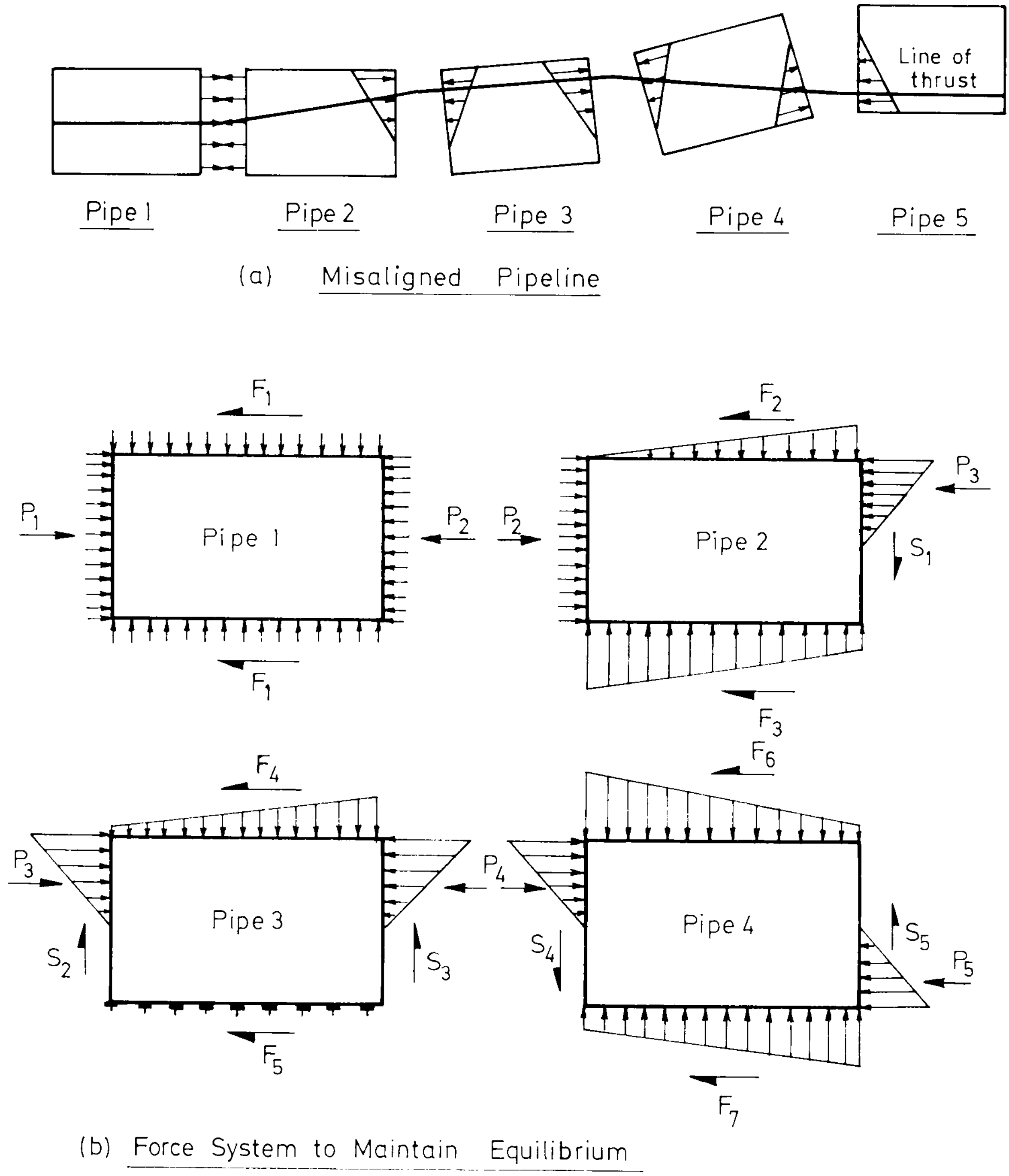


Figure 2.6 Misalignment forces.

suggests that:

- Thick packing materials are most beneficial, with 18mm thick medium density fibreboard the most suitable from the materials tested.
- Deflection angles greater than 0.2° combined with high axial loads are to be avoided.
- Wet materials improve load transfer capabilities by up to three times.
- Cyclic loading conditions need to be considered due to permanent packing material compression.

The significance of the packing material prompted Ripley to propose that pipes should be delivered to site with a standardised packer already in place.

The benefits of packing material in reducing the effects of axial stress concentrations in vitrified clay jacking pipes has been established by Boot and Husein (1991). The conclusions from this work were similar to those of Ripley.

Various attempts have been made to theoretically evaluate the induced stresses in the concrete as a function of packing material stiffness and geometry, Milligan and Ripley (1989), Nagata et al (1990) and the Concrete Pipe Association of Australia (1983). The first two methods assume linear strain and non-linear stress characteristics across the joint while the third simplifies the model by adopting a linear stress approach as illustrated in Figure 2.7. All three analyses fail to take account of the misalignment shear forces which are generated at pipe joints.

The analysis presented by the Australian CPA is summarised below:

Total packer and pipe deformation $\qquad \Sigma\Delta a = \Delta a + \Delta L$

Deformations related to stresses

$$\underset{\text{Joint}}{\frac{\sigma_j a}{E_j}} = \underset{\text{Packer}}{\frac{\sigma_j a}{E_p}} + \underset{\text{Concrete}}{\frac{\sigma L}{E_c}}$$

but $\sigma = \dfrac{\sigma_j t_j}{t}$ where t = wall thickness and t_j = wall thickness at joint

therefore $\dfrac{a}{E_j} = \dfrac{a}{E_p} + \dfrac{t_j L}{t E_c}$

the corresponding joint elasticity coefficient E_j is given by $\quad E_j = \dfrac{atE_pE_c}{atE_c + Lt_jE_p}$

The problem reduces to the stress distribution in an annular cross-section where the tensile stresses are disregarded, Marks (1978).

From the diagram $\beta = \tan^{-1}\dfrac{\Delta a}{z} = \tan^{-1}\dfrac{a \max \sigma_j}{zE_j}$

Rearranging $\quad \beta = \tan^{-1} \dfrac{a}{E_j}\dfrac{\sigma_{jo}}{R} \dfrac{\max\dfrac{\sigma_j}{\sigma_{jo}}}{\dfrac{z}{R}} \qquad \dfrac{z}{R}$ from tables, Marks (1978).

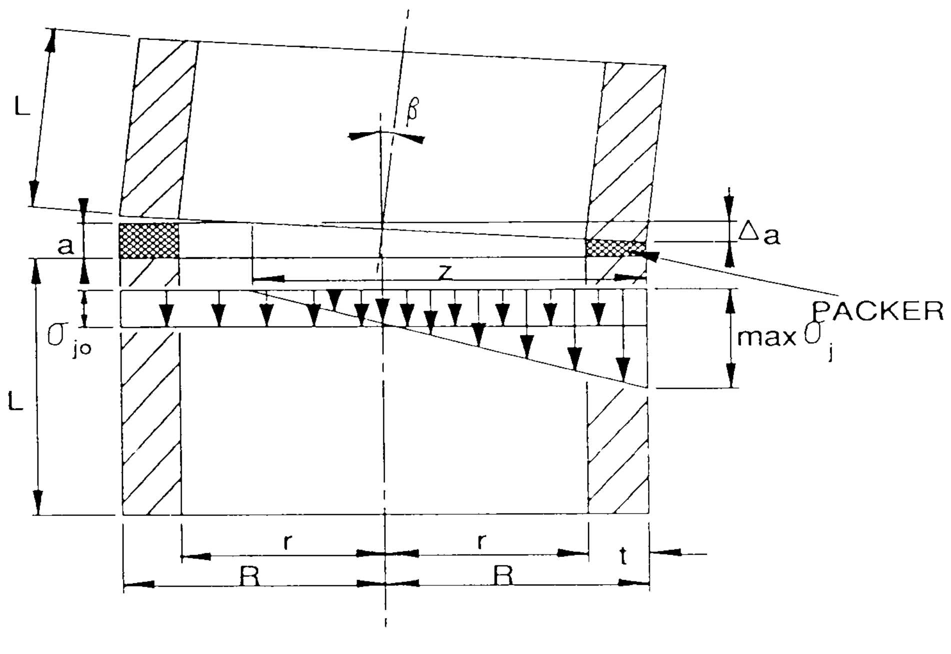


Figure 2.7 Australian joint stress distribution.

Misalignment between consecutive pipes can also result from a lack of pipe end squareness. A test based on the measurement of joint gaps before and after one pipe is rotated through 180 degrees is incorporated into BS5911. An alternative method, proposed for the CEN standard on jacking pipes, involves measuring the generatrices s_1, s_2 and the diagonals r_1 and r_2 as shown in Figure 2.8.

The squareness of ends at the spigot, es, and socket, em, are determined by :

$$es = (s_1^2 + r_1^2 - s_2^2 - r_2^2)/(4ln)$$

$$em = (s_1^2 - r_1^2 - s_2^2 + r_2^2)/(4ln)$$

Where ln is the internal barrel length.

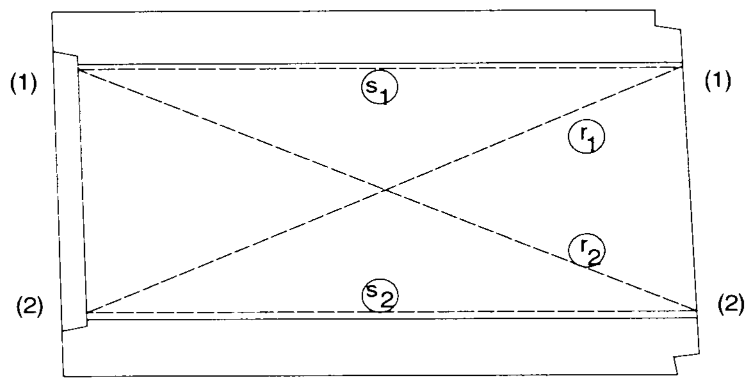


Figure 2.8 Measurements of lengths.

It should be noted that pipes complying with end squareness tolerances of BS5911: Part 120 could have a joint gap of up to 8mm on a 900mm internal diameter pipe. This equates to a deflection of 0.38 degrees even before pipes are deflected relative to each other. It is therefore important to be aware of the end squareness of pipes.

2.5 Geotechnical aspects of pipejacks

Detailed investigations of the subsurface conditions are required for all tunnelling projects. The available data must be carefully evaluated to determine construction methods that will avoid damage to existing surface structures or underground utilities. Factors such as high groundwater tables, unstable soils, and the presence of boulders or contaminated ground, if not recognised prior to construction, can make completion of a pipeline much more difficult. Important physical properties which should be determined include the strength, grain size, plasticity characteristics and permeability of the ground, and their variability along the pipeline. Typical spacing of exploratory boreholes should be about every 50m along the alignment depending upon the degree of complexity of the geological conditions. The purpose of these borings is not only to determine the conditions at each location but also to develop a subsurface profile which highlights boundaries between the various soils. The main difficulty arising from changes in ground conditions is when a fault or soft area is unexpectedly encountered at the face leading to difficulties in maintaining shield alignment. Groundwater also has an important influence on the behaviour of the ground. Groundwater levels should be determined in the borings and hydraulic conductivity evaluated.

An important factor related to soil conditions that significantly influences project feasibility, planning and construction costs is the potential and magnitude of surface and subsurface settlement. Surface settlement is usually considered the primary source of damage to roads, utilities and shallow foundations. In pipe jacking surface settlement is generally associated with loss of ground due to soil squeezing or running into the heading and overcut on the shield. Empirical methods have been developed for estimating surface settlement due to soft ground tunnelling by study of observed settlements on past projects, O'Reilly and New (1982), O'Reilly (1988). A laboratory based study concerned with evaluating the mechanisms controlling soil deformations around pipejacked tunnels is being carried out at Loughborough University, Chapman (1992).

Finally, the resistance that can be mobilised by the thrust wall is a function of the soil strength. It would seem appropriate to allow the thrust wall to develop the full maximum passive resistance because it is a temporary construction loading situation. However the large strains required to develop full passive resistance can lead to significant problems with non-uniform thrust wall movement resulting in eccentric pipe loading and pipe damage, Cole (1977). Movement of the thrust wall can be controlled by limiting the allowable passive resistance to about one half of the maximum value although the effects of repeated load application on the ground need further investigation.

2.6 Instrumentation and monitoring of field work

2.6.1 Review of tunnel, piling and pipe jacking instrumentation projects

The instrumentation and monitoring of tunnel linings and the ground in the vicinity of a tunnel are carried out to provide a better understanding of the mechanisms which affect lining and ground behaviour, during and after tunnel construction. The results of such monitoring help to improve the design methods for tunnel linings and enable indications of the likely surface settlements to be established, and thus the risk of damage to structures to be assessed.

Instrumentation programmes for tunnel projects have been carried out for many years. Cording et al (1975) present a detailed state of the art report for instrumentation of tunnels in soil and rocks. John and Crighton (1989) describe the geotechnical control and monitoring being carried out for the Channel Tunnel project.

Much of this instrumentation is limited to monitoring from within the tunnel using instrumentation stations that remain stationary with respect to the tunnel drive. Of more relevance to pipe jacking is the work that has been carried out on instrumented piles to improve our understanding of pile-structure interaction. Yong (1983) presents an interesting overview of the practical aspects of instrumentation used to monitor twelve bored cast-insitu piles and two displacement piles. The instrumentation concentrated on determining load distribution characteristics using mechanical extensometers, vibrating wire strain gauges, resistance strain gauges and custom made load cells. A comparative study of their performance indicated that the mechanical devices were less prone to failure or damage than the electrical devices. This substantiates earlier work by Londe (1982) who reported from statistical findings that the casualty rate for vibrating wire strain gauges is less than 10%, while Reese and Hudson (1968) indicated that failures in resistance strain gauges can be as high as 50%. The major problems experienced with the strain gauged devices was due to moisture penetration.

Coop (1987), Ponniah (1989) and Bond and Jardine (1989) report on the performance of instrumented model piles used to investigate the fundamental behaviour of driven open ended steel piles used in offshore structures in clay soil. Measurements of axial load at different positions along the pile length, radial total pressure and pore water pressures were reported. The Bond and Jardine pile also included surface contact stress transducers capable of measuring radial total stress and shear stresses at specific locations on the pile surface.

Such heavily instrumented schemes contrast sharply with the limited published data from pipe jacking sites in the UK which tend to be confined to line, level and jacking pressure readings on mechanised drives only. Overseas the situation is slightly better with schemes such as the New York, Staten Island sewer upgrade reported in Civil Engineering (1988) providing a £1.1 million allowance in the contract for instrumentation. The scheme involved pipe jacking 2.15km of moulded fibreglass pipe through sands and clays. Several pipes were fitted with vibrating wire strain gauges and surface mounted extensometers to measure longitudinal and circumferential stresses. Load cells were positioned in the jacking system. Subsurface settlement indicators have also been installed over the length of the project. Discussions with the project engineer, Beloff (1989), confirmed that the instruments had performed satisfactorily and indicated relatively uniform compression of pipes and the presence of locked in stresses after jacking pressure release. No problems with high local stresses at joints were recorded. Unfortunately data from the project has not been published.

2.6.2 Planning and execution of tunnel fieldwork on active construction sites

Instruments should be designed to be easily installed and, if possible, fixed or cast into the lining before it is incorporated into the permanent works thus minimising delays in the tunnelling programme, and, where possible, readings should be taken remotely from the location of the instruments.

The instruments should be relatively cheap to enable sufficient numbers to be installed. All instruments should be reliable and constructed to withstand the environment in which they will be installed. This environment in a pipe jacking tunnel is likely to be wet, dirty and very confined. They should be sufficiently robust to withstand the curiosity of the tunnel labour force and covered to withstand shock. The instruments should be designed for the required accuracy and be simple to calibrate. Instruments installed in the lining should be designed to cause as little modification to the structural performance of the lining as possible, otherwise they will be measuring conditions different to those of the standard lining.

The readings obtained from the instruments should be analysed as the work proceeds, enabling errors in the instruments or the reading techniques to be found during the course of the work rather than at a later date. The output should be designed to suit the results required for the final objectives and not obscured by a large quantity of data, that may never be used. All of these objectives have been incorporated into the design of the instrumentation scheme for the pipe jacking research reported in Chapter 3.

It will be apparent that instrumentation programmes for tunnels are normally carried out during the construction phase of the project. The use of instruments is not merely the selection of suitable devices but a comprehensive step-by-step engineering process beginning with a definition of the objective and ending with interpretation of the data. Each step is critical to the success or failure of the entire programme and none more so than the planning and implementation of fieldwork on active construction sites to minimise the contractual pressures that inevitably arise. This particular aspect is the subject of a forthcoming code of practice which is being drafted by the SERC Geotechnics Steering Group. Draft guidance notes list the following key elements for successful field instrumentation research:

Planning Phase

- Maximise lead in time - be involved with the design
- Have clear and realistic objectives
- Understand roles and motivations of parties involved
- Work with consultants for proper provisions in contract documents
- Explain aims of research to the Contractor and encourage his interest
- Use expertise of experienced groups such as TRL and BRE

Fieldwork Phase

- Try to install instruments when the site is not busy
- Make research staff part of the Resident Engineer's team
- Arrange for a member of the contractor's staff to be personally responsible
- Communicate clearly - beware of unbriefed subcontractors
- Be prepared for delays and disruptions
- Process data promptly and feed back to site staff

CHAPTER 3:

INSTRUMENTATION

3.1 Introduction

This chapter presents details of the instrumentation developed for the research programme. Specific instrument designs are described and calibration procedures and results discussed. The development of the instrumentation has been heavily influenced by Hannah (1985) and Dunnicliffe (1988). A full description of the design, manufacture and calibration phases including design drawings, supplier details and individual calibration data is presented in Norris (1992).

3.2 The instrumentation and measurement programme.

The field instrumentation consists of three instrument clusters or monitoring stations as shown in Figure 3.1. Two stations are incorporated into specially prepared "standard" concrete pipes which can be inserted at any position in a train of pipes, while the third is positioned at the jacking pit. The lead instrumented pipe is only used in drives through cohesive material and is fitted with a Ground Convergence Indicator which measures the rate of ground closure above the front end of the pipe string particularly when jacking is halted overnight or at weekends. The main instrumented pipe is located further back in the pipe string and contains the following instruments:-

(a) Four Contact Stress Cells, to measure both radial total stress and shear stress acting on the pipe surface, with their active face flush with the surface of the pipe and provided with

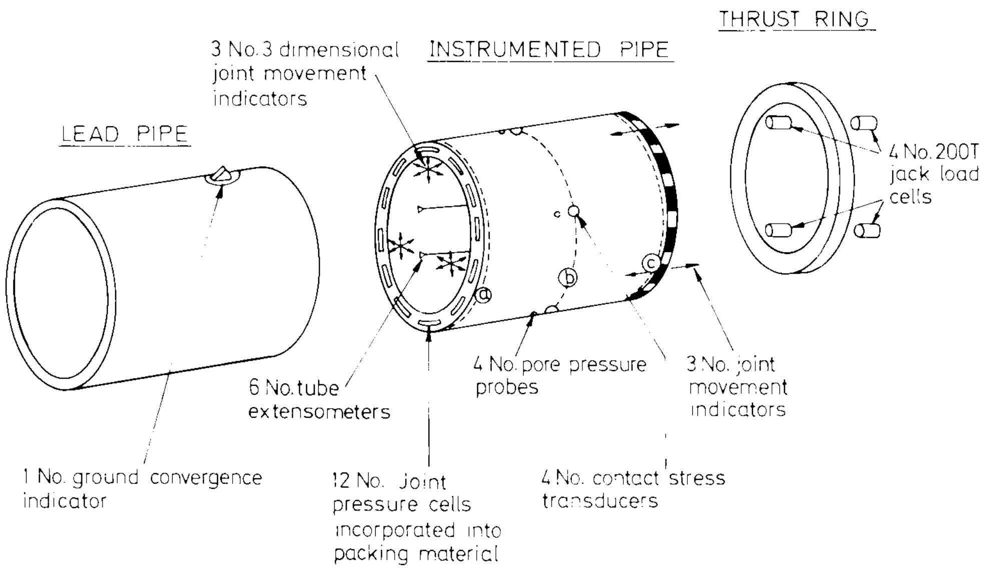


Figure 3.1 Schematic of instrument arrangement.

a similar surface finish;

(b) Four pore pressure probes adjacent to the contact stress cells, measuring the local pore pressure and hence allowing determination of the effective radial stress;

(c) Three joint movement indicators at each end of the pipe, to measure the movements across the joint gaps in three dimensions;

(d) Twelve pressure cells built into the packer in the joint at each end of the pipe to measure the magnitude and distribution of the loads transferred across the joints;

(e) Six tube extensometers fitted to the inner surface of the pipe and equi-spaced around it to measure the compression of the pipe under load; and

(f) A modular data acquisition system and purpose built stable power supply.

In the jacking pit the total jacking load is recorded by two or four load cells positioned between the jack rams and the thrust ring, and the forward movement of the pipe string by a displacement transducer mounted above the tunnel entrance. Readings from the instruments are backed up by a detailed log of all site activities, a daily survey of line and level of the full length of the tunnel, and additional observation, sampling and testing of the ground conditions to supplement site investigation data. Pipe dimensional checks, concrete strength and stiffness testing, and where appropriate, surface settlement surveys have also been carried out. A flow chart summarising the objectives of the instrumentation is shown in Figure 3.2.

SOIL-STRUCTURE INTERACTION		INSTRUMENT		PIPE PERFORMANCE
Localised total radial and shear stresses acting on the pipe surface	←	CONTACT STRESS CELLS		
Average shear stress over a single pipe length	←	JOINT PRESSURE CELLS	→	Local pipe joint stresses enabling load deflection characteristics of pipe joints to be evaluated insitu.
Average face and shear resistance along pipeline	←	JACK LOAD CELLS AND PRESSURE CELLS		
		TUBE EXTENSOMETERS	→	Overall pipe compression
Changes in pore water pressure adjacent to the pipe to enable ground loading to be expressed in effective stress terms.	←	PORE PRESSURE PROBES		
Ground convergence rates around pipes	←	GROUND CONVERGENCE INDICATOR		
		PIPE JOINT MOVEMENT INDICATORS	→	3-D monitoring of pipe joints to measure joint angles and provide relationships between alignment and pipe end stress distributions
SITE LOG OF ACTIVITIES AND LABORATORY/INSITU SOIL TESTING				

Figure 3.2 Flowchart indicating objectives of each instrument type.

An inventory of the equipment and costs at 1991 prices are presented in Table 3.1.

Item Description	Number	Replacement cost at 1991 prices (£) (excluding VAT)	Total Cost at 1991 prices (£) (excluding VAT)
1. Contact Stress Cells a) Housing b) Cambridge transducer	 5 8	 1500 900	 7500 7200
2. Pore Pressure Probe (including Druck PDCR81)	5	750	3750
3. Pipe Joint Pressure Cells	34	550	18,700
4. Tube Extensometers (including LVDTs)	6	850	5100
5. Pipe Joint Movement Indicators (including LVDTs) a) single dimension model b) three dimensional model	 3 3	 400 850	 1200 2550
6. Jack Load Cells (including caps)	4	1700	6800
7. Celesco Units a) 0-100" b) 0-150"	1 1	300 400	300 400
8. Ground Convergence Indicator	1	1500	1500
9. Data logger equipment including purpose built stable power supply steel enclosures and female connectors.	9	1200 (av)	10,800
10. P.Cs a) 386 OPUS model b) 286 NESS model	1 1	1400 850	1400 850
11. Cabling a) flexible power b) rigid power c) signal	300 m 50 m 400 m	200 25 80	200 25 80
12. Male connectors/power supply plugs & sockets	ITEM	1500	1500
13. Instrument container	1	3200	3200
		TOTAL	£73,055

Table 3.1 Equipment inventory

3.3 Instrument design

The main instrument characteristics required for the pipejacking research include:

- Easy installation into the pipes prior to incorporation into the permanent works, and subsequent removal, thus minimising delays to the tunnelling programme.
- Reliability and robustness in the tunnel environment.
- Designed for required accuracy and simple to calibrate.
- Minimal modification to the structural performance of the pipe and the soil-pipe interaction.
- Readings taken remotely from the location of the instruments.

All of these characteristics have featured in the evolution of the individual instrument designs.

3.3.1 Contact stress cell

The contact stress cell is the most complicated of the research instruments. It measures both the radial total stress and the shear stress acting at specific locations on the pipe surface. The assembly is shown in Plates 3.1 and 3.2 and Figure 3.3. At the heart of the instrument is a Cambridge Earth Pressure Cell which is based on the original work by Arthur and Roscoe (1961) and Stroud (1971). The cell is machined on a CNC milling machine from a single block of aluminium alloy 2014A and is wired up with three independent strain gauge circuits, two to sense the radial stress and the other to sense the shear stress. Assembly of the instrument involves bolting the pressure cell to the main stainless steel housing and covering it with a two part cap which protects it from direct contact with the soil. Any load that bears onto the loading platen is transferred to the main housing via the earth pressure cell, apart from a small portion of the load that is lost through the hot bonded rubber seal. Ground water is prevented from entering the instrument by four seals. The loading platen and the frame are joined by a hot bonding

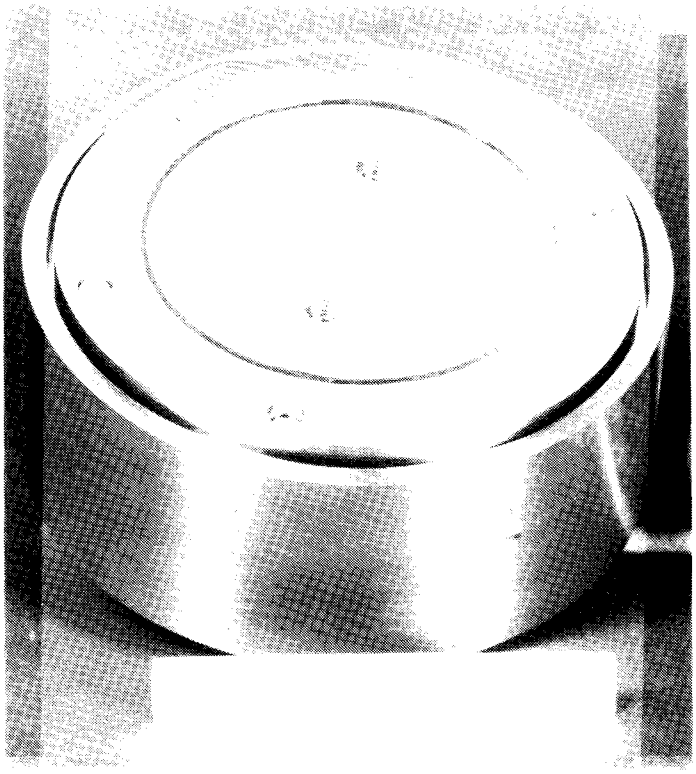


Plate 3.1 Contact stress cell

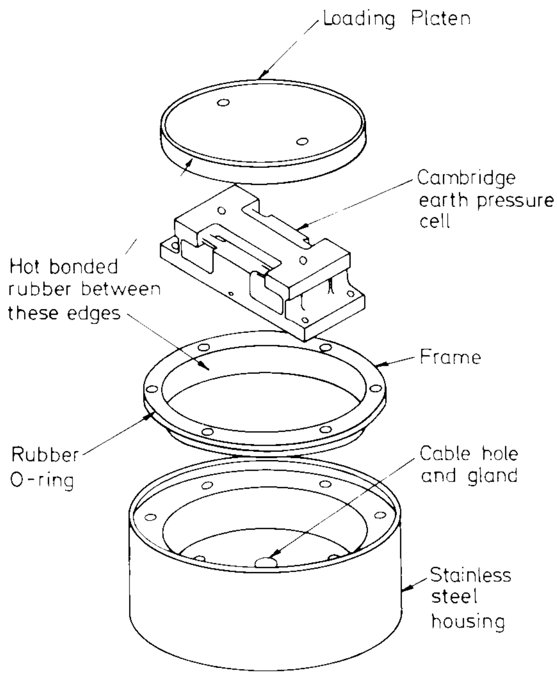


Figure 3.3 Schematic of cell

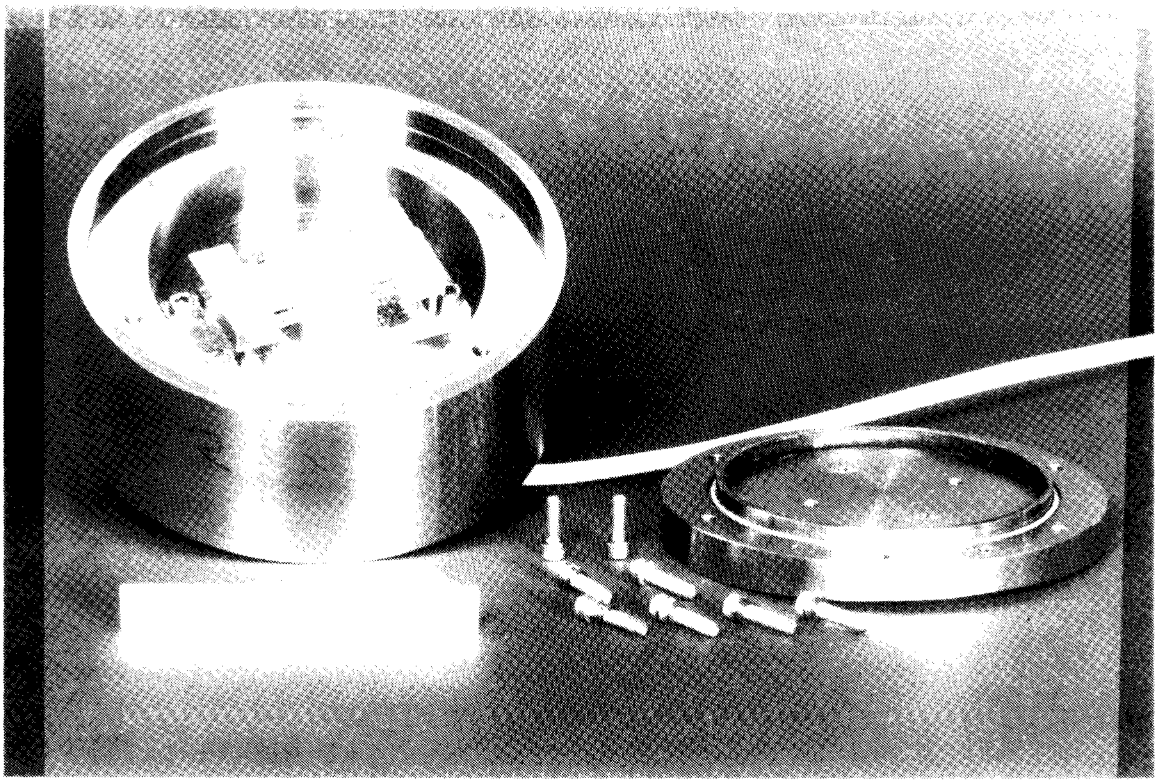


Plate 3.2 Exploded view of contact stress cell

process which injects pressurised rubber between the adjoining edges which are held in their correct relative positions by an aluminium moulding jig. The two part cap is sealed against the main housing by a rubber "O" ring. The bolts which connect the loading platen to the Cambridge cell are sealed with malleable copper washers and finally the cable outlet at the base of the housing is detailed to accept a watertight cable gland.

As full details of the design calculation are not included, the main design requirements are noted:

- Load capacities : Radial ≥ 500 kPa : Shear ≥ 200 kPa
- Linear relationship between signal and load
- Minimal compliance, especially to radial stress
- Minimal cross-sensitivity
- Adequate capacity against buckling of the cell webs under working loads
- Fast response to changes in load
- Loading platen size based on the recommendations of Kallstenius and Bergau (1956) to minimise the effects of soil arching over the instrument.

The assembled contact stress cells are calibrated in a calibration rig which applies simultaneously shearing and compression forces to the active face, Plate 3.3. Load application was in accordance with the procedure summarised in Table 3.2. A typical set of responses for an instrument under radial stress and shear stress calibrations is presented in Figure 3.4.

Profiling and surface texturing of the active face of the contact stress cell and adequate fixity of the instrument in the pipe wall are of paramount importance if the instrument is to record representative field data. A series of interface shear box tests using Leighton Buzzard 14/25 sand and various construction materials indicated that a ground

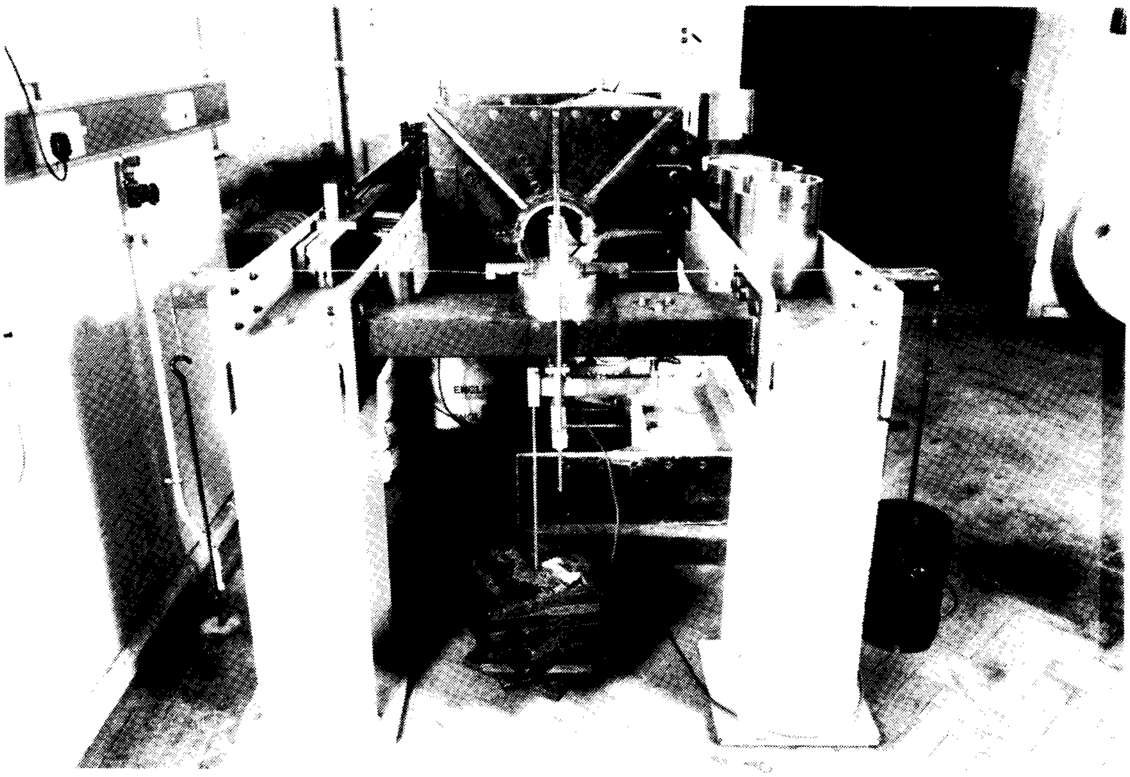


Plate 3.3 Calibration rig for contact stress cells.

Step	Stage	Calibration	Radial kPa	Shear kPa
1	Assembly	-	-	-
2	Exercise	Sustained loads	450 450	+202 -202
3	Exercise	Rapid cycling Rapid cycling	450 450	0→+202 0→-202
4 5 6	Shear Calibration	Vary shear stress whilst keeping radial stress constant	450 348 243	±202 ±202 ±202
7 8	Radial stress calibration	Vary radial stress whilst keeping shear stress constant	0-450 0-450	0 25
9 10	Eccentric loading	Apply radial load at +10mm Apply radial load at -10mm	0-450 0-450	0 0
11	Creep testing	Sustained loads	450	+202

Table 3.2. Calibration procedure for contact stress cells.

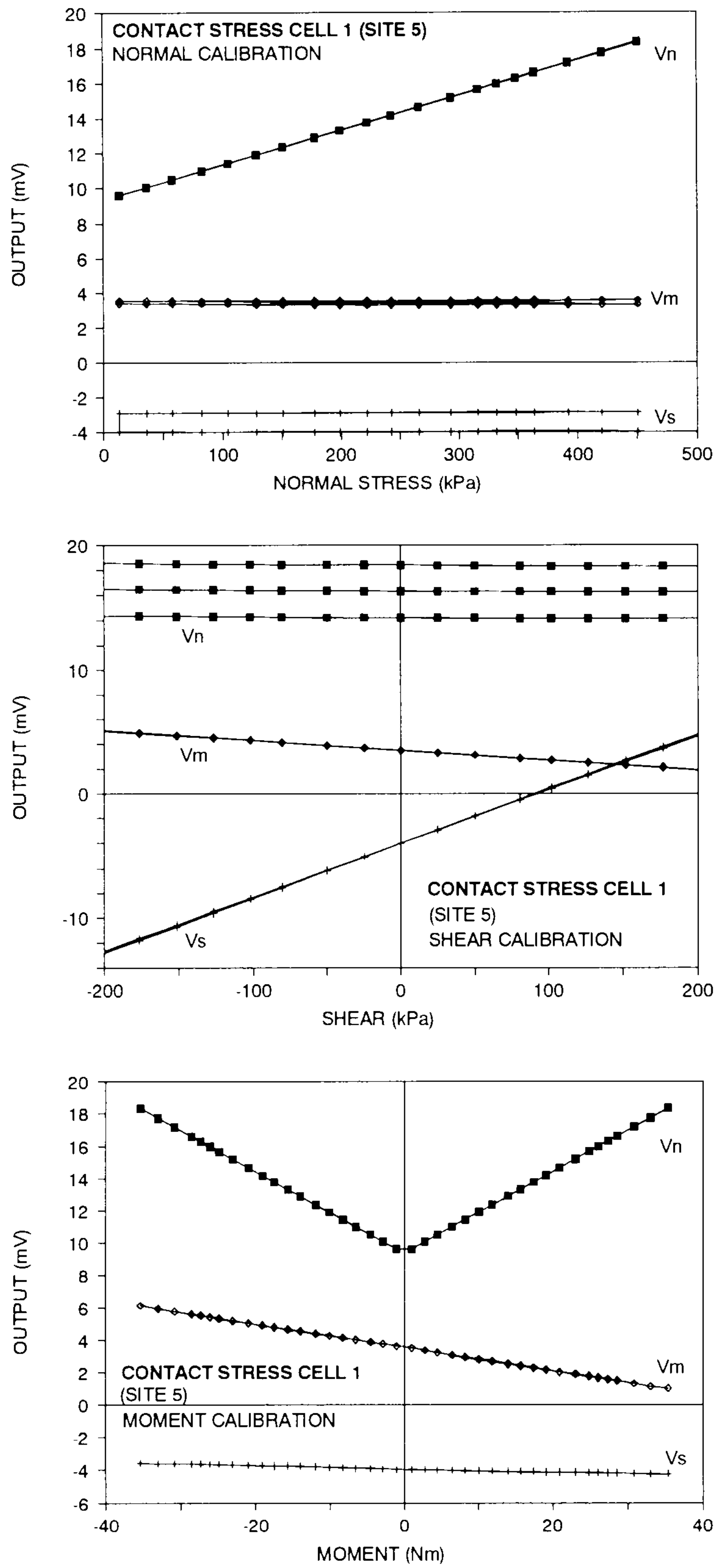


Figure 3.4 Typical calibration responses from a contact stress cell.

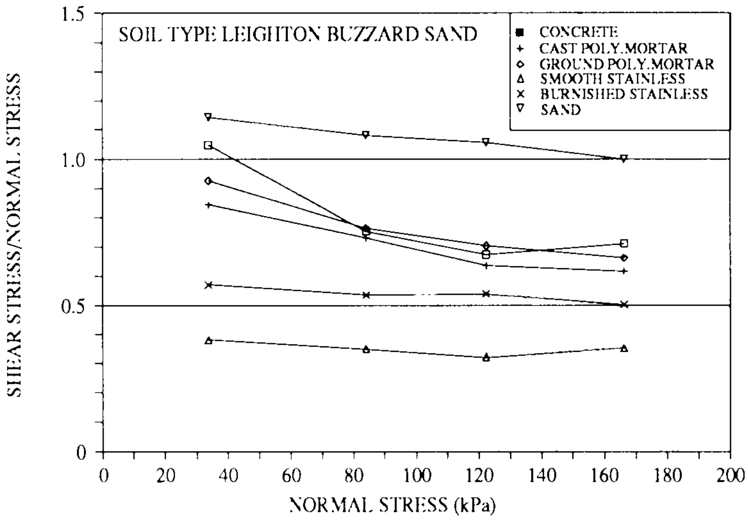


Figure 3.5 Shear box interface tests.

polymer modified mortar finish was the most suitable, Figure 3.5, satisfying both frictional similitude and ease of incorporation into the instrument by gluing a 5mm thick mortar disc into a recess in the loading platen prior to profiling on a CNC machine. Direct adhesion of the cells into specially cast high tolerance holes using a fast curing structural glue was performed on site.

3.3.2 Pore pressure probe

The pore pressure probes are located as close as possible to the contact stress cells. A fast acting probe originated by Bond & Jardine (1989) is used to establish whether rapid pore pressure build up and dissipation occurs as the pipes are pushed through the ground. The probe is illustrated in an exploded view in Figure 3.6 and Plate 3.4. At the heart of the pore pressure probe is a Druck PDCR81 pressure transducer. It is a very sensitive instrument, giving a full-scale output of ± 15mV/V. The transducer is supplied with its ceramic filter removed and is glued into the titanium holder using Araldite two part epoxy resin, care being taken to ensure that it does not protrude from its recess in the holder.

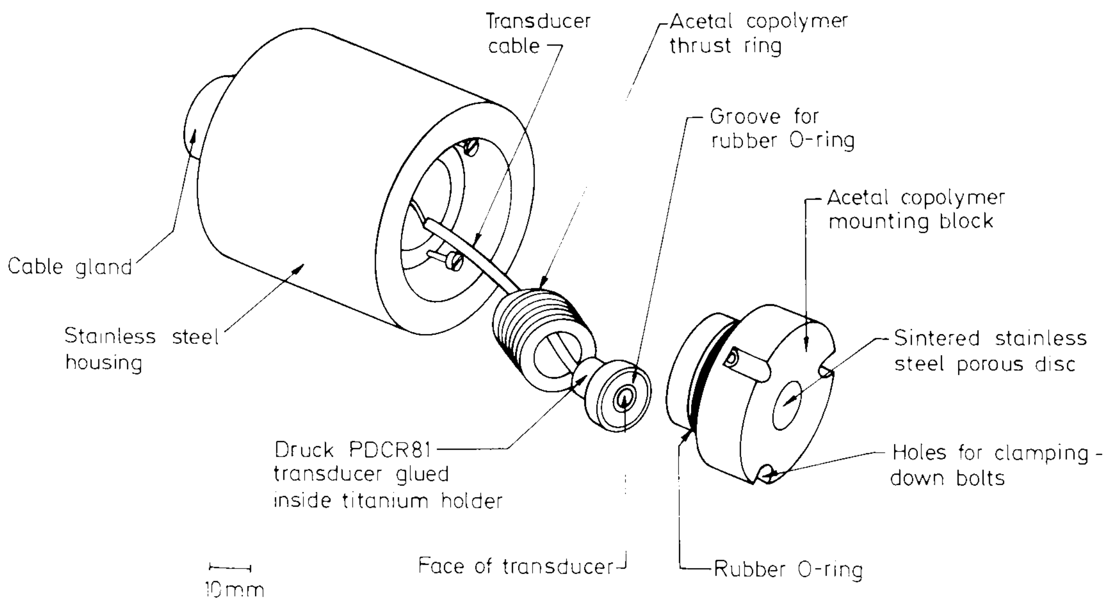


Figure 3.6 Schematic of pore pressure probe.

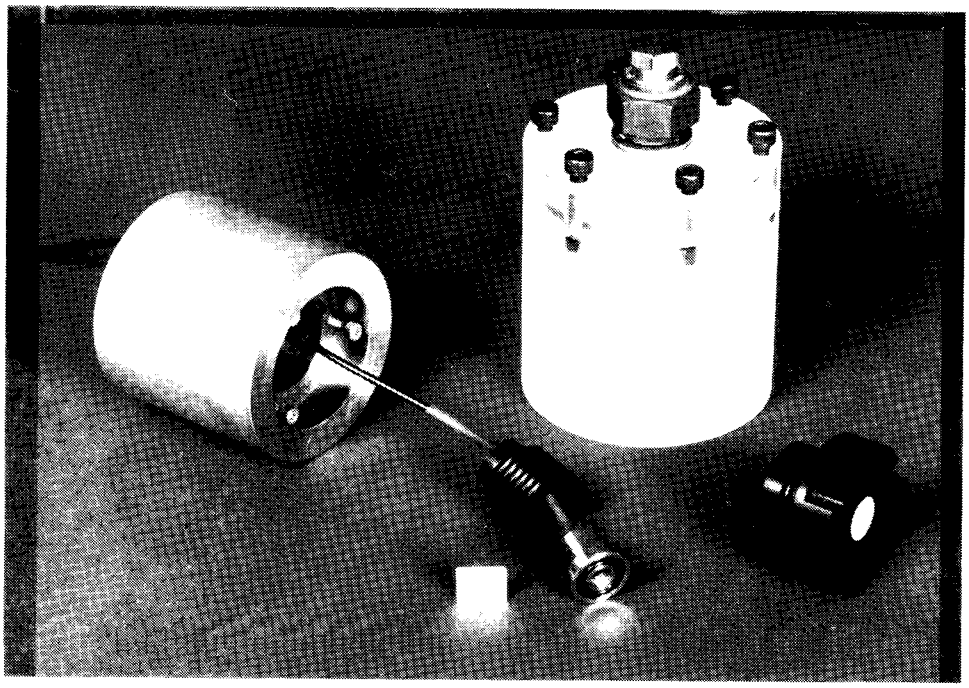


Plate 3.4 Pore pressure probe assembly.

The mounting block and thrust ring are made from acetal copolymer, a stable engineering plastic in order to avoid two metal corrosion between dissimilar metals. The mounting block carries a sintered stainless steel porous filter, in its front face, which is fixed in place by heat shrink fitting and hence is changeable.

On assembling the probe, the holder is clamped into position by the thrust ring which screws into the back of the mounting block. An O-ring seals the holder against the mounting block. Because the pressure sensing diaphragm of the transducer is set back from the front edge of its casing there is a small cavity ($\approx$ 180mm^3) between the diaphragm and the back of the probe's porous filter.

Bond and Jardine list the principal design features as:

- Response time $t_{95} < 1$ second (in the laboratory)
- Immediate recovery from cavitation
- Ease of assembly and saturation in the laboratory
- No loss of saturation during transfer from the laboratory to the ground
- Robust design for site use
- Porous filter is changeable.

Saturation of the filter pores with glycerine is carried out using a vacuum chamber to evacuate the air beforehand. Transducer calibration and transportation to site are carried out in a perspex cylindrical container, Plate 3.4, which can be connected to a Druck digital pressure indicator via a glycerine/nitrogen interface. Transfer of the saturated probe to a stainless steel housing which is glued into a cast hole through the pipe wall is left until the instrumented pipe is ready for jacking, thus minimising possible problems with desaturation.

3.3.3 Pipe joint pressure cell

The pressure cell used in the pipejack joint is a commercially available instrument which comprises a rectangular flat jack formed from two sheets of stainless steel, welded around the edges; the space between the plates being filled with Tellus T46 oil. A closed system is achieved by connecting the cell to a strain gauged diaphragm pressure transducer, Plate 3.5.

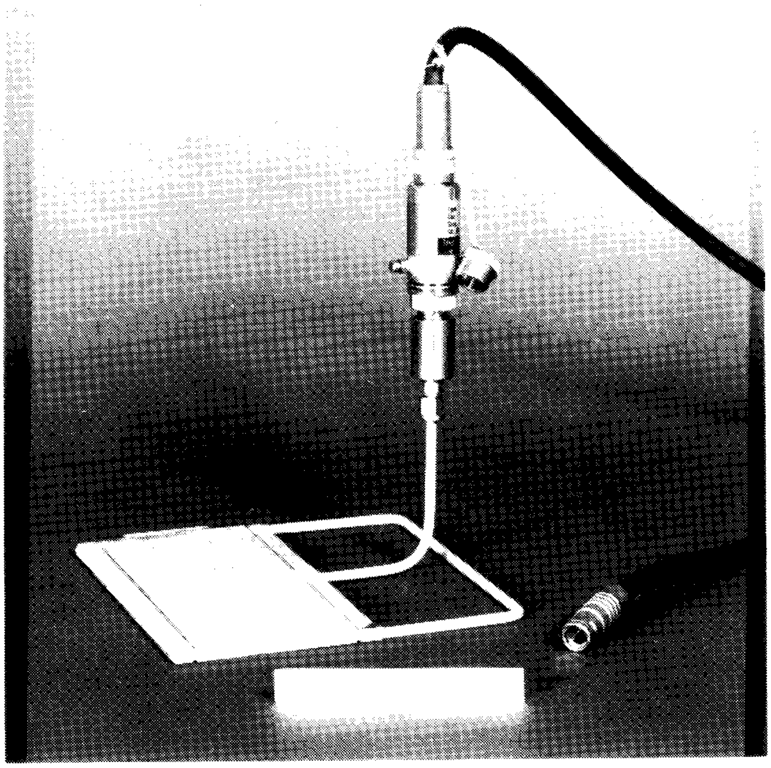


Plate 3.5 Pipe joint pressure cell.

Unusual features for the purpose of the research include a 5mm diameter handle to assist in extracting the cell from the pipe joint, once the monitoring period is finished, and a limited number of cells fitted with platinum resistance temperature probes to monitor insitu temperature changes. Calibration of the instrument was performed in a 50 ton Denison loading machine. The tests were conducted using concrete blocks as loading media with the cell and packer composite sandwiched between them.

The response of the instrument under the cyclic application of a uniform stress was found to be linear and repeatable. However, a principal assumption in the performance of the cell is that its high ratio of area to thickness approaches the ideal of an infinitely thin element minimising the influence of stress distortion due to variations in modulus between the sensor pad and the surrounding material. This assumption was found to be invalid for the pipe jack joint Figure 3.7.

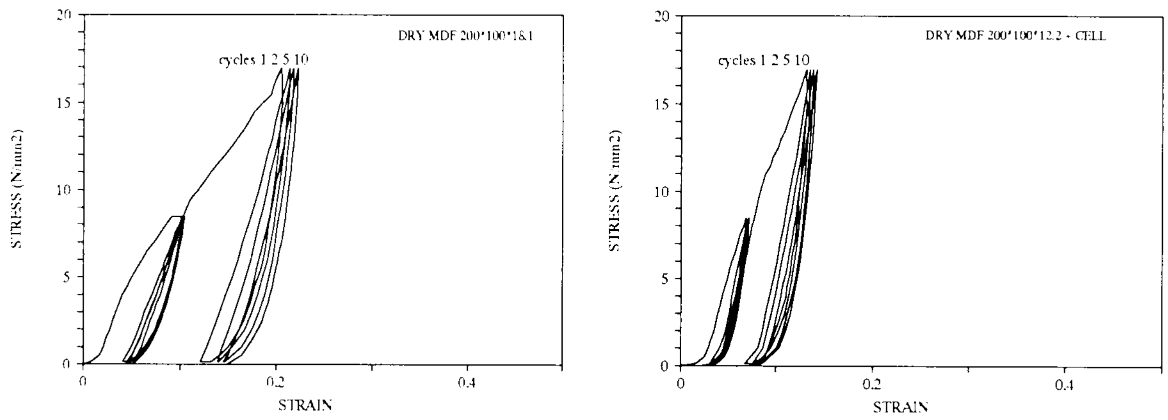


Figure 3.7 Stress-strain responses of packer and composite.

The stiffer response of the composite in the instrumented joint leads to redistribution of normal stresses over the cell and an over prediction of the free field value. A subsidiary set of calibration tests investigating this effect has been conducted by Barton (1992). It should be noted that the effect can be minimised by inserting dummy cells to provide a complete ring of cells and this has been done on schemes 4 and 5.

3.3.4 Tube extensometer

The tube extensometer design is based on a tunnel convergence measurement device developed by Chekan and Babich (1982) and enables concrete movements over a 1.6m gauge length to be recorded using an LVDT mounted at the centre of the device.

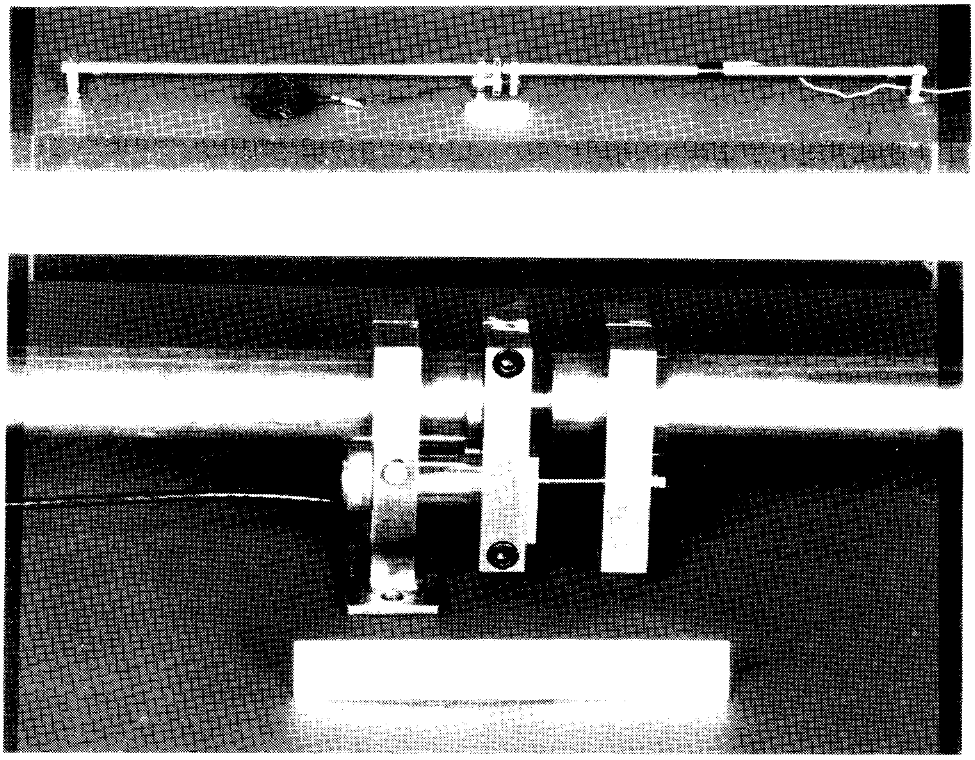


Plate 3.6 Tube extensometer.

The instrument as illustrated in Plate 3.6, consists of two 25mm diameter lengths of silver steel machined to enable one length to slide inside the other for a distance of 100mm. Friction is minimised by gluing two linear motion bearings into the sleeve. Support brackets attached to each end of the device facilitate bolting to tufnel blocks cast into the pipe surface and a central sliding support minimises sagging over the central portion. The LVDT s used in the tube extensometers were calibrated against a micrometer screw bench prior to assembly in the tube extensometers. The tube extensometers were in turn calibrated against an optical grating connected to a digital display unit on a milling machine bed. A typical response is illustrated in Figure 3.8.

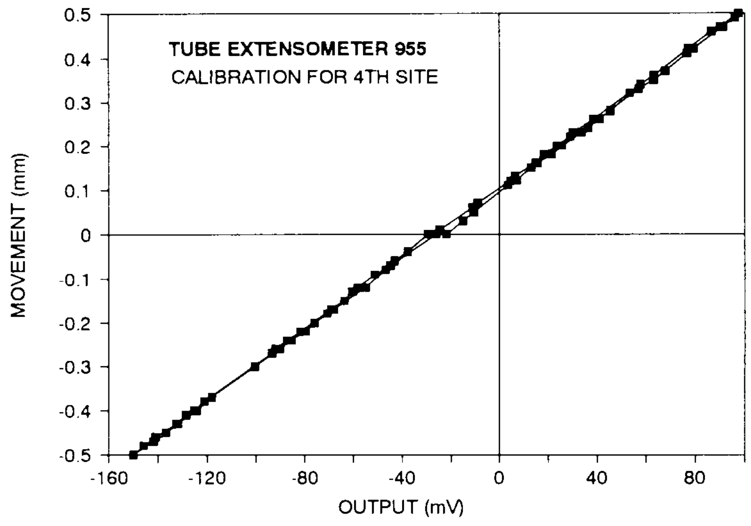


Figure 3.8 Typical tube extensometer calibration response.

3.3.5 Pipe joint movement indicator

The pipe joint movement indicator provides detailed information on the joint gap compression, angular deviation and shear movement as jacking proceeds. The instrument, Plate 3.7, consists of two aluminium blocks geometrically shaped to hold three LVDT s in orthogonal directions. A removable cover plate holds the two parts in their correct relative positions prior to surface mounting on either side of the instrumented pipe joint. The installation procedure involves bolting one block to the instrument pipe while it is above ground and then completing the assembly in the tunnel by gluing the second block to the concrete surface using the cover plate for support. The LVDT s are calibrated against a micrometer screw bench.

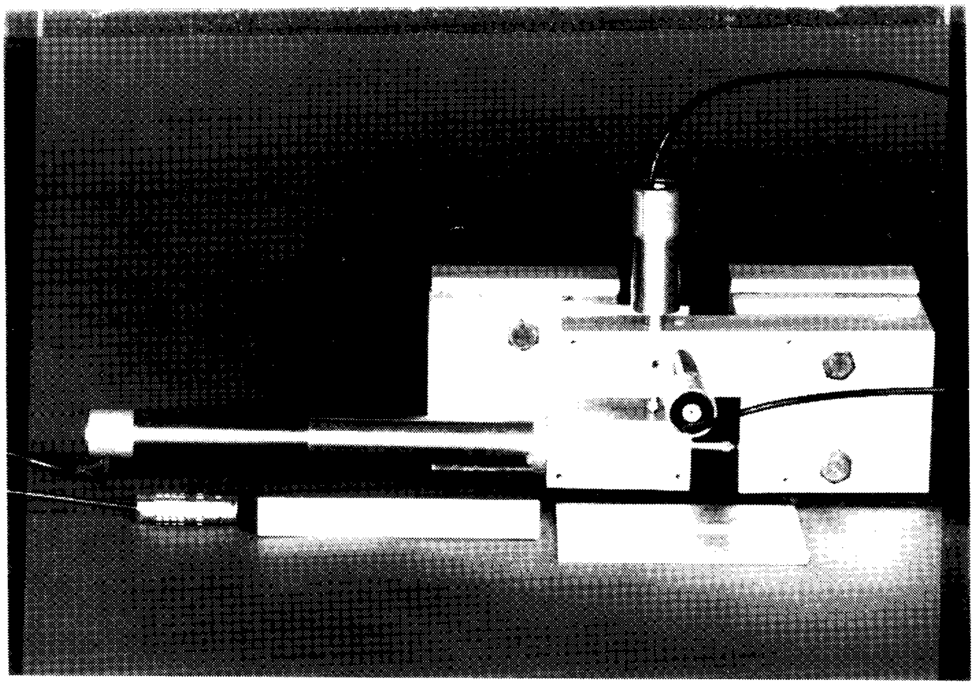


Plate 3.7 Three directional joint movement indicator.

3.3.6 Jack ram load cell

The jack ram load cell is a commercially available 200 tonne heavy duty compression type. The standard cable and its connection to the cell have been modified to ensure operation under submersed conditions. A typical cell is illustrated in Plate 3.8. The cell is fitted with a carrying handle which eases installation and provides physical protection to the cable connection. A domed loading cap greatly reduces the effects of offset loads and damage to the thrust ring. The cell is attached to the main jack rams using 4 M10 threads in its base and a series of coupling plates. A protective hood prevents accidental damage to the load cell. The jack load cells are calibrated against a 50 ton Dension machine, concentric compression being achieved using a ball bearing and spreader cap arrangement.

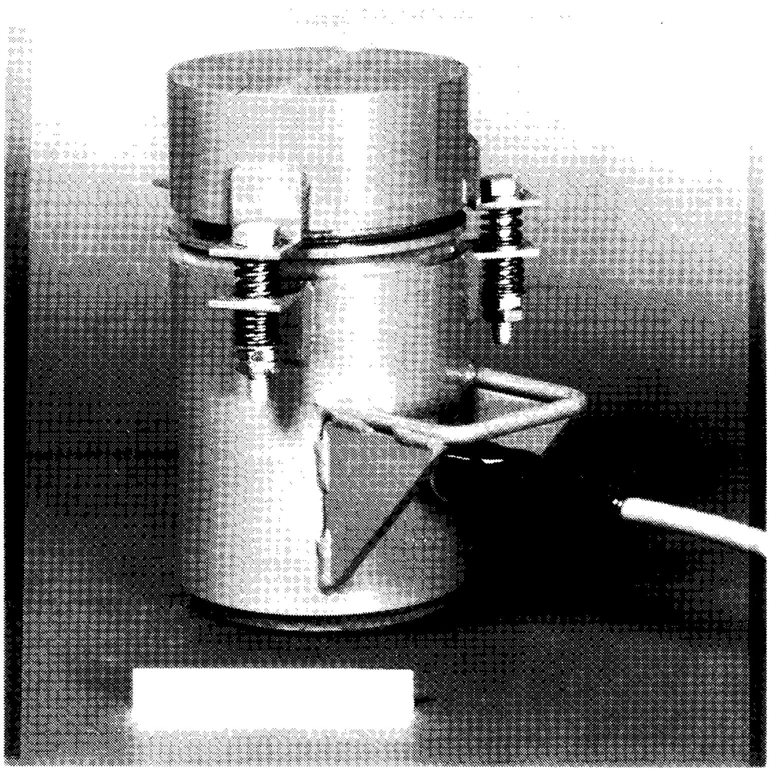


Plate 3.8 Jack ram load cell.

3.3.7 Celesco unit

The Celesco position displacement unit provides an electrical signal proportional to the linear extension of a stainless steel cable through the use of a precision rotary potentiometer. It is a commercially available instrument and is illustrated in Plate 3.9. Laboratory calibration is carried out using a series of one metre rules laid end to end.

3.3.8 Ground convergence indicator

Monitoring the progressive closure of soil onto a jacked pipeline is useful in understanding the mechanisms of load transfer between pipe and soil. Initially it was intended to use ports in the pipes normally provided for grout and bentonite injection to measure the rate of closure but problems with continuity of access to the pipeline meant

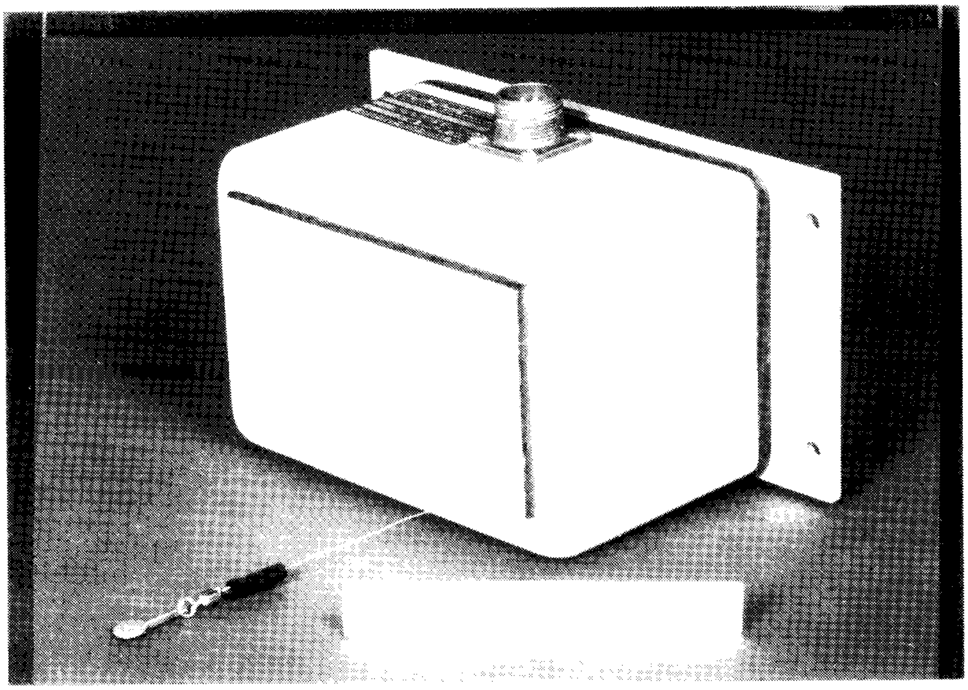


Plate 3.9 Celesco displacement transducer.

Plate 3.10 Ground convergence indicator.

that an automated method of measurement was necessary to supplement periodic manual readings. The ground convergence indicator has been designed for this purpose and consists of a circular main housing which fits into a steel liner bolted to the inner surface of the pipe. The principle of operation involves monitoring the movement of a fin attached to a shaft using a rotary potentiometer. The fin provides a maximum movement of 30mm relative to the top of the housing which is slightly larger than the anticipated overcut on most pipe jacks. Care has been taken to seal the compartment housing the potentiometer from moisture ingress while allowing the void in the recess chamber accommodating the fin to stabilise with the surrounding ground water. Jamming of the fin is prevented by PTFE wipers. The device is illustrated in Plate 3.10. The device was calibrated against a 300mm steel rule held perpendicular to the top surface of the main housing and parallel to the spring loaded fin.

3.3.9 Data acquisition & power supply

Dunnicliffe (1988), Smith (1988) and Kimber (1989) provide a good basis for evaluating the advantages and limitations of automatic data acquisition systems. Many of the advantages suggested by the authors such as continuous monitoring of a large number of instruments, increased reading sensitivity and accuracy and the provision of data storage in a suitable form for direct computer analysis are applicable to the pipe jacking environment with its limited access to the works. Of the limitations, the principal factor, absence of a knowledgeable observer, is circumvented by the author keeping a detailed log of all factors affecting the progress of construction. The ideal data logging system for the pipejack site should be modular in form minimising encroachment upon working space, rugged enough to operate under harsh site conditions, capable of reading a variety of different transducers, capable of operating off site generated electrical power and facilitating direct transmission of data to a PC located above ground.

3.3.9.1 Data acquisition

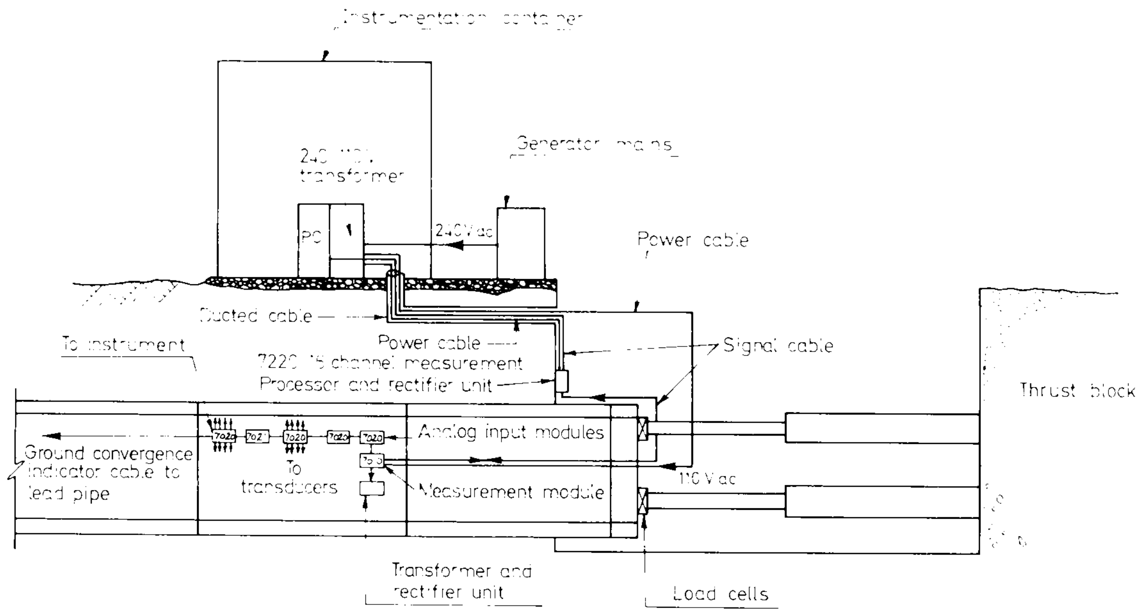


Figure 3.9 Schematic of data acquisition system.

The data acquisition system is shown diagrammatically in Figure 3.9. A modular data acquisition system is mounted inside the main instrumented pipe to enable readings to be taken remotely from the location of the instruments. For simplicity a serial information communication technique has been employed which allows a 286AT personal computer to be used as the basis of control. The system enables a family of commercially available standard "Datascan" control and analogue input modules capable of accepting information from different types of transducer to be located close to the measurement station. Short lengths of analogue signal cables and communication in digital form between the control modules (7010/7220) and the host computer minimise the risk of signal corruption. The system is capable of 16 bit measurement performance and is readily expanded to 80 channels which is the maximum required during the research. Each control module contains a non-volatile memory which retains the set-up information, even after a system power down, which is particularly desirable because the power and signal cables which enter the pipe jack are disconnected each time a pipe is introduced into the drive. The system is powered by a 24 Volt dc supply. Auto range is standard allowing ranges of 20mV, 150mV, 1.3V and 10V.

The 7020 provides the facility to connect 16 dc voltage signal inputs. The 7021 provides eight channels of inputs for strain gauges and platinum resistance thermometers with on board energisation of 1.8V. Measurement accuracy using 16 bit resolution for the 20mV range, the most commonly used range in the research, is:

$$\pm\ 0.02\%\ \text{rdg} + 0.01\%\ \text{range} + 5\mu\text{V}.$$

which is equivalent to 0.006% of the full range value.

Measurement speed is dependent upon the number, type and measurement resolution of channels connected to each measurement module, the network distribution of the measurement modules and the communications link with the PC. For the arrangement shown in Figure 3.9 a single measurement on every channel can be made in 2 seconds. This is acceptable because the instrument logging strategy during the pipejacking research requires measurements every 5 seconds during pushes and 1 minute at all other times. Configuration of the system is carried out using a commercially available software package called Unigen.

It will be apparent from Plate 3.11 that the modules have not been designed or packaged for rough field conditions and that it has been necessary to house them in individual steel boxes to protect them against the tunnel environment, mechanical damage

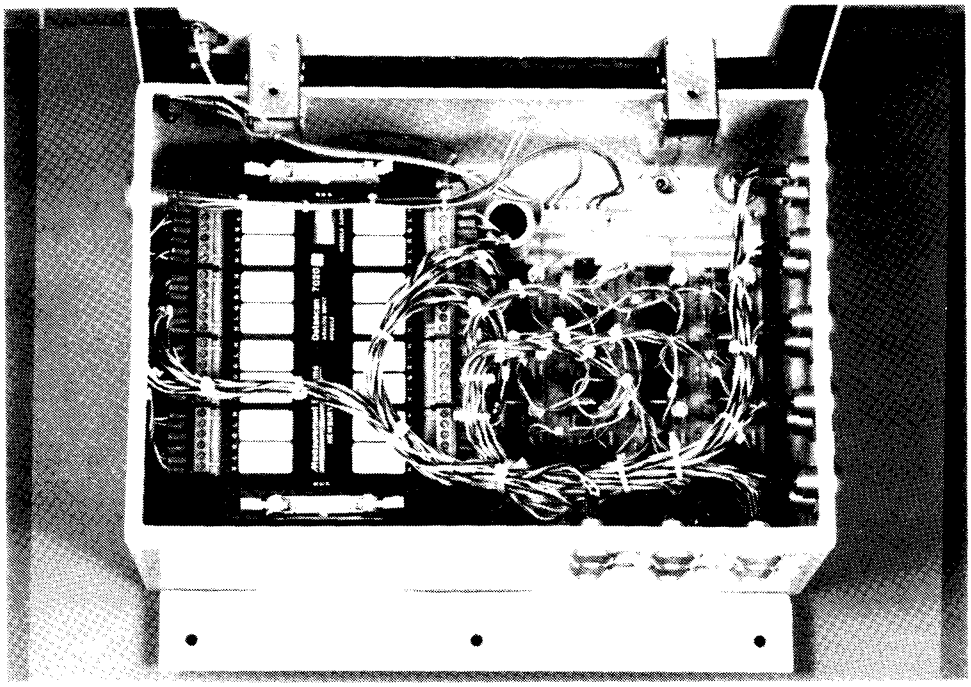


Plate 3.11 Regulator and datascan unit.

and the possibility of total immersion. Each box is designed to be din rail mounted and internally arranged to accommodate a single input module and a dedicated power supply.

3.3.9.2 Power supply

Each transducer requires an energising supply of electric current which cannot be provided by the datascan units. A purpose built "stable" power supply has therefore been designed and incorporated into the system. The supply provides a 5V or 10V dc voltage which is maintained at ± 0.025% of the nominal value.

For the pipe jacking research the power source is generally the national grid although a 20kVa portable generator has been used on one site. The incoming 230V, 50Hz ac supply is first converted to 110V with a centre tapped earth, portable isolation stepdown transformer. This is to satisfy the current safety regulations for site use. The 110V ac is fed to a "rectifier unit", housed in a steel box where it is converted via transformers, bridge rectifiers and a smoothing circuit to low voltage dc supply. The low voltage supply is then fed via screened cables to the steel boxes housing the high stability voltage regulators for each transducer and the datascan analogue input modules. This arrangement ensures that the magnetic fields associated with the transformers are physically separated from the sensitive parts of the system (i.e. the datascan input modules). The low voltage supplies are isolated from earth so that earth currents cannot produce a "noise" signal. Care has also been taken to ensure that variations in the mains supply and spikes do not affect the energising supply to the transducers. Spikes do not easily transfer through transformers and even if they do they are conducted to ground via small value ceramic capacitors in the rectifier unit.

Wherever practical, lamps have been installed to indicate the presence of supply voltage. This helps greatly in fault finding. Each transducer has its own separate supply to enable the system to continue operating in the event of a short circuit in one of the transducer energising outputs. In addition transducer connectors have been standardised so that 4 way connectors are used for signal cables and 5 way and 6 way used for 10V and 5V transducer energisation respectively. Lemo environmentally sealed connectors have been used to interface the transducers to the steel boxes. Wherever possible, cables are screened and twisted pair to reduce the possibility of electro magnetic interference from other equipment.

3.4 Laboratory performance

Reliable, safe calibration procedures are a pre-requisite for field measurements. This is highlighted by the increasing application of modern electronic transducers which in most cases are interpreted as an indirect measurement of a physical quantity. Calibration procedures are therefore an integral part of the instrument quality assurance and should provide reliable information on the response of a single instrument subjected to various kinds of insitu excitation. It is essential that instruments are tested and calibrated on a regular basis using simple procedures. It is also important that calibration normals are incorporated into the procedures. Extensive laboratory calibration of the instruments has been carried out prior to the start of each site contract. A full set of calibrations typically takes 135 hours to complete. Typical instrument performances under laboratory conditions are presented in Table 3.3. All of the instrument types produce near linear repeatable responses. Moisture ingress, the main cause of zero drift in strain gauged transducers, has been eliminated in the designs resulting in small zero changes. Long term changes in the calibration coefficients of the instruments are negligible (<2%) with the exception of the pipe joint pressure cells. These appear to be sensitive to minor damage sustained during extraction from the joints.

Temperature coefficients are not shown in Table 3.3. Field monitoring of pipe jack tunnel temperatures indicates that a stable environment (± 1°C) exists when the instrumented pipe is insitu. Temperature changes are therefore not a major variable.

Instrument type	Working range	Excitation voltage	Calibration coefficients	Combined non linearity & hysteresis (% FS)	Long term zero stability (% FS)	Long term calibration coefficient changes (%)
Contact stress	Radial > 450 kPa Shear > 200 kPa	5V	30 kPa/mV/5V *	± 0.3 ± 0.6	2.5 2.2	1.7 1.8
Pore pressure	3 bar 5 bar 7 bar	5V	3.7 kPa/mV/5V 6.8 kPa/mV/5V 8.8 kPa/mV/5V	± 1.1	0.2	3.8
Pipe joint pressure	40 Ton (20 MPa)	5V	1.7 to 2.0 Tons/mV/10V	± 1.6	1.0	10.0
Tube extensometer	RDP ± 2.5 mm LSC ± 2.5 mm (± 1562 $\mu\varepsilon$)	10V	2.5 $\mu\varepsilon$/mV/10V 236 $\mu\varepsilon$/mV/10V	± 0.6	-	2.0
Pipe joint movement	(i) ± 25 mm (ii) ± 5 mm	10V	11.3 $\mu\varepsilon$/mV/10V 3.7 $\mu\varepsilon$/mV/10V	± 0.35	-	0.5
Jack load cells	200 Tons	10V	13.1 Tons/mV/10V	± 0.7	0.6	0.6
Celesco displacement	(i) 0-100 inch (ii) 0-150 inch	10V	257 μm/mV/10V 411 μm/mV/10V	± 0.15	0	0

Notes:
1. * denotes coupled term unable to state as a single direct calibration coefficient.
2. Values of combined non-linearity etc for the shear term are based on the shear circuit output under the shear calibration loading.

Table 3.3 Laboratory calibration performance.

CHAPTER 4:

SITE SELECTION AND PROCEDURES

4.1 Site selection for instrumentation purposes.

The research contract was set up with the intention of instrumenting pipe jacks on five separate schemes. It was envisaged that one scheme would be provided by each of the participating water companies, and that a number of sites would be offered by each over the contract period, allowing a selection of the most suitable sites to be made. It was further expected that there would be sufficient lead time on all schemes for the research activities to be built into the contract documents, so that the contractor would be fully aware of what he was taking on and the research assistant would have a clearly established position on site.

On this basis, a review was made of the large number of parameters which might affect the performance of pipe jacks. These included:

- Ground material
- Pipe size
- Lubrication
- Depth of cover
- Overburden loading
- Length of drive/position of instrumented pipe
- Pipe joint profile
- Packing material
- Method of excavation
- Time
- Method of pipe manufacture

It is apparent from studying the list that monitoring of five pipe jacks cannot cover all the operational and performance aspects of the construction technique. It was therefore necessary to judicially assign appropriate levels of importance to the variables so that the most significant parameters could be investigated in a relatively controlled manner.

Decisions were then taken as to which would be kept constant and which varied between schemes. The result appears in Table 4.1; it can be seen that the major variables were chosen to be the type of ground, the size of pipe and the type of pipe joint. Drives through cohesive and non-cohesive material were chosen to enable existing analytical ground loading models to be evaluated. Firm clay and sand/selected fill were considered the most suitable soil types since they were reported in the literature as producing the largest jacking forces per metre run of pipeline. Typical lengths and depth were chosen with relatively uniform ground conditions being preferred over the instrumented sections. The use of lubrication and sites with rapidly varying overburden or heavy buildings above the tunnel would be avoided, as they would complicate the already difficult interpretation of the instrument readings. Two pipe sizes, 1200mm and 1800mm internal diameter, were chosen to suit the laboratory work and provide data for extrapolation of results from one size tunnel to another in similar ground types. Both types of UK joint profile were to be investigated to establish which provides the best load distribution and hence least susceptibility to damage through overstressing. The packing material was specified as medium density fibreboard, on the basis of the laboratory work Ripley (1989) ; use of the same material throughout was essential to allow proper calibration of the pressure cells in the pipe joints.

It was considered that the method of excavation would have a significant effect on the rate of progress and straightness of a drive. Hand drives were preferred to machine drives because it was felt that the tunnelling machines were generally only used in poor ground conditions and would generate significant electrical noise in the tunnel. In standardising the method of excavation it was hoped that rates of progress would be similar between contracts.

Finally, it was always intended that the first drive would be a pilot scheme, using

	Scheme	1	2	3	4	5
Major Variables to be investigated in research	Ground Type	Firm Clay	Firm Clay	Sand/ Gravel	Sand/ Gravel	Firm Clay
	Pipe Size	1200	1800	1800	1200	1800
	Pipe Joint	Butt	In Wall	Butt	In Wall	Butt
To be kept as consistent as contract selection will allow.	Depth	5-7m	5-7m	5-7m	5-7m	9-12m
	Length	<100m	<100m	<100m	<100m	100-250m
	Positon of Test Pipe	Pipe No.3	Pipe No.3	Mid	Pipe No.3	Mid
	Lubrication	No	No	No	No	No
	Packer Type	MDF	MDF	MDF	MDF	MDF
	Excavation Technique	Hand	Hand	Hand	Hand	Machine
	Pipe Manufacturing Process	Any	Any	Any	Any	Any
	Overburden Loading	Simple	Simple	Simple	Simple	Simple
	Monitoring Period	Short Term	Short Term	Short Term	Short Term	Long Term

Notes: MDF - Medium Density Fibreboard

Table 4.1 Ideal site requirements for research.

all the instruments but in reduced numbers, to identify problems with the instrument designs, site procedures and data collection. There would then be time available to make any modifications necessary before the four main schemes, which would use the full instrument sets.

In practice the process of selecting schemes was very different. Details of the actual instrumented schemes are given in Table 4.2. A location map showing the geographical distribution of the sites is included as Figure 4.1.

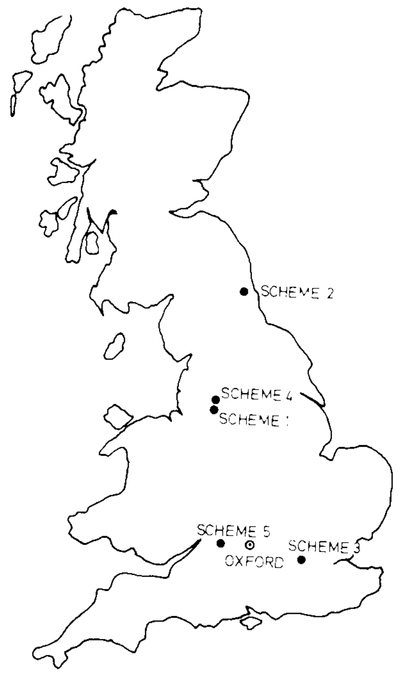


Figure 4.1 Geographical distribution of sites.

The pilot scheme proceeded more or less as planned, but by then the effects of privatisation of the water industry were having a major influence on both the organisation and immediate plans of the various areas. Although a number of pipe jacking jobs were identified, very few were even reasonably suitable in terms of location, timing and ground conditions for the research work.

After a long delay, the next two were taken on at short notice, with the contractors already on site, and the research introduced as a variation to the contract. Scheme 2 was

	1	2	3	4	5
Date	August 1990	January 1991	March 1991	July 1991	December 1991
Location	Bolton, Lancs	Gateshead, Tyneside	Honor Oak, SE London	Chorley, Lancs	Cheltenham, Glos
Client	NW Water Bolton M.B.C	Northumbrian Water	Thames Water	NW Water Chorley D.C.	Severn Trent Water Cheltenham BC
Consultant	---	---	Binnie/Taylor Woodrow	Halcrow	---
Contractor	Laserbore	DCT	Barhale	Barhale	Lilley
Pipe Supplier	Buchan	ARC	Buchan	Spun Concrete	Spun Concrete
Pipe I.D (mm)	1200	1350	1800	1500	1200
Ground Type	Stiff Glacial Clay	Weathered Mudstone	London Clay	Dense Silty Sand	Loose Sand & Gravel
Cover (m)	1.3 - 1.5	7 - 11	11 - 21	7 - 10	4 - 7
Drive Length (m)	60	110 *	78	158	384
Position of Test Pipe	Pipe No.3	Pipe No.10	Pipe No.15	Pipe No.16	85m from front end.
Lubrication	No	No	No	No/Yes	Yes
Packer +	MDF	MDF	MDF	MDF	MDF
Excavation	Hand	Hand	Hand	Hand	Slurry T.B.M.

Notes: * Monitoring only for part of drive
+ MDF = Medium Density Fibreboard

Table 4.2 Details of actual schemes monitored.

a late replacement for one that had been in an advanced stage of planning, but was then postponed; while scheme 3 closely followed scheme 2 requiring an early exit from scheme 2 and negligible time for repair or recalibration of instruments between jobs.

In parallel with schemes 2 and 3, arrangements were being made for scheme 4 on a job in Yorkshire. Contract documents incorporating the research work were in preparation, when this scheme was also postponed and would be too late to be included in the three year programme. Fortunately two other sites became available, both in cohesionless material, and this allowed a full set of five schemes in a variety of ground conditions. In both cases, the research was introduced as a variation to a contract that had already been let. Thus only in the case of the pilot test was the intended procedure followed.

The choice of parameters studied has thus in effect been decided by the availability of sites, although an attempt has still been made to keep some coherence in the research programme. The main variable has turned out to be the ground type, with drives in stiff glacial clay, weathered mudstone, stiff plastic (London) clay, dense fine silty sand and loose sand and gravel. Pipe internal diameter has varied between 1200mm and 1800mm, and cover depths from 1.5m to as much as 21m under an embankment.

One variable has disappeared. Either as a result of field experience, or from the findings of the laboratory tests, the superiority of butt to in-wall joints whenever significant jacking forces are expected appears to have been accepted, and steel-banded pipes have been specified for all schemes. The three drives through cohesive ground have been completed without the use of lubricant, though with some concern on scheme 3. The non-cohesive drives were lubricated, although in scheme 4 it was possible to monitor part of the drive prior to lubricating. Schemes 1 to 4 were hand excavated. The final scheme was driven using an Euro-Iseki Unclemole tunnel boring machine.

4.2 Details of the schemes

4.2.1 Scheme 1

The pilot test was completed successfully on a 1200mm internal diameter, 60m long drive in Bolton. The client, Bolton Metropolitan Borough Council acting as agents for North West Water Ltd., designed the Bury Road Resewerage Scheme to alleviate foul flooding which was affecting premises in the Breightmet area of Bolton. Part of the scheme, the Milnthorpe Road Retention Tank, was brought forward as a separate phase when identified by North West Water as being a suitable site for the pilot test.

The contract was awarded to Laserbore Ltd. Work commenced on site 23 July 1990 with the research element being carried out between 25 July and 24 August 1990.

Pipe jacking was chosen because the retention tank passed directly below the driveways of a number of houses and the client wished to minimise inconvenience to the owners and a superstore which could only be accessed from Milnthorpe Road, Figure 4.2a.

Vertically cast reinforced concrete pipes supplied by C.V. Buchan Ltd were used. The lead pipe was fitted with a single ground convergence indicator and the main instrumented pipe was positioned 5m behind the shield. This enabled the pipes to be pushed out of the tunnel into the reception pit and the main instrumented pipe to be jacked back into the tunnel for a distance of two pipe lengths.

A simplified longitudinal section through the drive is shown in Figure 4.2b. The tank was constructed with a cover depth of approximately 1.5m. The face log taken during the construction period, Figure 4.3, illustrates that the drive was predominently through stiff glacial clay. Soil strength and classification details were obtained from insitu and laboratory testing of material at four chainages. A summary of the results appears in Table 4.3 which indicates that the undrained shear strength increased from 150kPa to 300kPa with increasing tunnel length. Stability indices based on these values suggested that the face and tunnel bore would stand unsupported for some time after excavation.

a) Site Plan

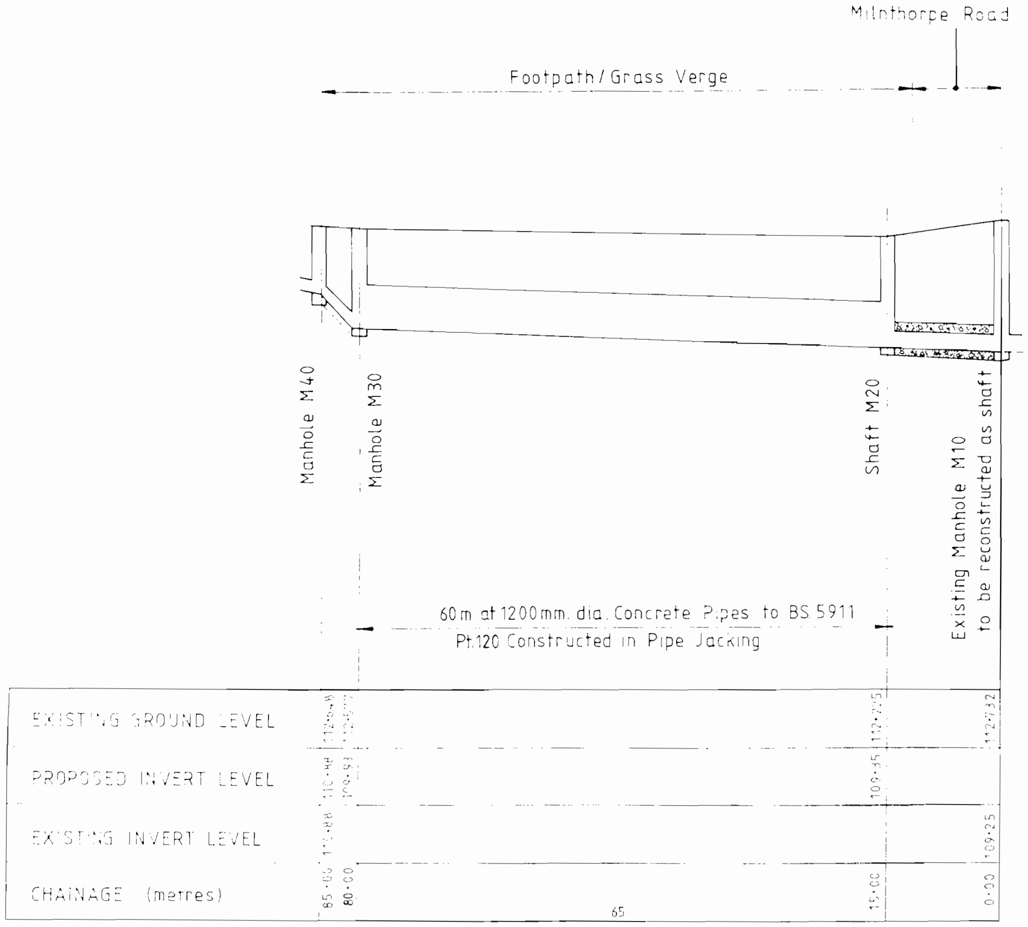


Figure 4.2 Details of scheme 1. **b) Simplified longitudinal section**

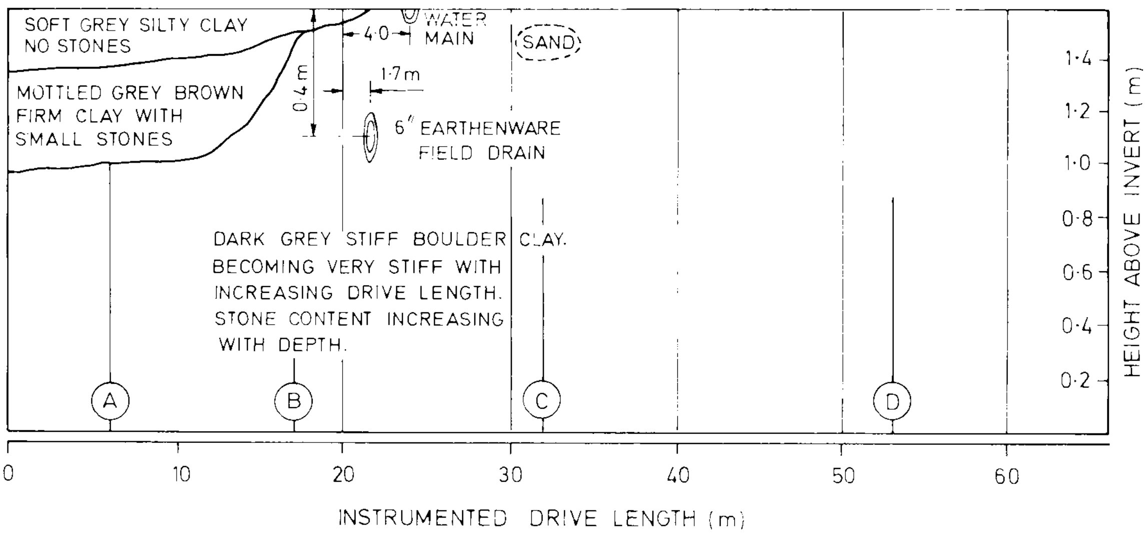


Figure 4.3 Face log from scheme 1.

Face sampling & insitu testing

		CH 6m 1	CH 17.5m 2	CH 32m 3	CH 53m 4
Soil description		Soft grey silty clay / Mottled grey-brown firm clay / Stiff grey boulder clay with stones.	Soft grey silty clay. / Grey/dark brown stoney stiff clay	Soft grey silty clay. Dark grey stiff boulder clay	Dark grey stiff boulder clay
Soil Density	Mg/m^3 2.4, 2.2, 2.0				
Index Testing	% 40, 30, 20, 10	LL, PL	LL		
Undrained Shear Strength S_u, ϕ_u	1, 2, 3, 4, 5	(3&5) 150kPa 0°	(2,4,5) 117kPa [200] Av 17° [0]	(2,4,5) 8.5kPa [245] Av 30° [0]	(2,4,5) 136kPa [305] Av 27° [0]
Consolidated Undrained with p.w.p. measurement c', ϕ'	1, 2, 3, 4, 5	(2,4,5) 16kPa 30°	(1,3,5) 0kPa 39°	(1,3,5) 6kPa 30°	(1,3,5) 25 kPa 34°
Pocket penetrometer Av (kPa)	300mm 1200 300	72 180 169	82 196 210	132 169 220	240 259 258
Shear Vane Av (kPa)	300mm 1200 300	52 136 147	54 160 176	70 149 182	>188 >188 >188

Table 4.3 Insitu and laboratory test data from face samples obtained during Scheme1.

[] denotes shear strength corresponding to $\phi_u = 0$.

4.2.2 Scheme 2

Instrumentation and monitoring of scheme 2 was carried out between 21 January and 25 February 1991 on a 110m hand driven 1350mm internal diameter tunnel mainly through weathered mudstone, coal measures and glacial clay which was part of Northumbrian Water's Tyneside Western Interceptor Sewer Project, Phase 3. The contractor was DCT and pipes were supplied by ARC.

A location plan, longitudinal section indicating the depth of tunnel and different soil types obtained from trial boreholes and a more detailed tunnel face log recorded during construction are included as Figure 4.4(a-c). Progress on the drive was delayed by one week when it encountered a backfilled mine shaft, approximately 2m square 15m into the drive, which was negotiated by constructing a timber heading and casting a reinforced concrete base over which the pipeline was jacked. The need to run scheme 3 hard on the heels of scheme 2, and the unforeseen delay resulted in only 40m of the drive being completed before leaving site. The instrumented pipe was positioned 25m behind the shield; 15m of the drive was monitored in detail.

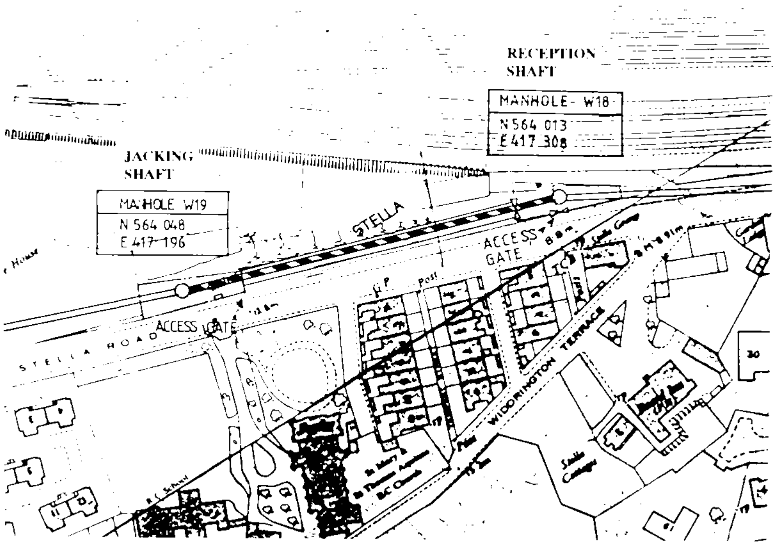
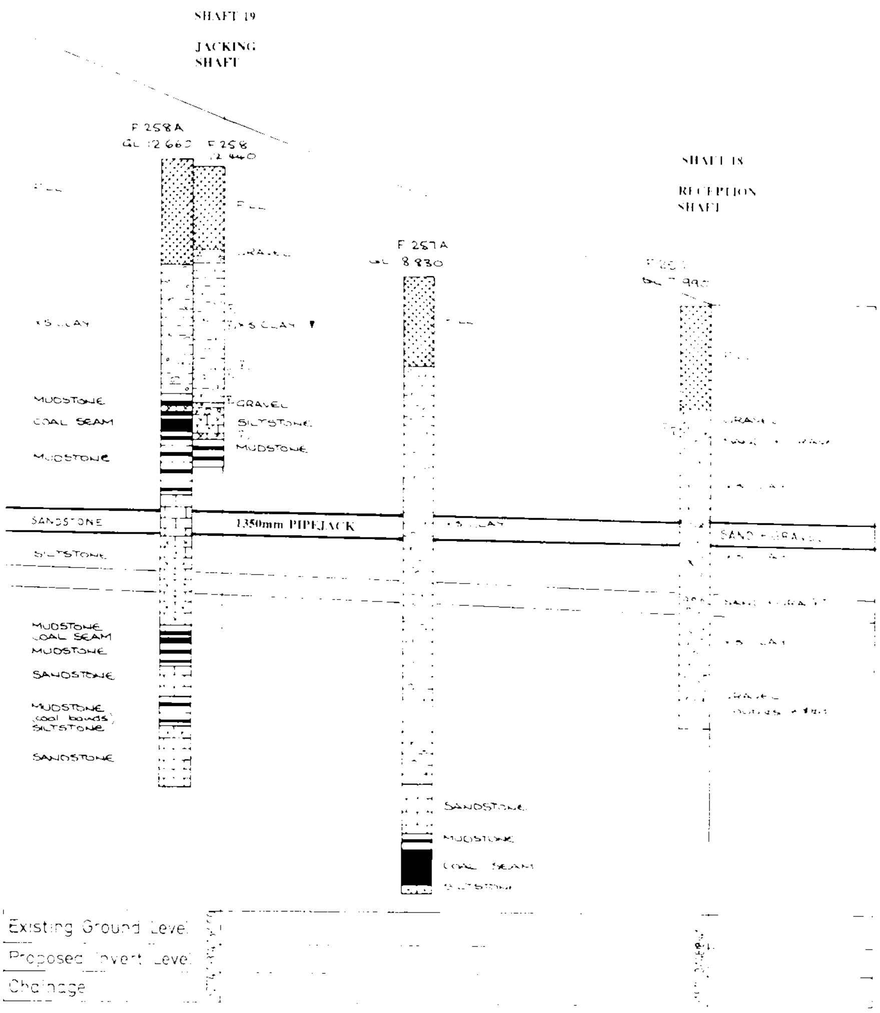
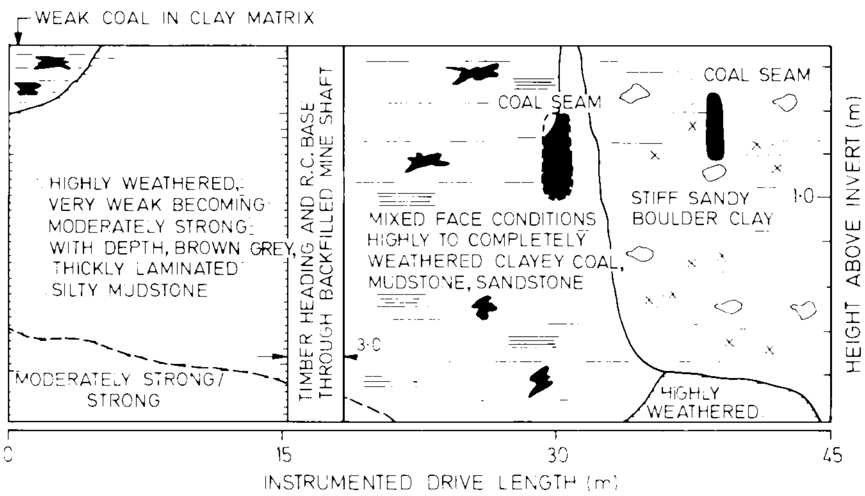


Figure 4.4a Site Plan.



b) Simplified longitudinal section and borehole data.



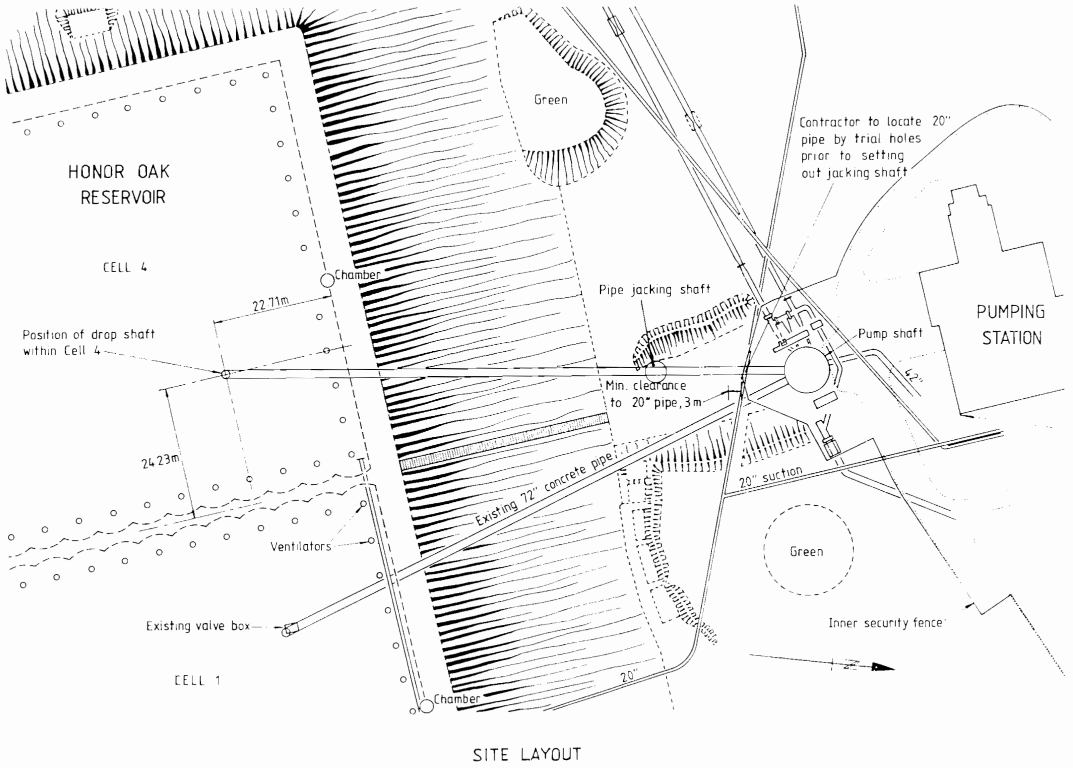
c) Face log.

Figure 4.4 Details of scheme 2.

4.2.3 Scheme 3

Scheme 3 involved instrumenting a pressurised pipeline which was installed by pipejacking at a reservoir in south east London to improve the mains water supply by drawing more fully on water storage and pumping facilities. The original 19th Century brick reservoir consisted of four cells interconnected by a network of pipes. Water flowed, via one cell, to a pumping shaft where it was pumped into the mains system. The new pipeline, some 110m in length, linked a second cell directly to the pumping station, thereby improving operating capacity.

The scheme was put forward by Thames Water Utilities and was awarded to Barhale Construction Ltd. The extent of the civil engineering works is illustrated in the simplified plan and longitudinal section of Figure 4.5. A temporary working shaft, approximately 16 metres deep was sunk on the line of the pipe jack and a short 25m drive made into the pumping shaft. This backshunt, once grouted, provided the thrust resistance for the longer instrumented drive under the reservoir. Simultaneously with the pipe jacking work, a reception shaft was sunk through the floor of the reservoir. Since the line was to be subject to a 2.5 bar pressure test, non-standard 1830mm internal diameter steel banded, concrete pipes with a wall thickness of 225mm were required and supplied by C.V. Buchan Ltd. Work started on the instrumented hand drive on 4 March 1991 and was completed by 22 March 1991, including 24 hour shiftwork between 18-22 March. The instrumented pipe was positioned 35m behind the shield. Cover depth varied from 11m at the temporary shaft to 21m under the reservoir. The ground material, stiff overconsolidated London clay, was consistent throughout the drive as indicated in the face log of Figure 4.6. The highly fissured state of the material resulted in preferential fracturing during excavation as indicated by the blocky nature of the spoil, Plate 4.1. A laboratory test programme based on samples obtained at chainage 22m was carried out by the Thames Geotechnics Group; a summary of the results appears in Table 4.4.



a) Site Plan

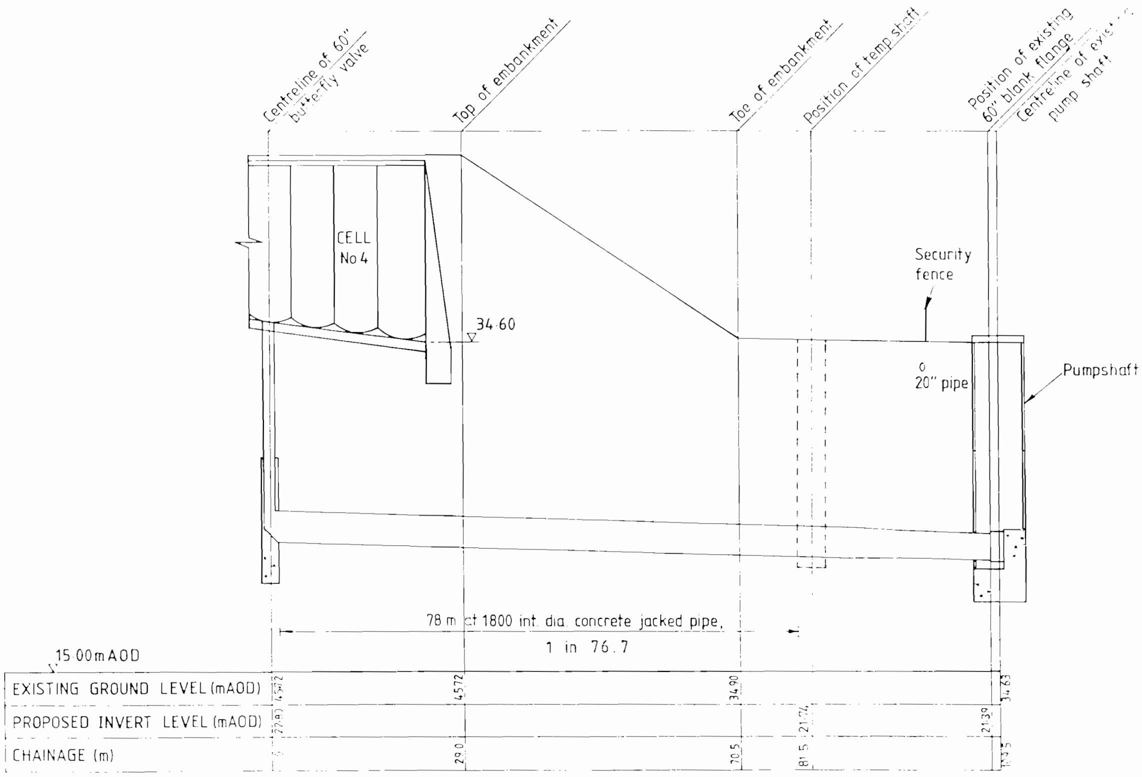


Figure 4.5 Details of scheme 3. **b) Simplified longitudinal section.**

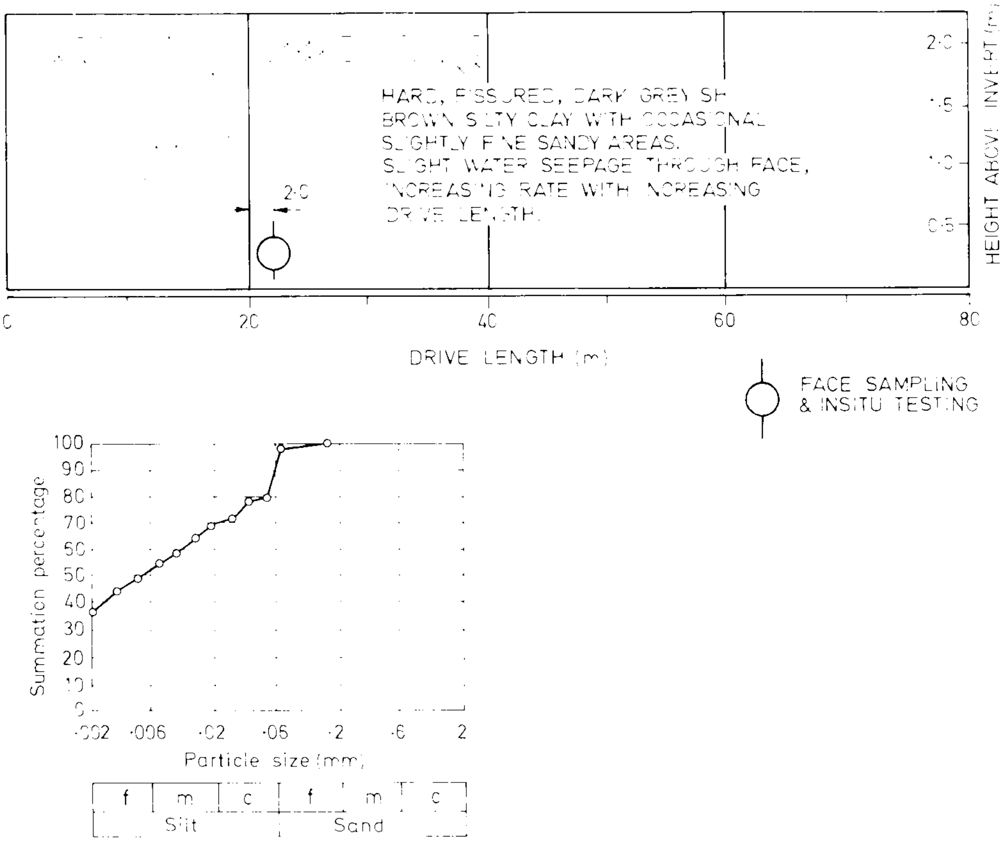


Figure 4.6 Face log and grading curve for scheme 3.

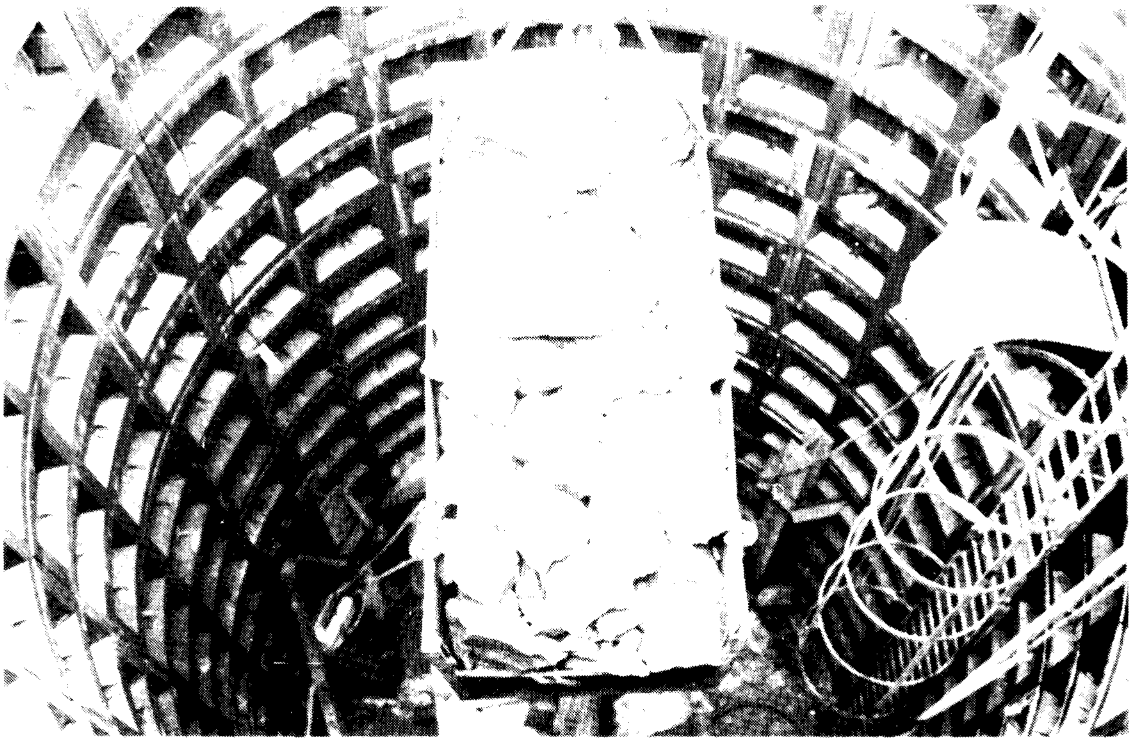


Plate 4.1 Excavated material.

Honor Oak

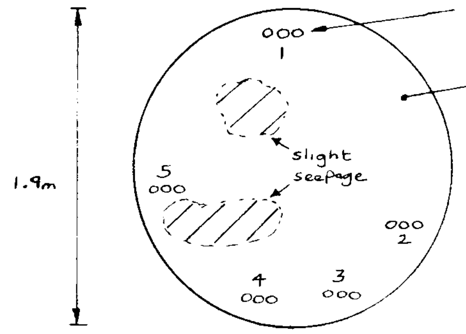
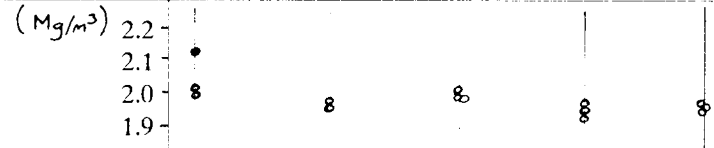
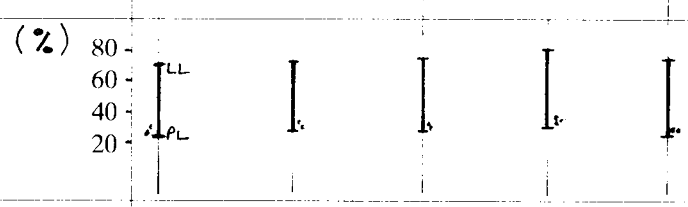
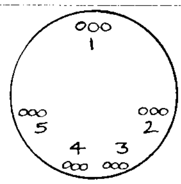
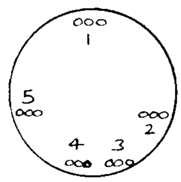
		Location: 1	2	3	4	5
Soil description		Hard, fissured dark greyish brown silty clay with occasional slightly fine sandy areas				
Soil density						
Index Testing						
Undrained Shear Strength S_u		(1A,B,C) 512 kPa	(2A,B,C) 225 kPa	(3A,B,C) 101 kPa [419]		(5A,B,C) 603 kPa Av
ϕ_u		0°	0°	18.6° [0°]		0°
Consolidated Undrained with p.w.p. measurement c'		(4A,B,C) 50 kPa				
ϕ'		31°				
Pocket penetrometer		Ground too stiff for results to be obtained				
Shear Vane		Ground too stiff for results to be obtained				

Table 4.4 Laboratory test data from face samples obtained at chainage 22m during Scheme 3.

[] denotes shear strength corresponding to $\phi_u = 0$.

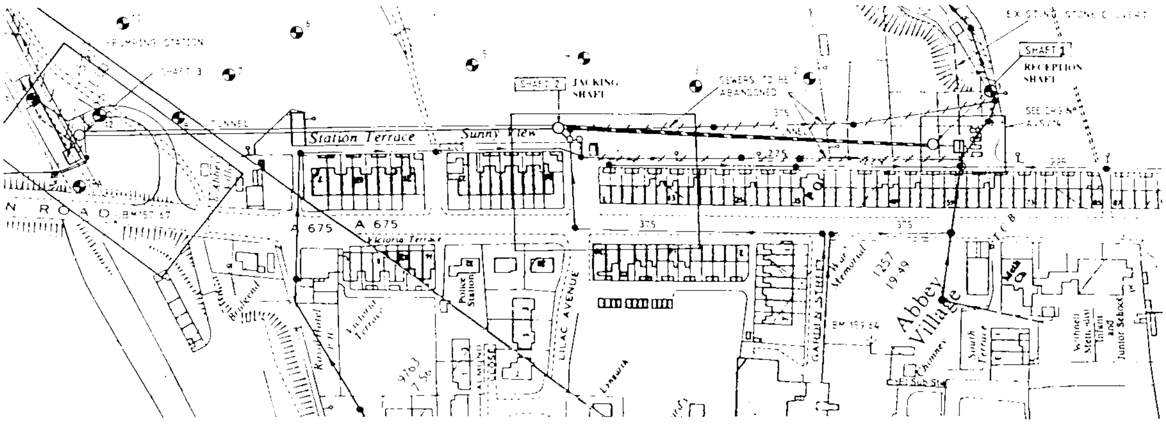
4.2.4 Scheme 4

Instrumentation and monitoring of scheme 4 was carried out between 15 July and 9 August 1991 on a 158m, 1470mm internal diameter drive from shaft 2 to shaft 1 in Figure 4.7. The scheme was part of North West Water's Abbey Village/Pleasington Sewerage scheme, Stage 2. The contractor was Barhale Construction, who had carried out the work on scheme 3, and Halcrow were the supervising engineer. The concrete pipes were supplied by Spun Concrete Ltd.

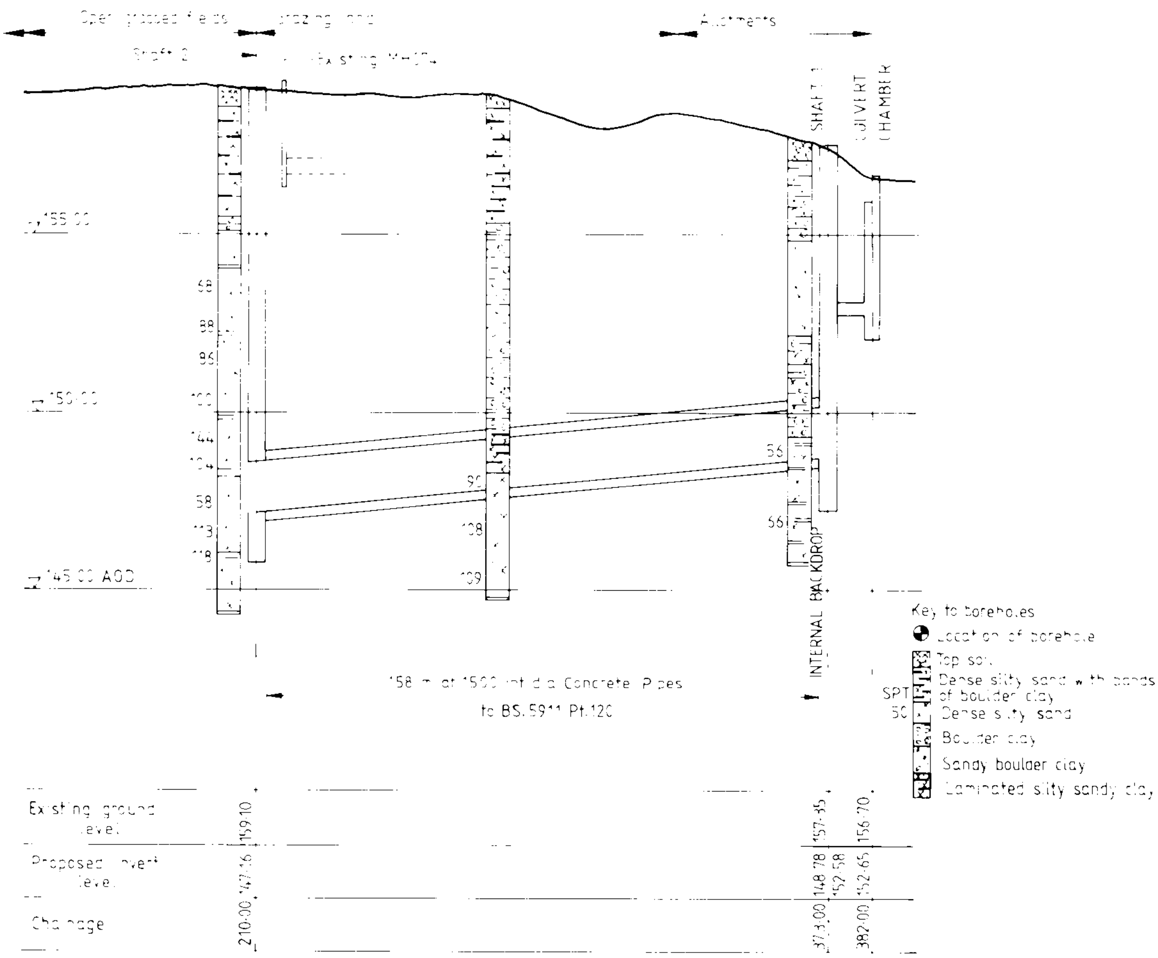
A detailed face log taken during the monitoring period shows that the tunnel was driven through initially dense brown silty sand with bands of firm clay, then laminated sandy silt with occasional bands of stiff clay and finally dense brown sand with stiff sandy boulder clay intrusions. Grading curves for the predominently non-cohesive materials are included with the log in Figure 4.8. Stability of the face was good as illustrated in Plates 4.2 and 4.3 suggesting that rapid collapse of the ground onto the pipes was unlikely and that the intended use of lubrication could be deferred until later in the drive, as requested by the research group. The main instrumented pipe was inserted 40m behind the shield; lubrication was introduced after 82m and the research equipment was removed after 115m of the drive was complete, to allow sufficient preparations for scheme 5. The additional site costs for this contract were borne by Yorkshire Water.

4.2.5 Scheme 5

The Hewlett Road Interceptor Sewer forms part of the All Saints Foul and Surface Water Resewerage strategy of Cheltenham Borough Council who are agents for Severn Trent Water. The 1200mm internal diameter sewer runs below the centre line of a major road and involved tunnelling through sands and gravels below the water table, Figure 4.9, which made standard tunnelling techniques difficult without the use of compressed air or

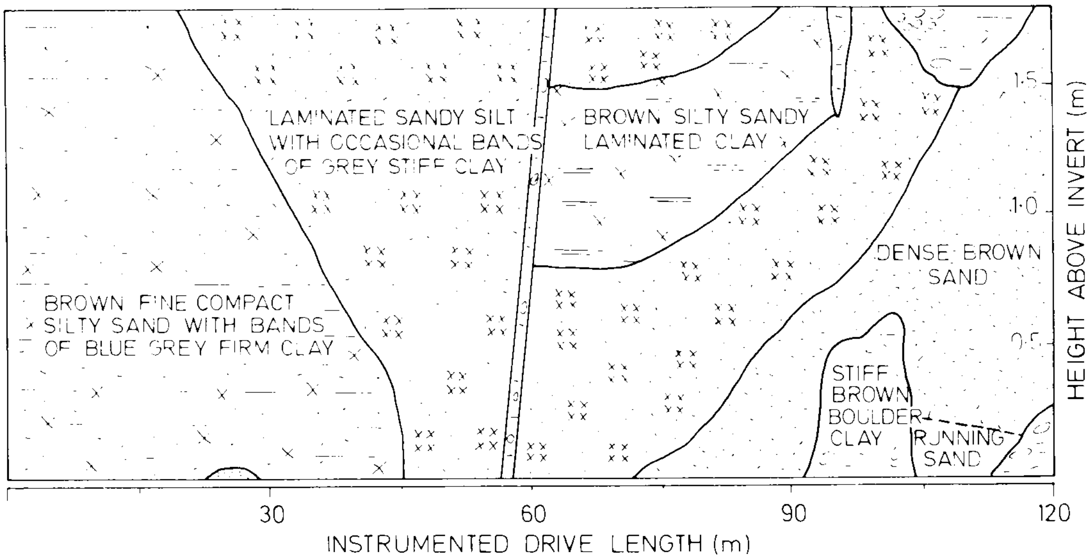


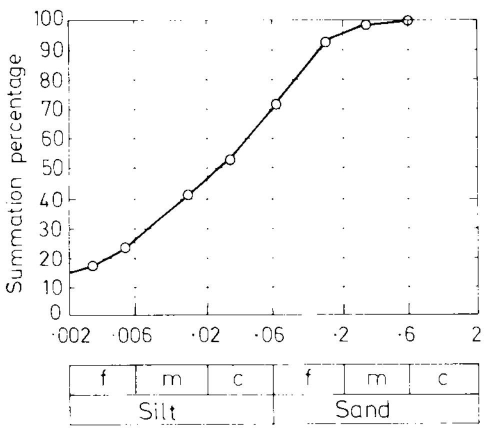
a) Site Plan

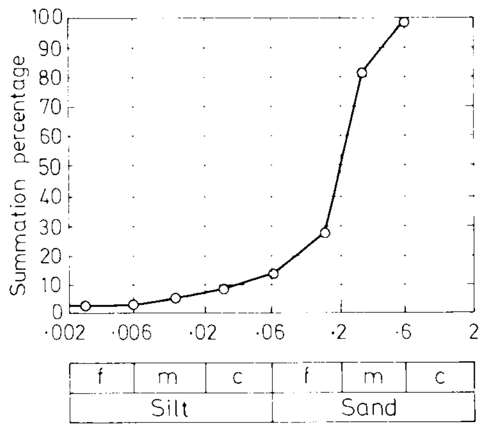


b) Simplified longitudinal section with borehole data

Figure 4.7 Details of scheme 4.







Laminated sandy silt **Silty sand**

Figure 4.8 Face log and grading curves from scheme 4.

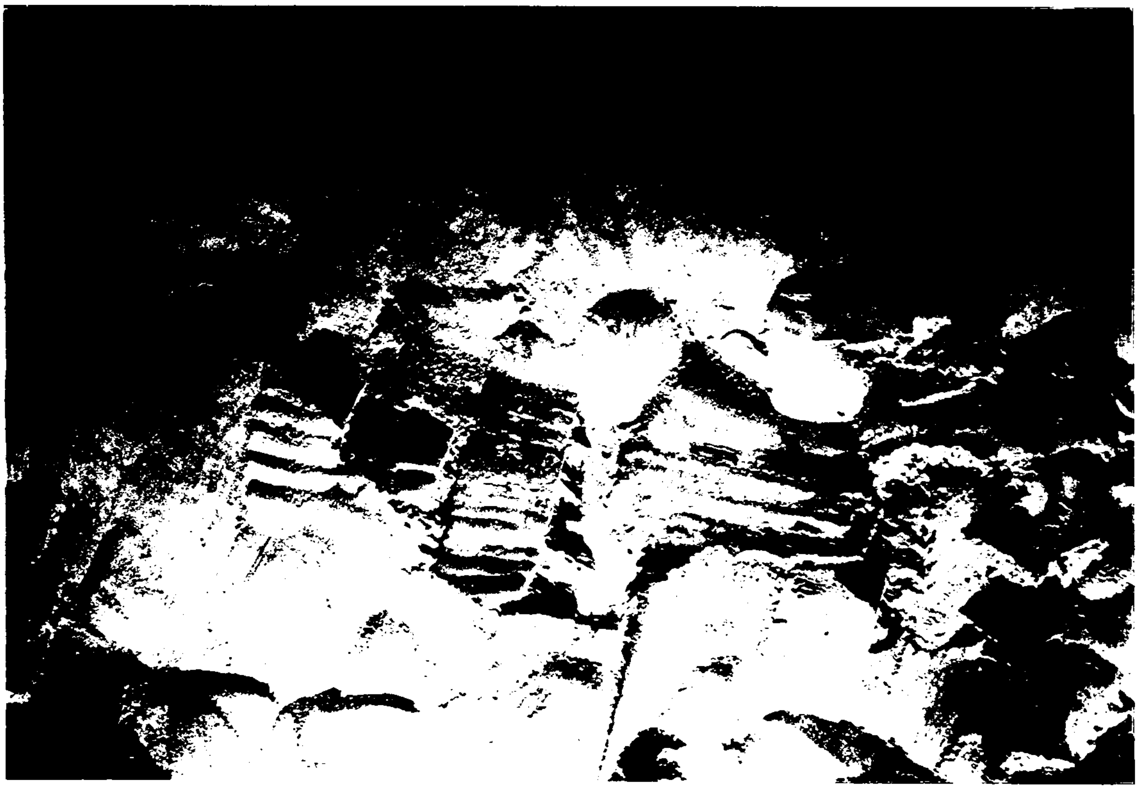
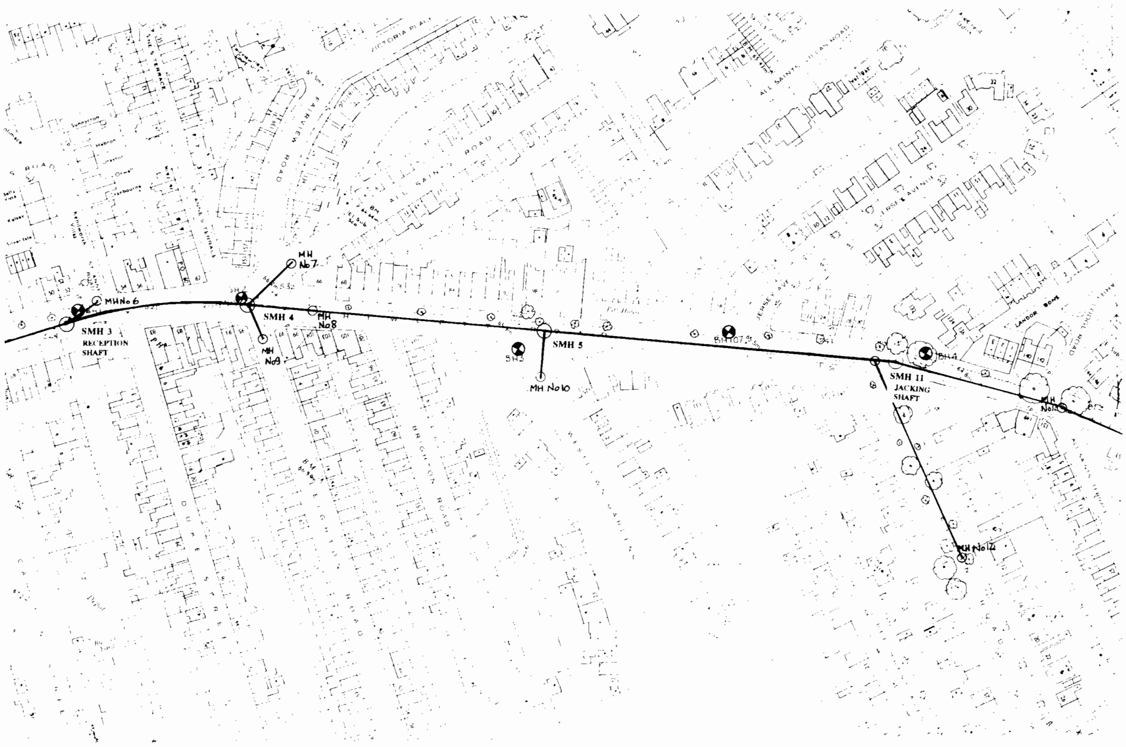


Plate 4.2 Fine silty sand with bands of blue grey firm clay.

Plate 4.3 Laminated sandy silt.



a) Site plan.

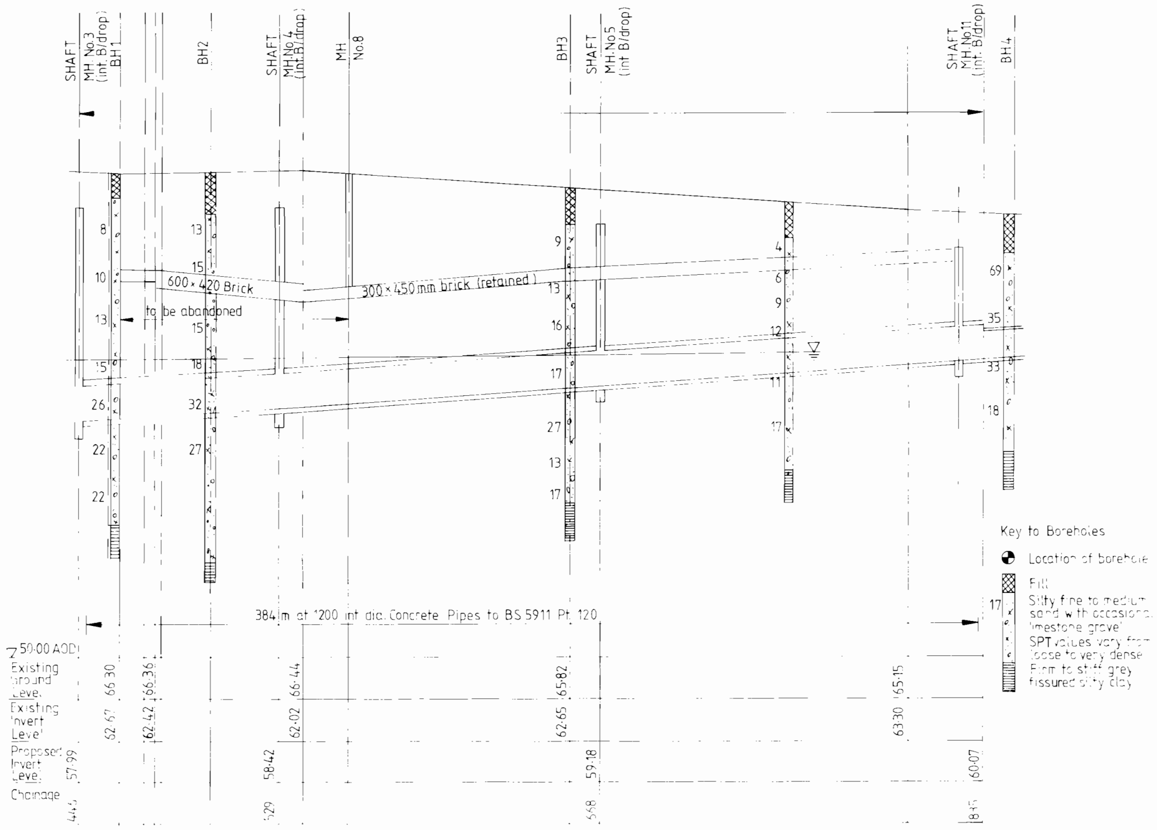


Figure 4.9 Details of scheme 5. **b) Simplified longitudinal section.**

ground water lowering techniques. The relatively shallow cover depth of the tunnel, 4-7m, and the presence of masonry houses built around the turn of the century, without substantial foundations, meant both techniques could be fraught with potential problems.

The contract was awarded to Lilley Construction based on the use of an Euro-Iseki Unclemole, which provides full face bentonite pressure support and incorporates an eccentric cone crusher head which reduces cobbles and small boulders (up to 30% of shield outside diameter in size) sufficiently to enable them to be transported away by the pumped bentonite spoil removal system. The contractor elected to drive the total 360m from shaft 11 using three interjacks at various locations along the pipeline.

The pipejacking research was carried out between 5 November and 11 December 1991. The instrumented pipe was positioned 84m behind the machine and was the first full length pipe after the 400m radius curve; the site monitoring was completed after installation of 260m of pipeline.

As a separate exercise an additional survey was carried out to investigate the effects of the tunnelling operations on the surrounding ground using piezometer and surface/subsurface settlement stations. Installation of the boreholes and instruments was carried out by Soil Instruments Ltd., and subsequent logging of the instruments was carried out by the Oxford pipejacking research group.

4.3 Planning and execution of the fieldwork

The success of site based research is not only dependent upon good design of instruments and data acquisition systems but also thorough planning of the site procedures and incorporation of the research activities into the contract documents, so that subsequent contractual pressures are minimised. Pre-contract meetings with the client and the contractor were extremely valuable in explaining the aims of the research and understanding the roles and motivations of the parties involved. A summary of the important research related bill items is included as Table 4.5.

ITEM DESCRIPTION	UNIT
1. Supply of special pipe.	Sum
2. Provide, establish and maintain protective rigid steel liner	Sum
3. Removal of protective rigid steel liner	Sum
4. Installation of compression load cells between thrust ring and jack rams and mounting of Celesco displacement unit and data acquisition box in the thrust pit.	nr
5. Removal of pit bottom instruments	nr
6. Standing by pipe jacking driving operations	hr
7. Supply and fitting of 100x12mm thick medium density fibreboard to BS1142: Parts 1 and 2 as packer to joint annular ring in joints with pressure cells	nr
8. Recovery of joint pressure cells from pipe joint and make good packing material	nr
9. Recovery by overcoring of pore pressure probes and contact stress cells, removal of cores containing instruments and making good holes.	nr
10. Entry into tunnel for inspection and survey	hr

Table 4.5 Main research related bill items

Other items of a more general nature including transportation and handling of the equipment container, electricity supply, safety, security and the contractor's responsibility to the instruments were also covered.

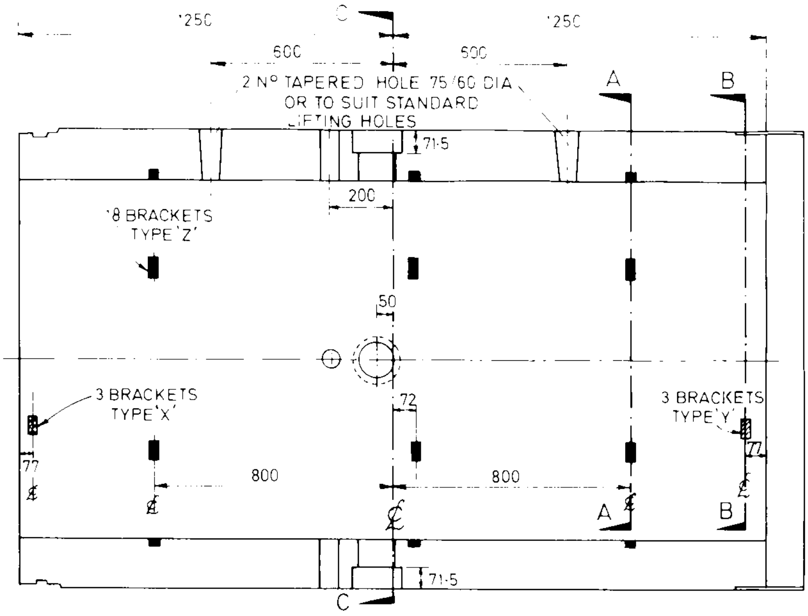
The primary bill items are all activities which can have a major disruptive effect on the contractor's programme of work. Fitting out of the instrumented pipe was an offline activity which was the responsibility of the Research Assistant. Unforeseen delays caused by the research were covered by an agreed standing by rate.

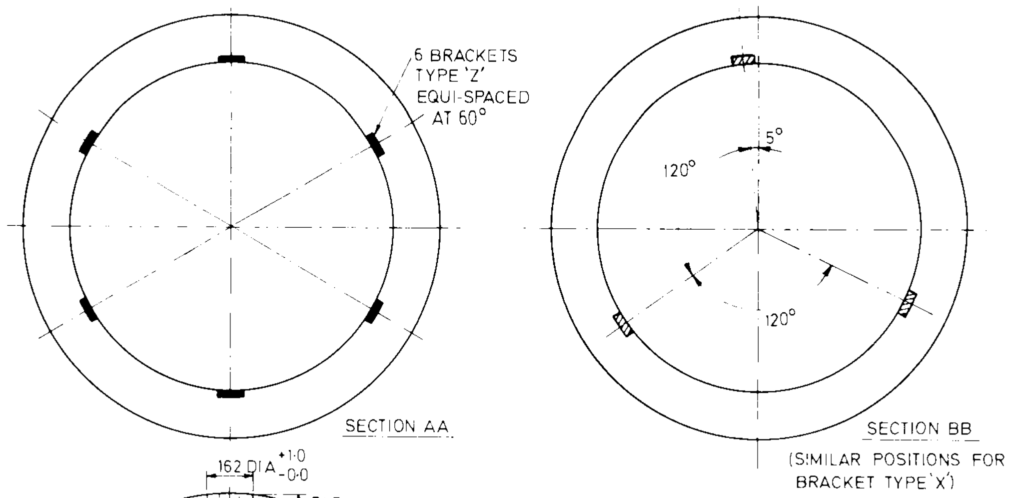
4.3.1 Instrument installation

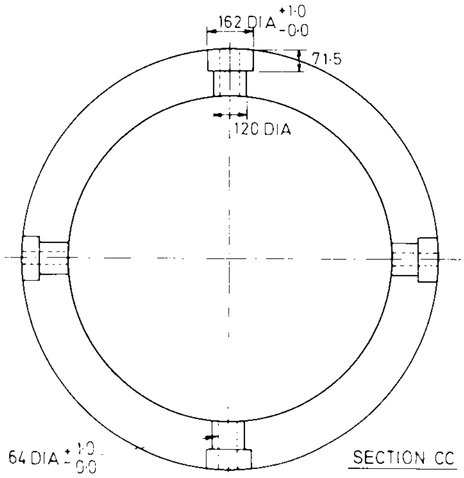
As far as possible, preparation of the instrumented pipes took place off-line. The exact methods of fixing the instruments varied with the way in which the pipes were formed (vertically cast or centrifugally spun). The high risk of instrument damage during the casting process and subsequent delivery to site ruled out installation at the pipeworks. It was therefore necessary to "build in" the necessary holes, inserts or brackets, Figure 4.10, and deliver the pipes to site with sufficient time to complete installation. Plate 4.4 illustrates the process adopted for pipes produced by the vertically cast method; high tolerance circular inserts were used between the inner and outer mould faces to produce holes for the glued in-wall instruments and pre-drilled tufnel tapered blocks for later tapping and bolting of the surface mounted instruments.

Pipes produced by the centrifugally spun process presented a problem with regard to the provision of surface mounted inserts because the internal surface was not a cast face. Bracket assemblies similar to that shown in Plate 4.5 were epoxy grouted into pre-drilled holes to provide correctly spaced threaded dowels.

On site, the first priority was to install the jacking pit instruments. The jack load cells were coupled to the rams, Plate 4.6 and the Celesco displacement unit mounted above the tunnel entrance and hooked over the steel band of the pipe in the jacking pit, Plate 4.7. Once communication was established between the pit bottom data acquisition box and the PC, data could be collected on the initial pipeline installation loads, and work could start on the "special" pipe. The instruments were fitted while the pipe was on the surface, and only parts of the joint movement indicators had to be glued in place after the pipe was in the tunnel. Plate 4.8 illustrates a typical instrumented pipe prior to lowering into the jacking pit. The total time for assembly and system checking took between two and three days. It was convenient over part of this period to monitor the effects of ambient temperature fluctuations on each of the instrument types for subsequent temperature compensation.







	Pore				Contact Stress			
Scheme	N°	top	axis	bottom	N°	top	axis	bottom
1	2	1050	-	1050	4	1200	1200	1200
2	2	-	-	721 1755	2	-	-	572 1914
3	4	1035	1035	1035	4	1200	1200	1200
4	4	1050	1050	1050	4	1200	1200	1200
5	5	1050	1050	600 1900	5	1200	1200	750 1750

Notes: 1. - no cell incorporated.
2. All dimensions in mm.

Figure 4.10 Layout of instrument pipe.

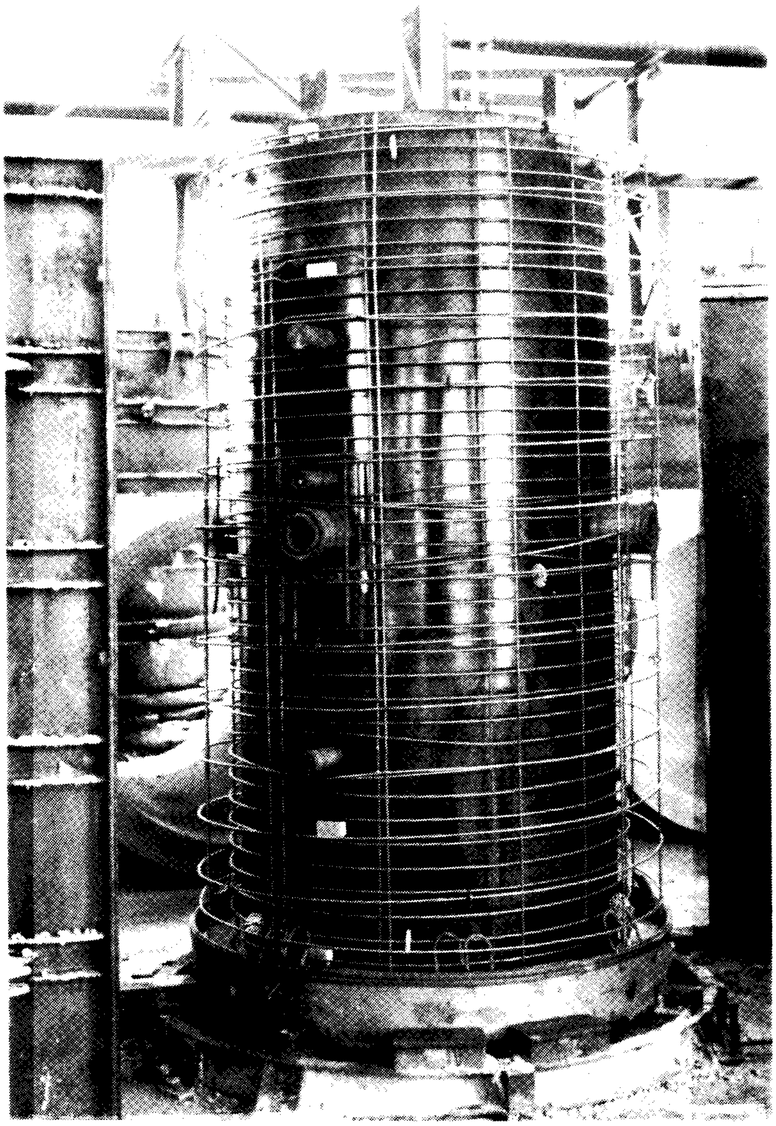


Plate 4.4 Casting the instrument pipe.

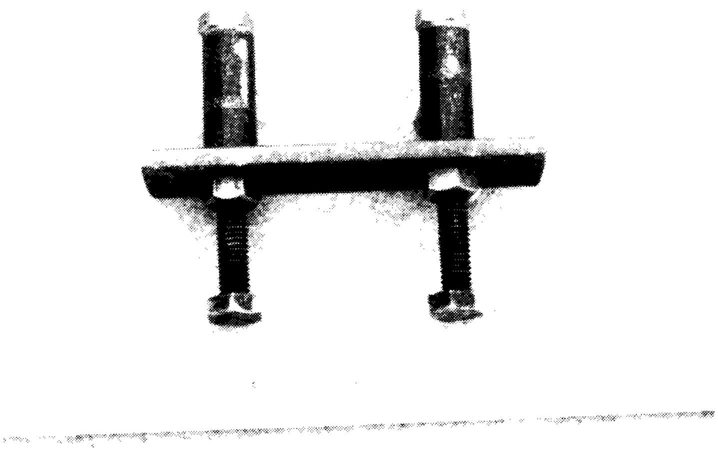


Plate 4.5 Bracket assembly for a spun pipe.

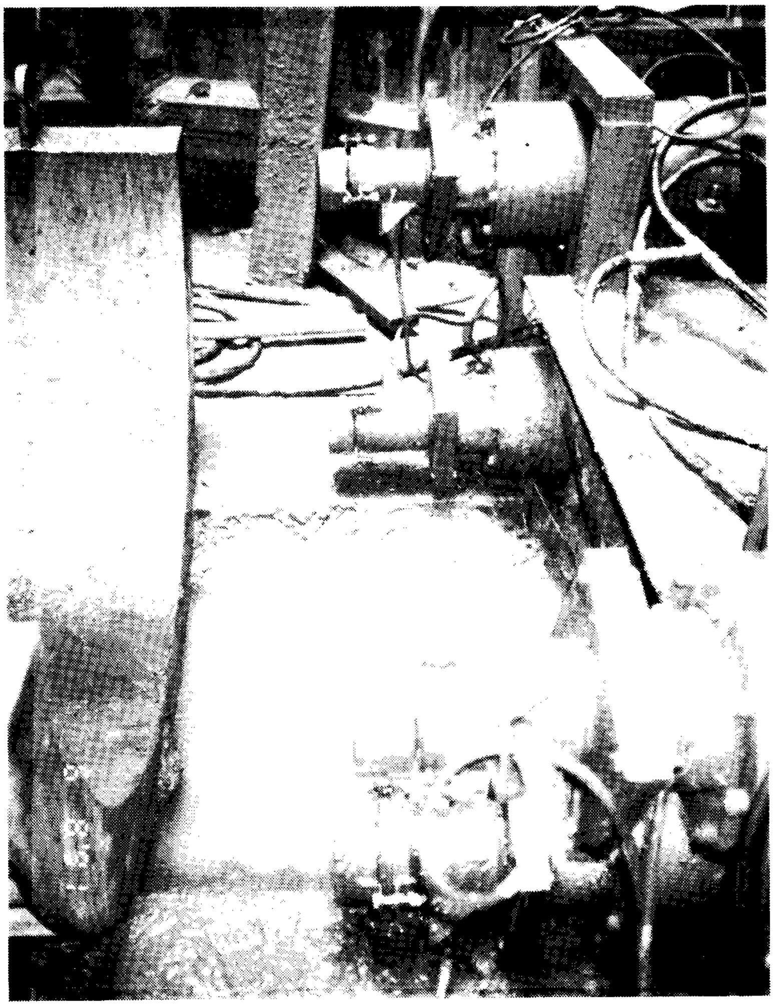


Plate 4.6 Coupled jack ram load cells.

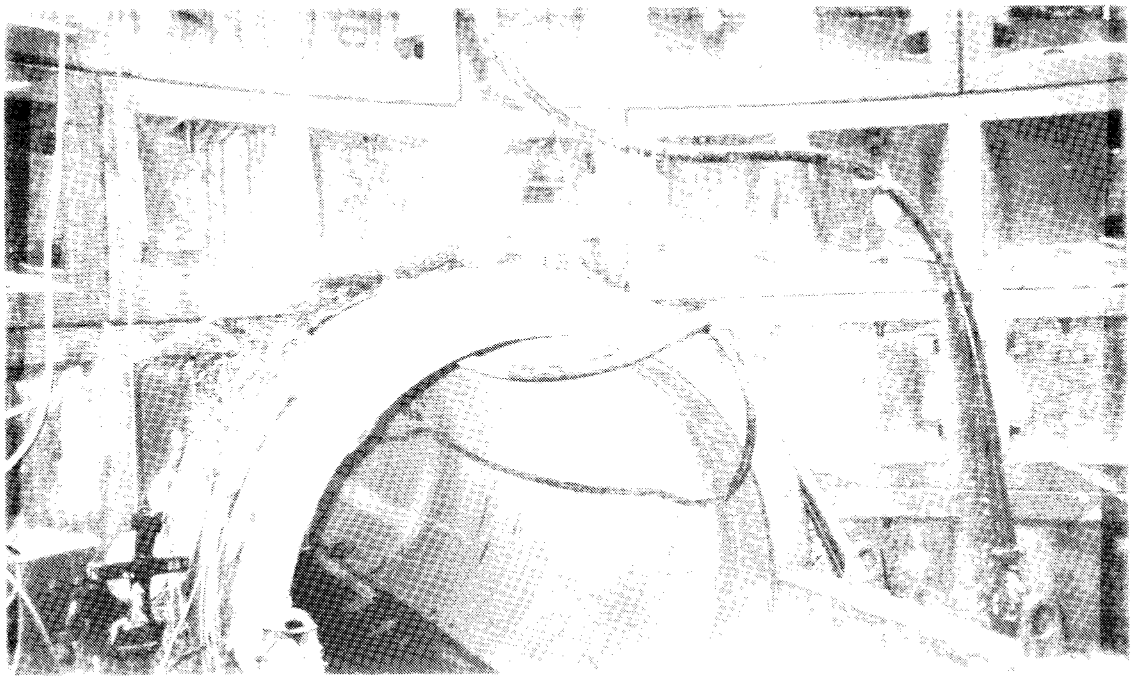


Plate 4.7 Positioning of Celesco displacement unit.

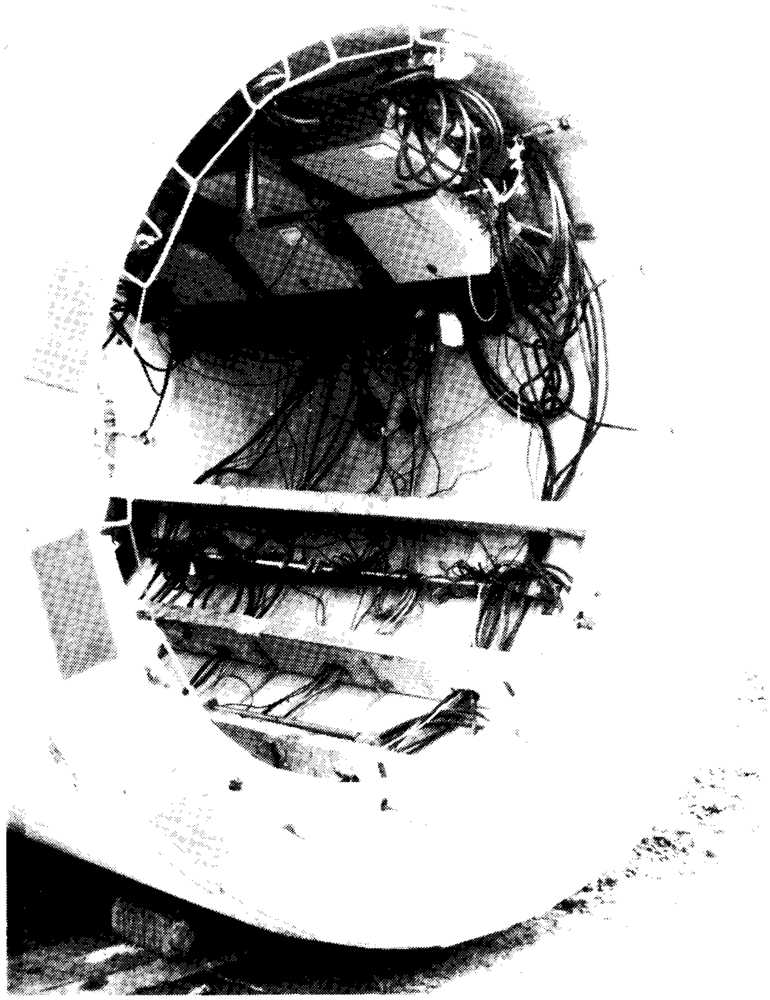


Plate 4.8 Instrumented pipe ready for lowering into the jacking pit.

4.3.2 Instrument protection

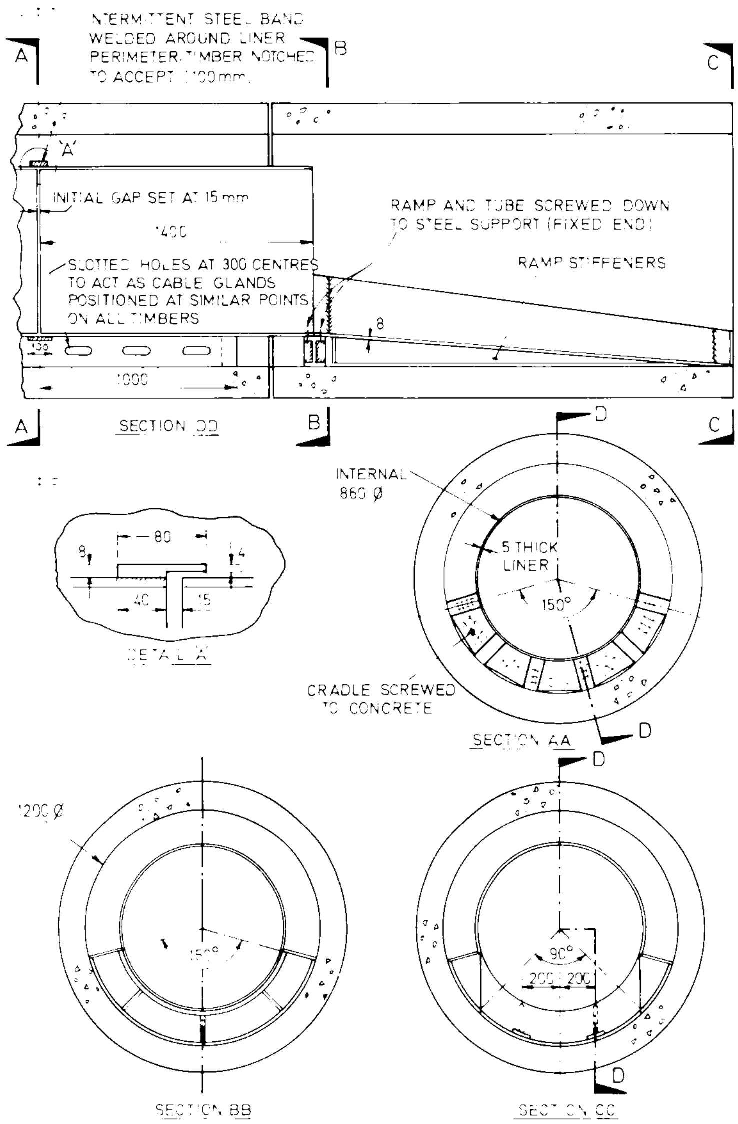


Figure 4.11 Protective liner details.

Protection of the instruments and data acquisition system against mechanical damage was provided by a steel liner which fitted inside the instrumented pipe and was detailed to avoid significant stiffening of its response, Figure 4.11 and Plate 4.9. The liner was typically 300mm smaller in diameter than the pipeline, (although different shapes were used to optimise the size of the constriction) and was fabricated in two 1.4m lengths, which was slightly longer than the 2.5m instrumented pipe length. Each overhanging portion was

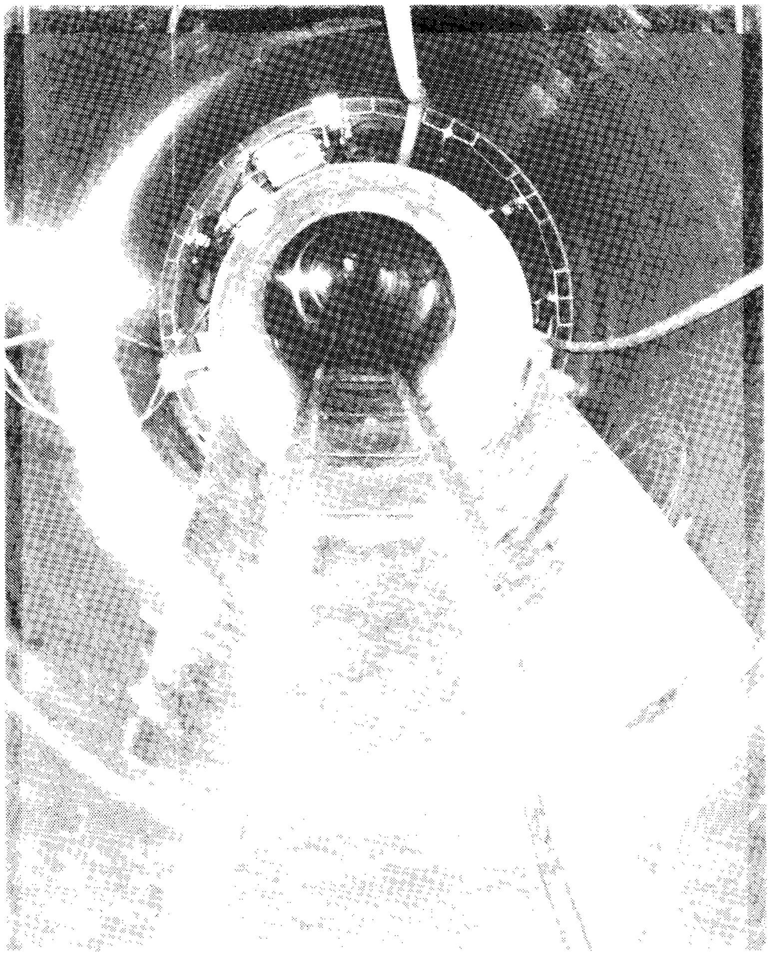


Plate 4.9 Protective liner in place.

bolted to a steel cradle fastened to the leading and trailing pipes. The liner was supported in the pipe by a timber cradle which was shaped to cover a 150 degree arc about the pipe invert and allow the instruments to be correctly positioned. Cross holes in the timber provided ducts for routing cables around the pipe circumference. Articulation of the liner was provided by a central steel banded joint which allowed the two halves to slide ±10mm relative to each other. A set of ramps in the leading and trailing pipes allow muck skips to travel through the liner. The timber cradle was fitted while the pipe was on the surface; the liner and ramps had to be fitted once the pipe was in the tunnel, causing up to 6 hours delay. This delay and the loss of production as a result of the constriction were priced by the contractor in the appropriate bill item. It has been found in practice that the protection it provided was well worth the slight reduction in productivity that it caused.

4.3.3 Instrument recovery

Removal of all the instruments took place at the end of the drive or, if before then, at a weekend. The jacking pit instruments were firstly unbolted from their anchorage points, then the liner was removed and the surface mounted extensometer, joint movement indicators and data collection boxes removed from the tunnel. The contact stress cells and pore pressure probes glued into the side walls of the pipe were recovered by overcoring and the resulting holes made good as required.

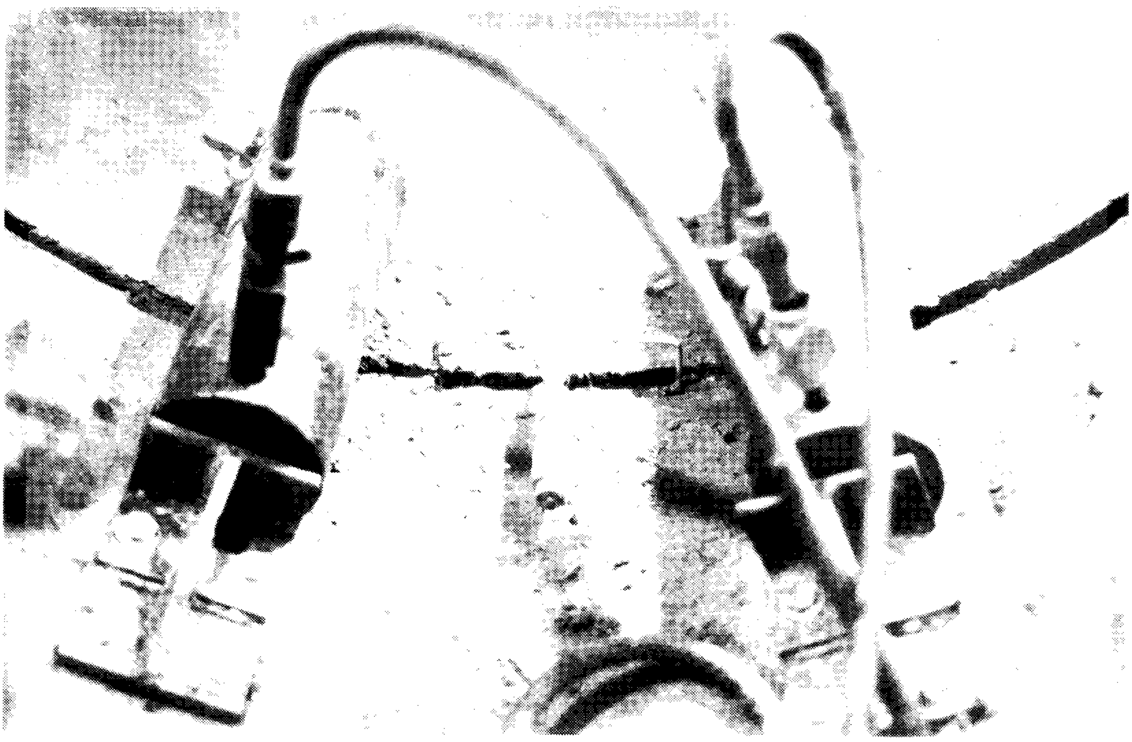


Plate 4.10 Removal of cells from pipe joints.

Removal of the pressure cells from the pipe joints required special anchorage brackets to be bolted to each side of the instrumented joint and cylinder jacks inserted to push the joint apart and allow removal of the cells and insertion of replacement packing material, Plate 4.10. This was only possible provided the instrumented pipe was near one end of the tunnel or an actual or dummy interjack so that the friction on only a few pipes had to be overcome. This could not be arranged in scheme three; here the cells were recovered by drilling and cutting out the packing material in the joint, extracting the cells and replacing the packer. This was successful, but was tedious and caused some superficial damage to the cells.

4.4 Data handling

Continuous logging of the jack load cells and the large cluster of instruments in the special pipe generated up to 20M bytes of data written to print files in ASCII format, which is compatible with Lotus 123 spread sheets. To ensure that excessive data were not handled, logging was performed on two different time bases. During pushes, typically of two minutes duration, data was logged every five seconds, while at all other times intervals of one minute were used. Regular backing up of data on floppy disk was carried out twice daily, with two copies being made and kept in separate locations at all times.

A 386 AT personal computer was used for processing. A limitation of the 123 software is that the ASCII files containing site data can only be 240 characters wide (16 channels of data) for importing into a spread sheet. Approximately 70 channels were used during the research and therefore the site data files had to be split into smaller manageable groups of instruments using a Fortran program. Although the data could be scaled and converted to engineering units at the time of capture, it was preferable to record only the voltage readings from the various instruments; the conversions being carried out in spread sheets at Oxford, minimising the possibility of introducing errors at source on site. Traces of each instrument response on a daily basis were produced. Figure 4.12 illustrates a typical set of responses from the bottom pore pressure probe and contact stress cell from scheme 1, obtained over a single working shift. The graphs clearly illustrate the magnitude of stress changes caused by jacking. These formed the starting point for detailed analysis of individual pushes and cross correlation between different instruments which was

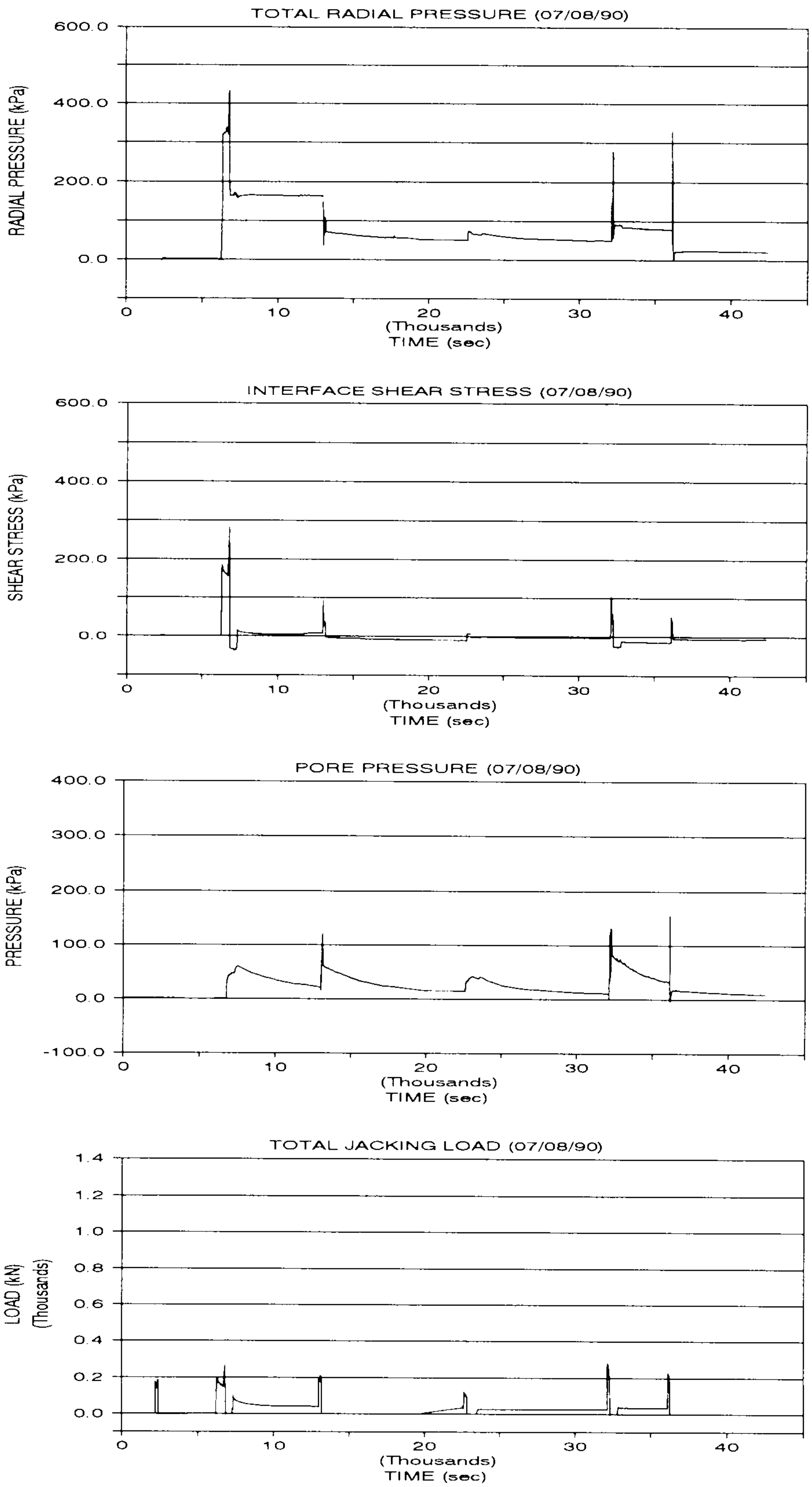


Figure 4.12 Typical set of interface stress records during a single shift.

generally carried out on a chainage basis. Back up copies of each stage of processing have been created and stored on floppy disk.

4.5 Performance of the instruments and site procedures

Once monitoring was in progress, good communication between the miners and the researcher ensured that there was very little disruption to the normal working pattern. The instrumentation was even of assistance to the contractor on certain schemes providing precise and continuous records of jacking loads which helped with decisions on whether or when to lubricate or use an interjack.

The careful selection and design of the instruments led to few in service failures, Table 4.6. The pipe joint pressure cells, jack load cells, pipe joint movement indicators, tube extensometers and data acquisition system demonstrated their fitness for purpose during the pilot test. The contact stress cells initially highlighted problems due to ground water ingress but this was overcome by redesigning the primary seals. The pore pressure probes were found to be susceptible to cable damage during extraction.

Only the Celesco unit and the ground convergence indicator performed poorly in the field. The Celesco unit was very prone to damage and required frequent repair, although it did provide useful data. The ground convergence indicator had some trouble with the ingress of fine particles past the PTFE seals causing jamming, but constraints of programming or ground conditions have in any case prevented this from being used after the first scheme.

The performance of the data acquisition system was extremely satisfactory. At no time was it necessary, during the five schemes, to stop the contractor as a result of a general systems failure, which would have necessitated removal of the protective liner to gain access to the individual boxes.

Schemes	Bolton (Scheme 1)		Newcastle (Scheme 2)		Honor Oak (Scheme 3)		Abbey Village (Scheme 4)		Cheltenham (Scheme 5)	
Instrument type	Number	Failures/Cause	Number	Failures/Cause	Number	Failures/Cause	Number	Failures/Cause	Number	Failures/Cause
Contact stress	4	2/Moisture ingress	2	1 Moisture	4	1/Overload 3 Glue line failures	4	-	5	Cables cut by coring contractor
Pore pressure	2	-	2	-	4	1 Cable severed 2 Glue line failures	4	1/Faulty repair cable	5	3/Faulty cable repairs
Pipe joint pressure	12	-	16	-	24	5 Cells crushed during extraction	24	-	16	-
Joint movements	3	-	6	-	6	-	6	1/LVDT cable cut	6	1 LVDT faulty
Tube extensometers	3	-	2	-	5	-	6	-	5	-
ERS gauges	3	3/Moisture	N/A	N/A	2	2/Moisture	N/A	N/A	N/A	N/A
Jack load cells	2	-	2	-	4	-	4	-	4	1/Cable cut
Temperature probes	4	-	5	1 Moisture	4	-	4	-	5	-
Ground convergence	1	1/Fine particles caused jamming	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Pipeline displacement transducer	N/A	N/A	1	1/Draw wire snapped	1	1/Draw wire snapped	1	-	1	1/Draw wire snapped
Data acquisition	Complete	-	Complete	-	Complete	1/Cabin network interface failure	Complete	1/Pit bottom box failure	Complete	-
Total No. channels	42		40		68		67		57	

Table 4.6 Field reliability

The success of the site procedures can readily be seen in Figure 4.13 which illustrates daily productivity in terms of metreage for each of the four hand drives.

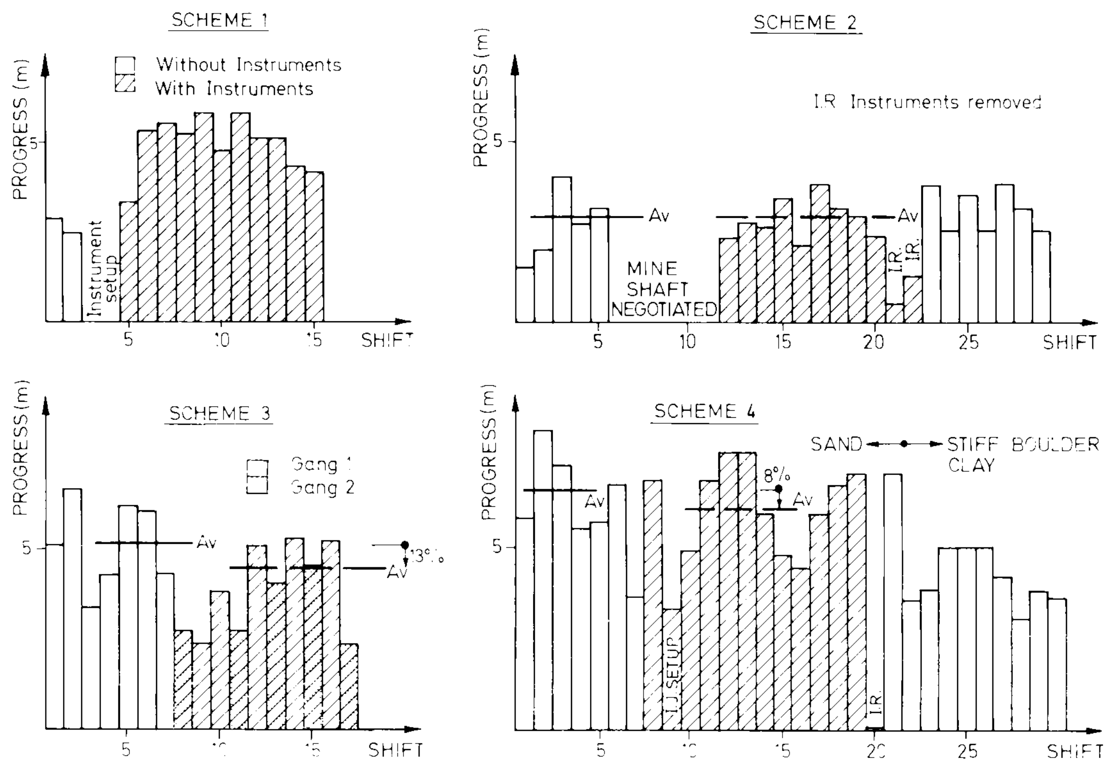


Figure 4.13 Effect of research on pipe jacking progress.

The reductions of 13% and 8% in average production for schemes 3 and 4 with instruments in place, are over-estimates since the effect of increased haulage distance must also be taken into account. Scheme 2 illustrates no change which is a result of the difficulty in excavating the face material. The data from scheme 5 is not included because a pumped bentonite spoil removal system was used and all surveys were performed outside normal working hours. Comparison of scheme 1 is not possible because the major part of the drive was completed with the instruments in place.

Problems outside the control of the researchers occurred during the early stages of the fieldwork. Scheme 1 started during some of the hottest Lancashire weather for decades and was followed by torrential rain which affected ground conditions to the depth of the pipe. An unexpected water main exacerbated the situation by temporarily flooding the pipeline. Scheme 2 took place during severe winter weather; the extremes of temperature were potentially damaging to instruments exposed on the surface and made correct application and curing of the glue to the in-wall instruments very difficult. An uncharted backfilled mine shaft further delayed production on this scheme.

By contrast, schemes 3 to 5 progressed surprisingly smoothly, although the rate of progress on the final scheme was affected by the need to carry out remedial work to pipe damage caused by localised crushing of pipe joints.

CHAPTER 5:
PIPE END LOAD TRANSFER

5.1 Introduction

The lack of clear guidance on allowable pipe end loads is reflected in the clause of the PJA's "guide to pipe jacking design" which has been one of the best safeguards to date: *"The permitted tolerances on line and level shall be within ±75mm of true line and ±50mm of true level at any point in the drive. Adjustments in line and level should be gradual and the pipe manufacturers permitted draw and deflection must never be exceeded at any joint".*

The main concern for contractors is what is the manufacturer's permitted draw or deflection, and what load will the pipe take when it occurs? Certainly the angular deflections quoted in BS5911 : Part 120 of 1° for pipes between 900 - 1200mm internal diameter and 1/2° for pipes between 1350mm and 1800mm should be treated circumspectly as they are minimum angles for watertightness.

This chapter concentrates on the measured tunnel alignments, the pressure distributions in the pipe joints, and the relations between them for schemes 1,3,4 and 5. Data have been selected from these sites for the following reasons:-

(i) Scheme 1 - shallow tunnel through firm clay, ground contact at the base of the pipe only, instrumented pipe near the front end where face loading conditions dominate;

(ii) Scheme 3 - deep tunnel in highly plastic clay, large contact pressures between pipe and ground;

(iii) Scheme 4 - tunnel in relatively stable bore through generally non-cohesive material, instrumented pipe inserted approximately at mid length;

(iv) Scheme 5 - lubricated machine drive in sand and gravel below the water table, relatively large deviations from line and level and some pipe damage under jacking loads.

Details of the instrumentation arrays in the rear joint of the special pipes are illustrated in Figures 5.1 and 5.2. The joint packing material in the instrumented joints was standardised as medium density fibreboard, to correlate with the laboratory experiments and assist interpretation of the joint cell readings.

5.2 Directional control of pipe jacks

5.2.1 Changes of pipeline alignment in practice

Excavation at the face of a pipe jack can deviate from the intended line and level. Constant corrections to the measured deviations induce the pipe string to take a zig-zag course, known as "wriggle", which results in deflections at pipe joints. Throughout the research, changes in pipeline alignment have been monitored by carrying out line and level surveys (usually daily) for the full length of the tunnel during the construction period. The resulting plots are presented in Figures 5.3 to 5.6. For clarity only three surveys are shown for each scheme. Schemes 1,3 and 4 would be considered well controlled drives while scheme 5 significantly exceeded specification. Pipe damage caused by localised crushing of the joint was experienced in a number of pipes at positions A and B of Figure 5.6.

Examination of the change in position of pipes reveals that misalignment of the pipelines did not significantly alter throughout construction. This is surprising since the restraining action can only be provided by ground reactions which were expected to be different in the various types of ground. Further information on this aspect of the work can be found in Chapter 6. In addition it would appear that the directional control was generally poorest at the start of each drive. This can have serious repercussions, particularly on long drives, because the worst misalignments tend to coincide with the most heavily loaded pipes. Care taken during the setting up of the thrust pit and in particular accurate and regular monitoring during the early phases of a pipe jack while personnel are familiarising themselves with the ground conditions should reduce this effect.

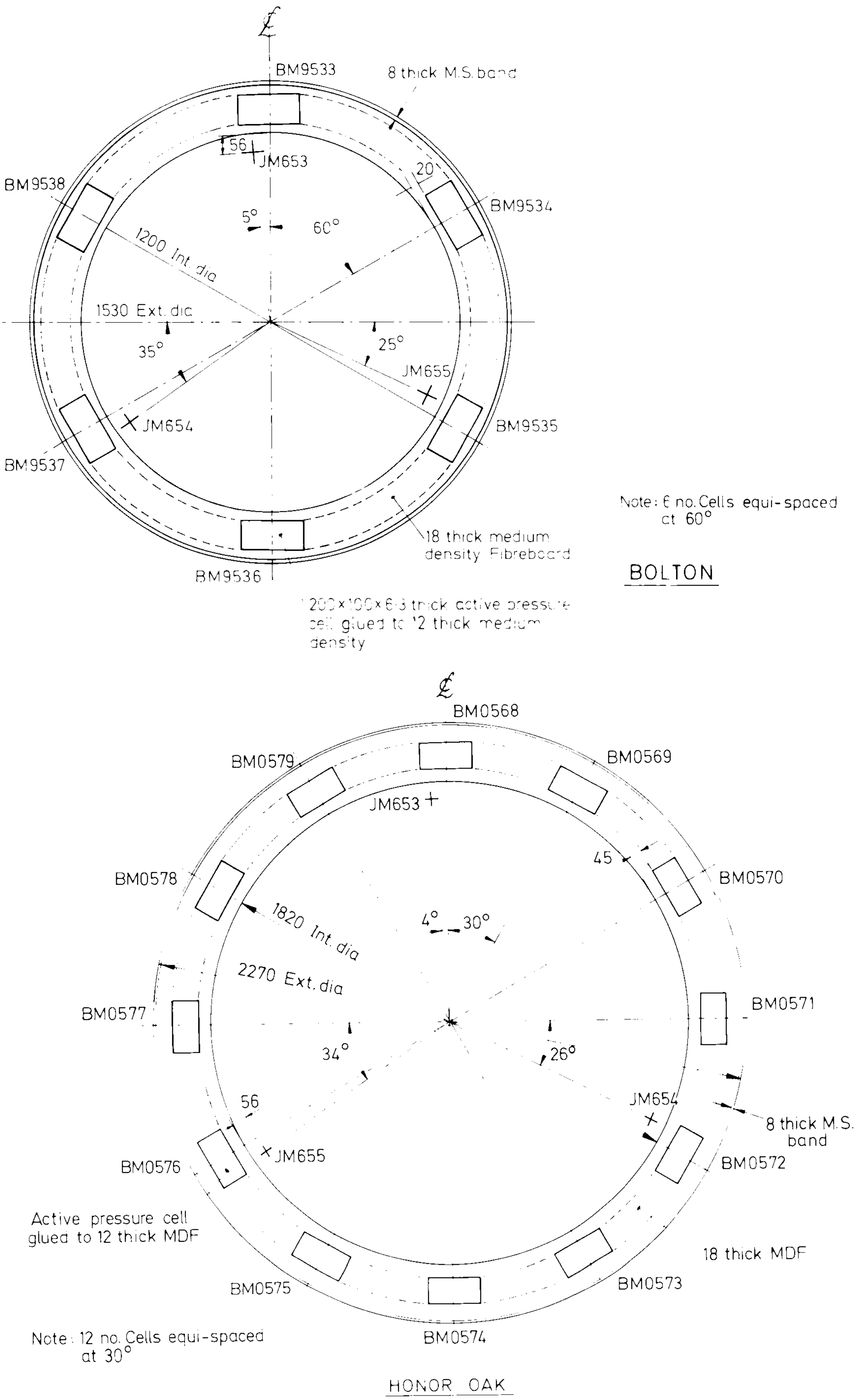
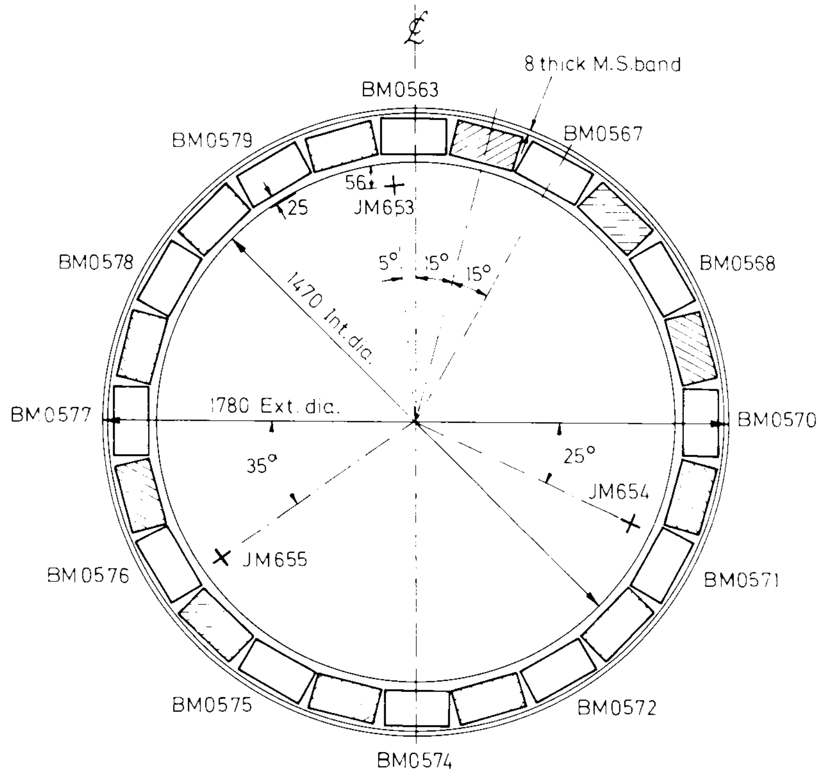
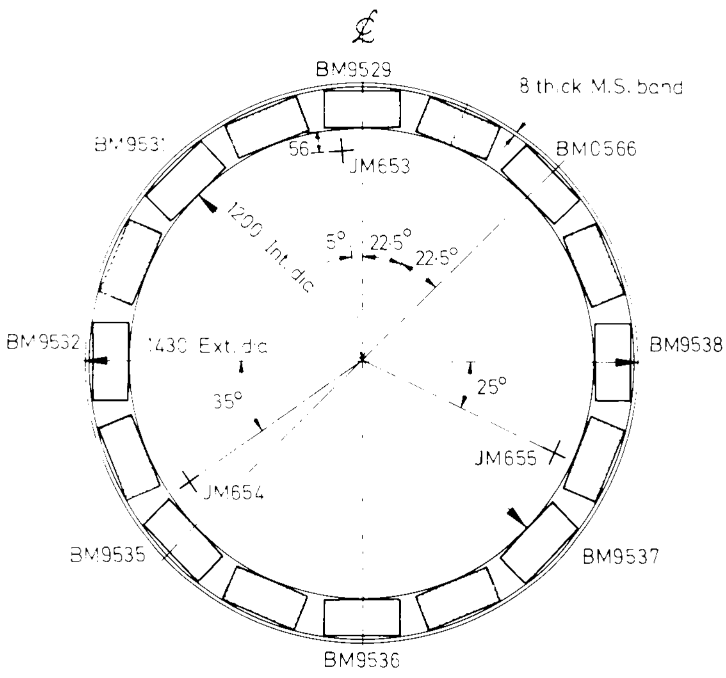


Figure 5.1 Schematic of the instrumented rear joint in schemes 1 and 3.



Note 12 No Active cells equi-spaced at 30° glued to 12 mm MDF
12 No Dummy cells offset at 15° glued to 12 mm MDF

ABBEY VILLAGE



Note: 8 no. Active cells equi-spaced at 45°
8 no. Dummy cells offset at 22·5°

Active pressure cell glued to 12 thick MDF
Dummy pressure cell glued to 12 thick MDF

CHELTENHAM

Figure 5.2 Schematic of the instrumented rear joint in schemes 4 and 5.

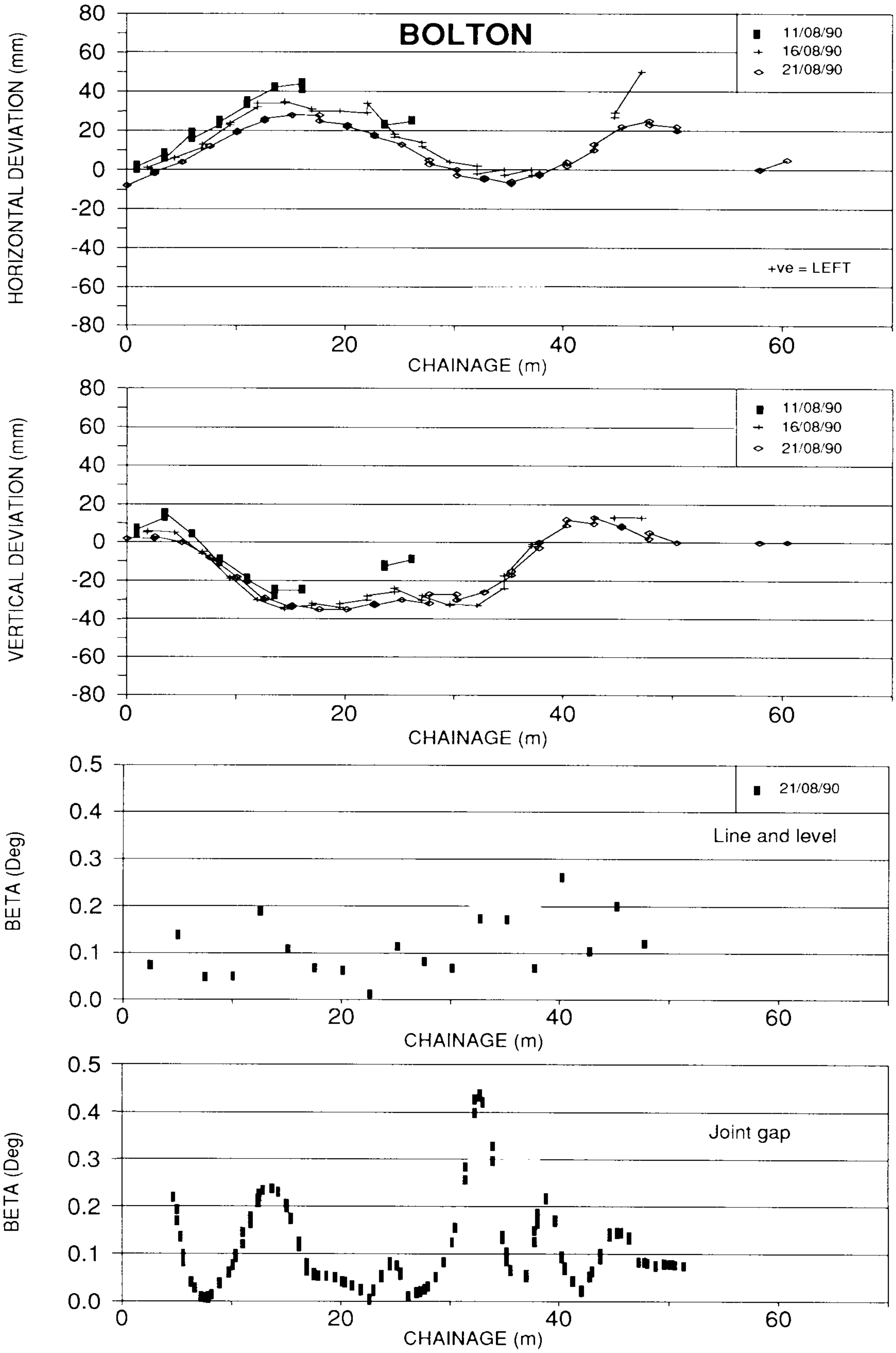


Figure 5.3 Tunnel alignment surveys for scheme 1.

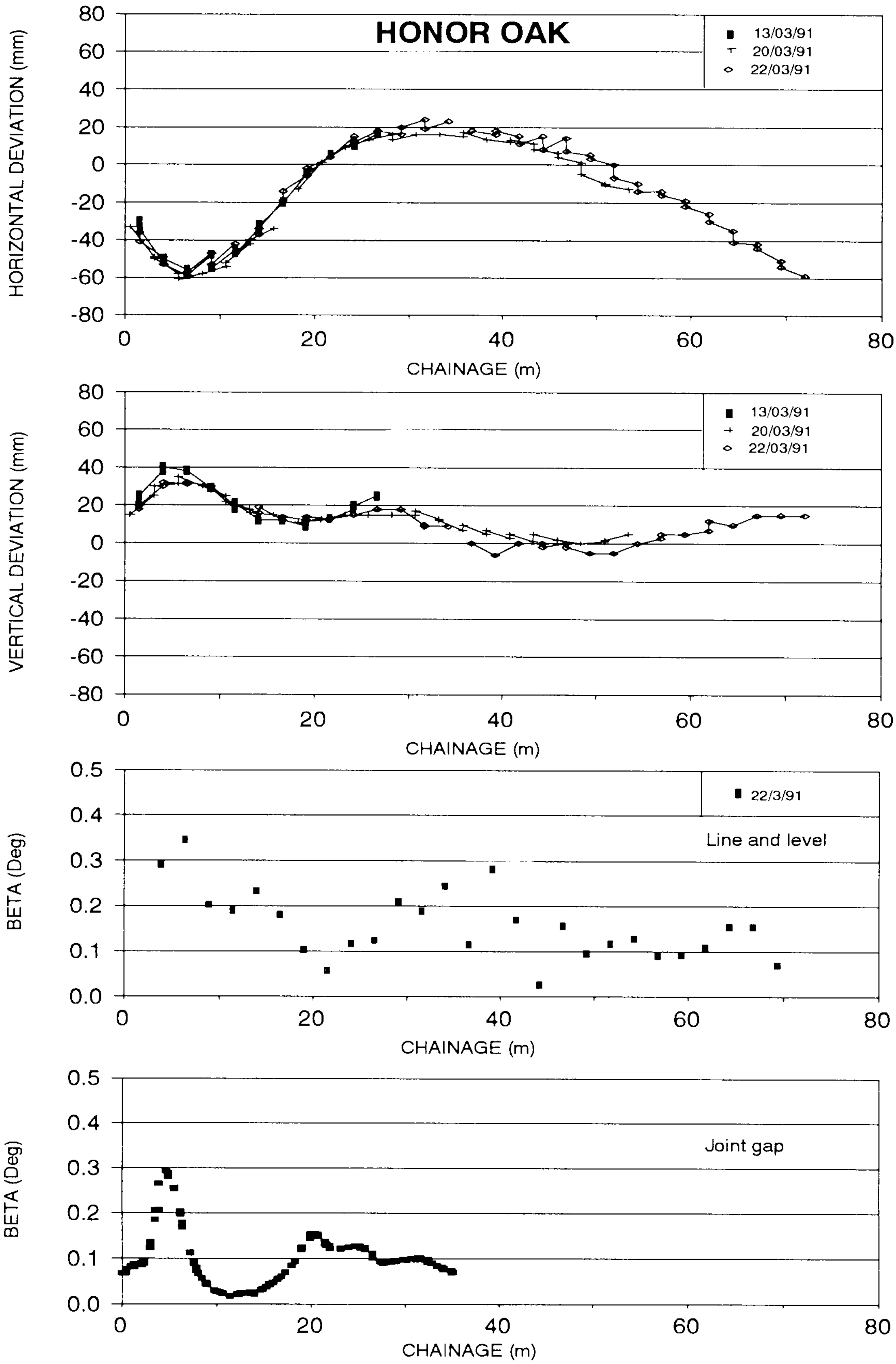


Figure 5.4 Tunnel alignment surveys for scheme 3.

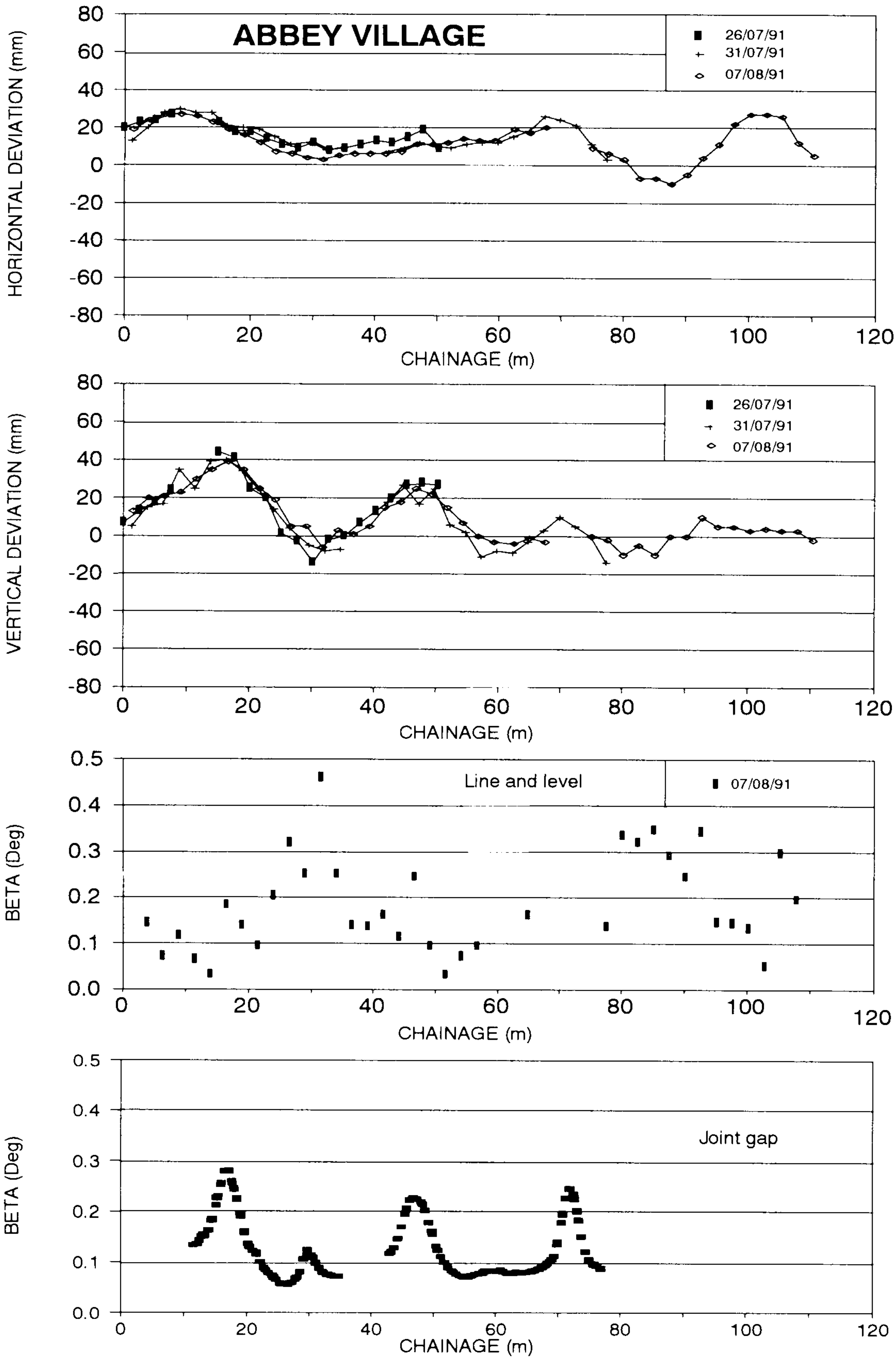


Figure 5.5 Tunnel alignment surveys for scheme 4.

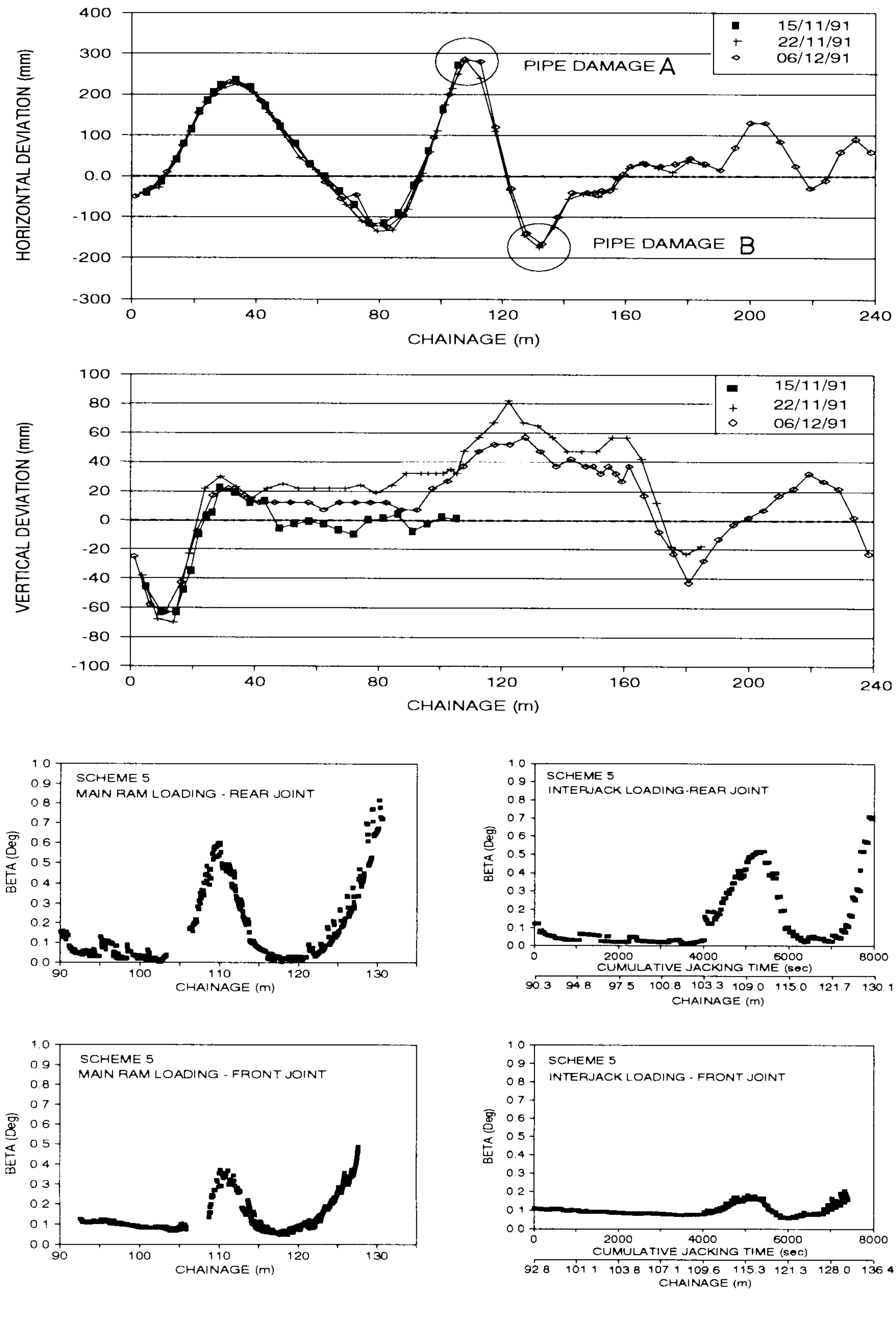


Figure 5.6 Tunnel alignment surveys for scheme 5.

An interesting feature of scheme 5 is the apparent fluctuation in vertical alignment between chainages 50m and 110m. The level of the tunnel rises by approximately 20mm between the surveys of 15 November and 22 November and then falls by 10mm as indicated by the survey of 6 December 1991. Comparison of the external diameter of the pipe and the outside diameter of the Unclemole indicate a 20mm difference. Site records prior to the 15 November show that lubrication was delayed until 25m of the drive was completed and that a rapid rate of pipeline advance was being achieved, typically 20m per shift, which slowed to 10m per shift during the period 15 - 22 November. It would appear that the initial delay in lubricating and the high rate of advance prevented a fully effective lubrication zone from being developed around the pipe. The subsequent reduction in the rate of advance and systematic lubrication of the pipeline resulted in successful stabilisation of the tunnel bore and pipeline buoyancy. As the machine moved away from Shaft 5, at chainage 160m, the main lubrication effort was shifted to the next section of the drive with what appears to be a slow dissipation of the lubricating fluid to the surrounding ground in the initial tunnel section.

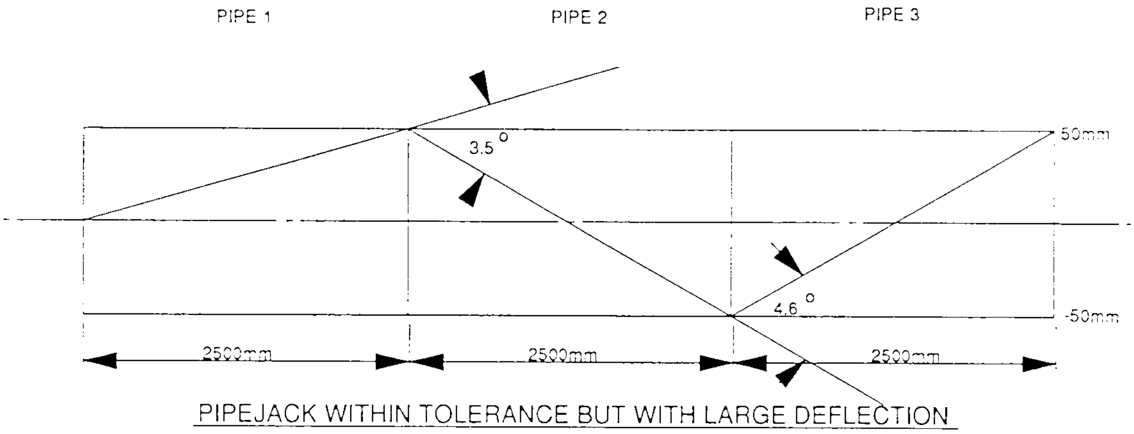


Figure 5.7 Possible deflection angles under typical line and level specification.

The importance of the directional control of a pipe jack is illustrated in Figure 5.7. Based on an allowable tolerance on line and level of ± 50mm, Hough (1986) shows how large deflections can become. Two 2.5m long pipes can have a maximum deflection angle of 4.6° and still be within line and level tolerance. Deflections of this magnitude considerably exceed the 1° angular deflection of the quality assurance control test of Table 6 in BS5911 Part 120 and would result in the pipe joint being incapable of performing its required functions. It is therefore clear that it is not sufficient to monitor only line and level if high loads are being applied but to assess the three dimensional orientation of pipes to enable full interpretation of misalignment angles. The joint angular deflection can readily be determined from line and level surveys as summarised in section 5.2.2.

5.2.2 Evaluation of joint angular deflection from line and level surveys

Assume two pipes have an angular deflection β at their common joint as illustrated in Figure 5.8. The spatial positions of the centres of each pipe cross section relative to the true alignment are represented by points A and C at the ends of each pipe and B at the centre of the joint between them. The coordinates of these points represent the horizontal and vertical offsets measured from the tunnel laser during the surveys and the chainage of the individual pipes.

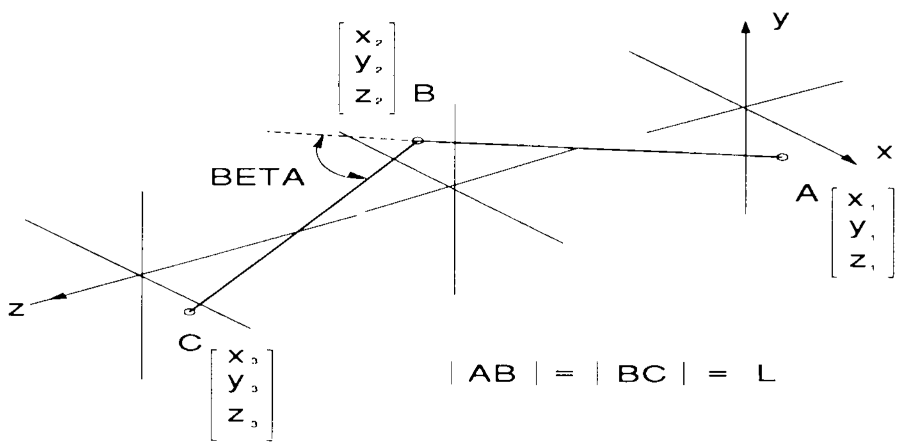


Figure 5.8 Coordinate system for determining β from line and level surveys.

Direction cosines of AB are:

$$l = \frac{x_2 - x_1}{L}, \quad m = \frac{y_2 - y_1}{L}, \quad n = \frac{z_2 - z_1}{L}$$

(i.e) $l^2 + m^2 + n^2 = 1$

Likewise direction cosines of BC are:

$$l' = \frac{x_3 - x_2}{L}, \quad m' = \frac{y_3 - y_2}{L}, \quad n' = \frac{z_3 - z_2}{L}$$

By definition the misalignment angle β is given by:

$$\text{Cos}\,\beta = ll' + mm' + nn'$$

Plots of angular deflection derived from the line and level surveys on schemes 1, 3 and 4 are included in Figures 5.3 to 5.5. Values of between zero and 0.3 ° were typically measured on the three drives. The apparent scatter in the values is a result of the sensitivity of the model to errors in the measured differences between successive pipes; these can result from manufacturing tolerances on pipe diameter, shear displacements at joints or more importantly because of the inherent difficulty of measuring small changes in line and level off a tunnel laser beam. The angles for scheme 5 could not be determined because it was necessary to reduce the tunnel survey time by recording the position of every second pipe.

5.2.3 Evaluation of angular deflection and centre of compression from joint gap monitoring

The previous assessment of pipe deflection is generally only applicable to unloaded or partially relaxed pipelines since most surveys are carried out during break times. To obtain an assessment of angular deflection when pipes are being jacked it is necessary to automatically monitor the changes in joint gap. This is performed using the three equi-spaced joint movement indicators positioned at each of the instrumented pipe joints, Figures 5.1 and 5.2. A deflected joint may be described in terms of the notation and sign convention of Figure 5.9.

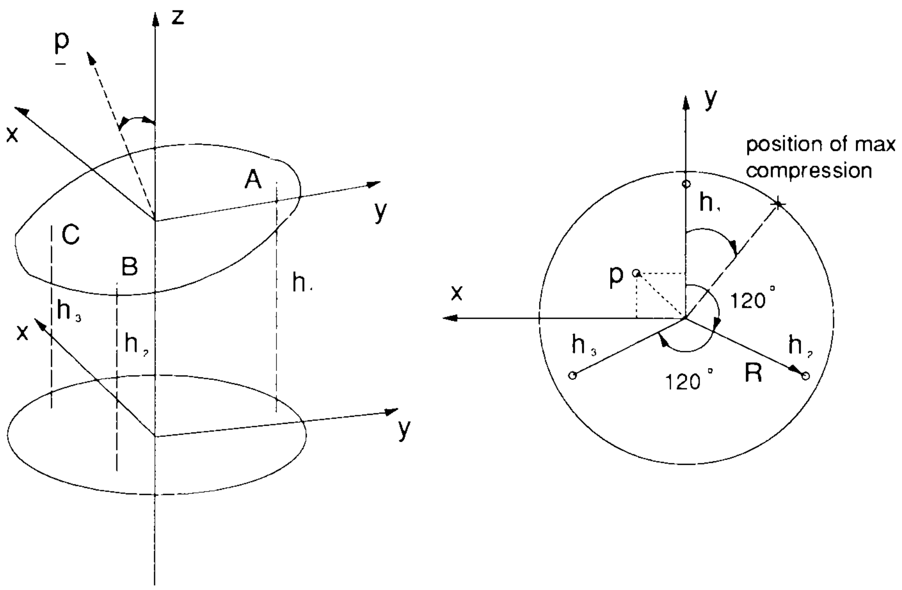


Figure 5.9 Notation for deflected instrumented joint.

Coordinates of points A, B and C are:

$$\begin{bmatrix} 0 \\ R \\ h_1 \end{bmatrix}, \quad \begin{bmatrix} \dfrac{-\sqrt{3}R}{2} \\ -\dfrac{R}{2} \\ h_2 \end{bmatrix} \quad \text{and} \quad \begin{bmatrix} \dfrac{\sqrt{3}R}{2} \\ -\dfrac{R}{2} \\ h_3 \end{bmatrix} \quad \text{respectively}$$

It follows that:

$$\text{Vector AB} = \begin{bmatrix} \dfrac{-\sqrt{3}R}{2} \\ \dfrac{-3R}{2} \\ (h_2-h_1) \end{bmatrix}; \ \text{BC} = \begin{bmatrix} \sqrt{3}R \\ 0 \\ (h_3-h_2) \end{bmatrix}; \ \text{CA} = \begin{bmatrix} \dfrac{-\sqrt{3}R}{2} \\ \dfrac{3R}{2} \\ (h_1-h_3) \end{bmatrix}$$

If vector p = $\begin{bmatrix} p_1 \\ p_2 \\ p_3 \end{bmatrix}$

Then for p to be normal to AB:

$$\begin{bmatrix} p_1 \\ p_2 \\ p_3 \end{bmatrix} \quad \begin{bmatrix} \dfrac{-\sqrt{3}R}{2} \\ \dfrac{-3R}{2} \\ (h_2 - h_1) \end{bmatrix} = 0$$

Similarly for BC and CA.

Therefore for p to be normal to the plane:

$$\begin{bmatrix} \dfrac{-\sqrt{3}R}{2} & \dfrac{-3R}{2} & (h_2 - h_1) \\ \sqrt{3}R & 0 & (h_3 - h_2) \\ \dfrac{-\sqrt{3}R}{2} & \dfrac{3R}{2} & (h_1 - h_3) \end{bmatrix} \begin{bmatrix} p_1 \\ p_2 \\ p_3 \end{bmatrix} = 0 \qquad \begin{matrix} (i) \\ (ii) \\ (iii) \end{matrix}$$

From (ii) $\quad \dfrac{p_1}{p_3} = \dfrac{(h_2 - h_3)}{\sqrt{3}R}$

and from (i) and (iii) $\quad \dfrac{p_2}{p_3} = \dfrac{(h_2 + h_3 - 2h_1)}{3R}$

The direction cosines of p are:

$\dfrac{p_1}{L}, \dfrac{p_2}{L}, \dfrac{p_3}{L}$ where $L = \sqrt{p_1^2 + p_2^2 + p_3^2}$

Therefore $\quad \dfrac{L}{p_3} = \sqrt{\left(\dfrac{p_1}{p_3}\right)^2 + \left(\dfrac{p_2}{p_3}\right)^2 + 1} = \dfrac{1}{\cos\beta}$

$$\beta = \cos^{-1}\left[\frac{1}{\sqrt{\dfrac{(h_2 - h_3)^2}{3R^2} + \dfrac{(h_2 + h_3 - 2h_1)^2}{9R^2} + 1}}\right]$$

Since β is very small, α to a close approximation is given by:

$$\tan\alpha = -\frac{p_1}{p_2}$$ where α is the radial angle to the point of maximum compression

$$\alpha = \tan^{-1}\left\{\frac{\sqrt{3}(h_3 - h_2)}{(h_2 + h_3 - 2h_1)}\right\}$$

Changes in pipe alignment, β, at the rear instrumented joint are presented for schemes 1, 3, 4 and part of 5 in Figures 5.3 to 5.6. Values are recorded at the start and end of each push. It is apparent that the variation in β for scheme 1 is greater than for schemes 3 and 4; all three schemes satisfying line and level specification. This was anticipated because the instrumented joint in scheme 1 was close to the shield and therefore not only influenced by variations in line and level but corrective action by the miners. The position of maximum angular misalignment, 0.3 °, in scheme 3 shows excellent agreement with the position of rapid change in line and level. It also clearly demonstrates the advantage of gradual corrections to deviations which result in typical misalignment angles of 0.1° for the remainder of the drive. The alignment for scheme 4 was exceptionally good in terms of line and level, and probably reflects the optimum performance that can be achieved on a hand drive. However the changes in direction are too abrupt causing misalignment angles as large as 0.29 °, which demonstrates that uncritical adherence to specifications based on absolute line and level can be counter-productive. The section of tunnel subject to pipe damage, between chainages 90m and 135m in scheme 5, illustrated values of β as large as 0.82 °.

Automatic monitoring of joint gaps at both ends of the pipe provided the opportunity to check the repeatability of misalignment angles at any given chainage. The relationship for scheme 3 is typical of all the hand drives, Figure 5.10, illustrating good agreement for peak values and overall profile but some divergence at smaller angles. For scheme 5 the instrumented pipe was positioned two half pipes behind the first interjack. The plots of β, Figure 5.6, show that the rear joint is unaffected by the close proximity of the interjack loading; alignment induced deviations dictate joint behaviour. By contrast the front joint illustrates much smaller deflection angles. The interjack may provide additional degrees of freedom which combined with the increased flexibility of the half pipes allows significant re-orientation of the joint.

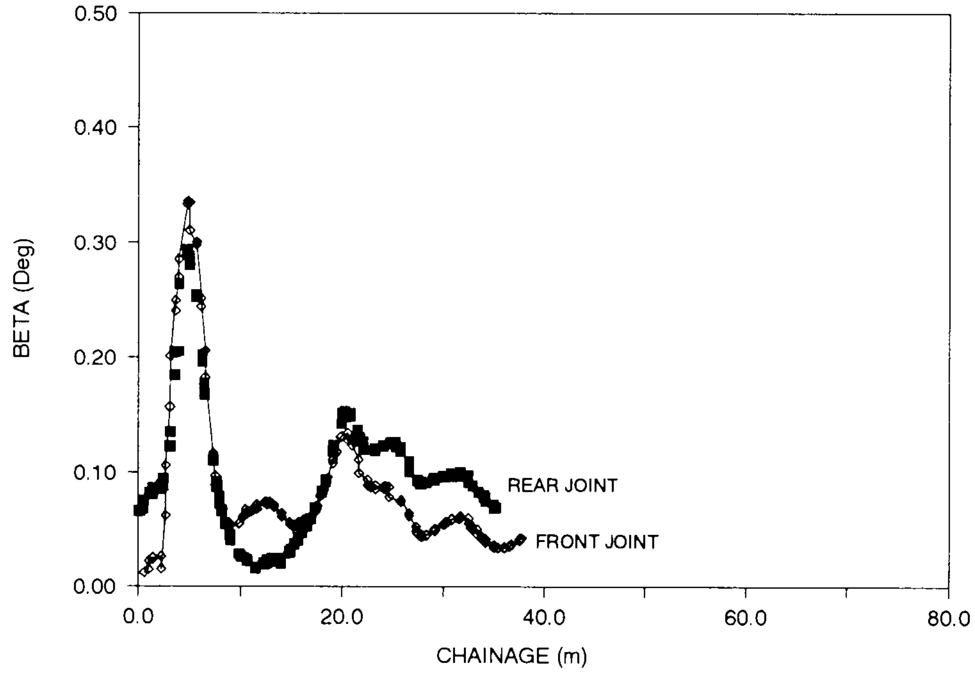


Figure 5.10 Comparison of the angular deviation in the front and rear instrumented joints at the same position throughout the Honor Oak drive.

The joint gap monitoring also enabled changes in β to be established between the unloaded tunnel state and when full jacking load was applied. Responses from schemes 1, 3, 4 and 5 are included in Figures 5.11 and 5.12. Small reductions, typically 0.02 °, were illustrated in the unlubricated hand drives suggesting the pipelines try to straighten under load but are restrained by ground reactions. It would appear that normal tunnel surveys, which are usually undertaken during breaks of work when the pipe string is unloaded, are sufficient to establish pipe misalignments for site control purposes. In the fully lubricated drive the pipe buoyancy allows larger reductions in joint angle, typically 0.08 °. Close examination of the end stress distributions in scheme 5, section 5.3, shows pipe damage occurred at angles between 0.5 ° and 0.6 ° when combined with the maximum jacking loads.

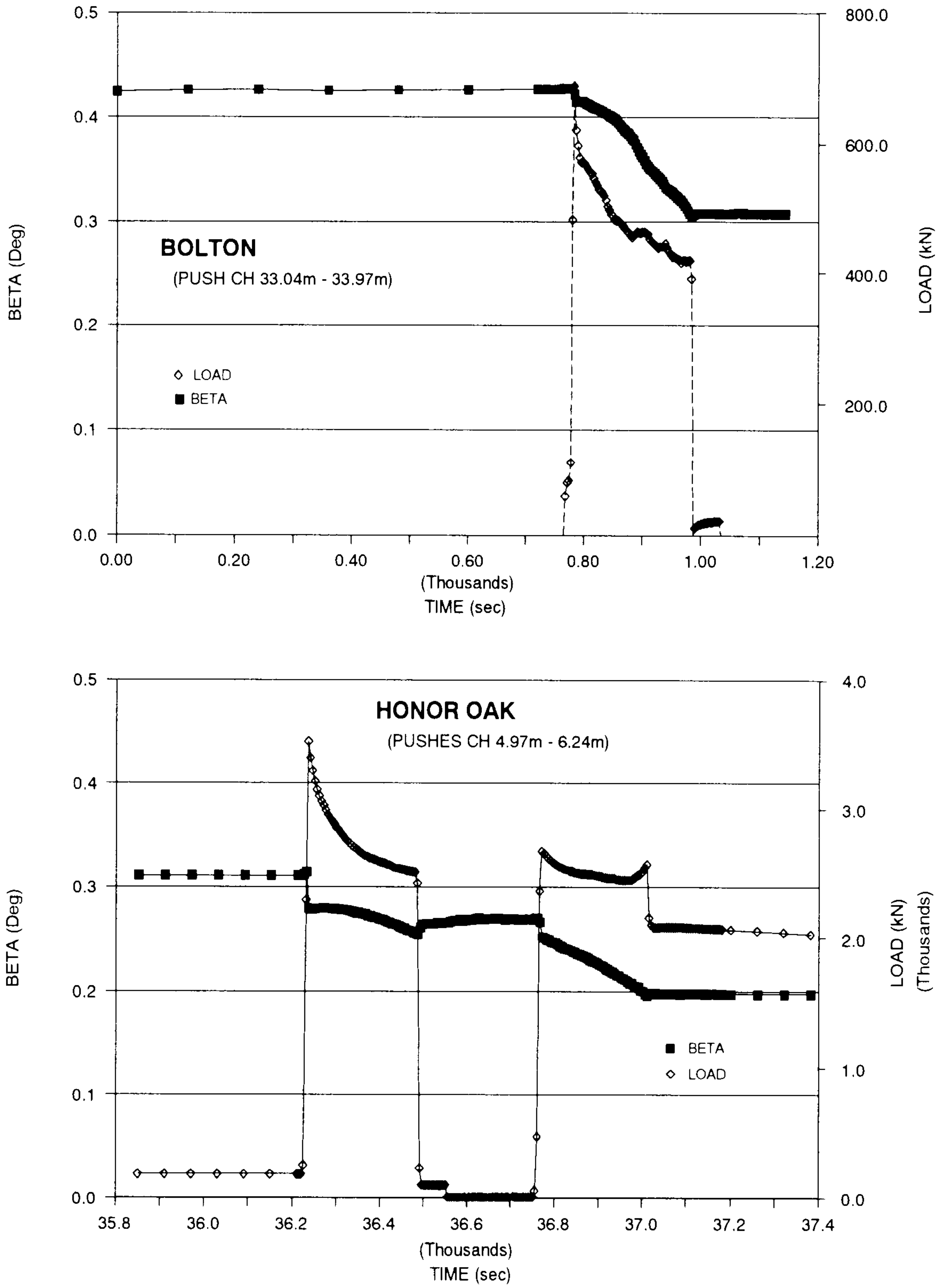


Figure 5.11 Variation in Beta due to application of jacking load; schemes 1 and 3.

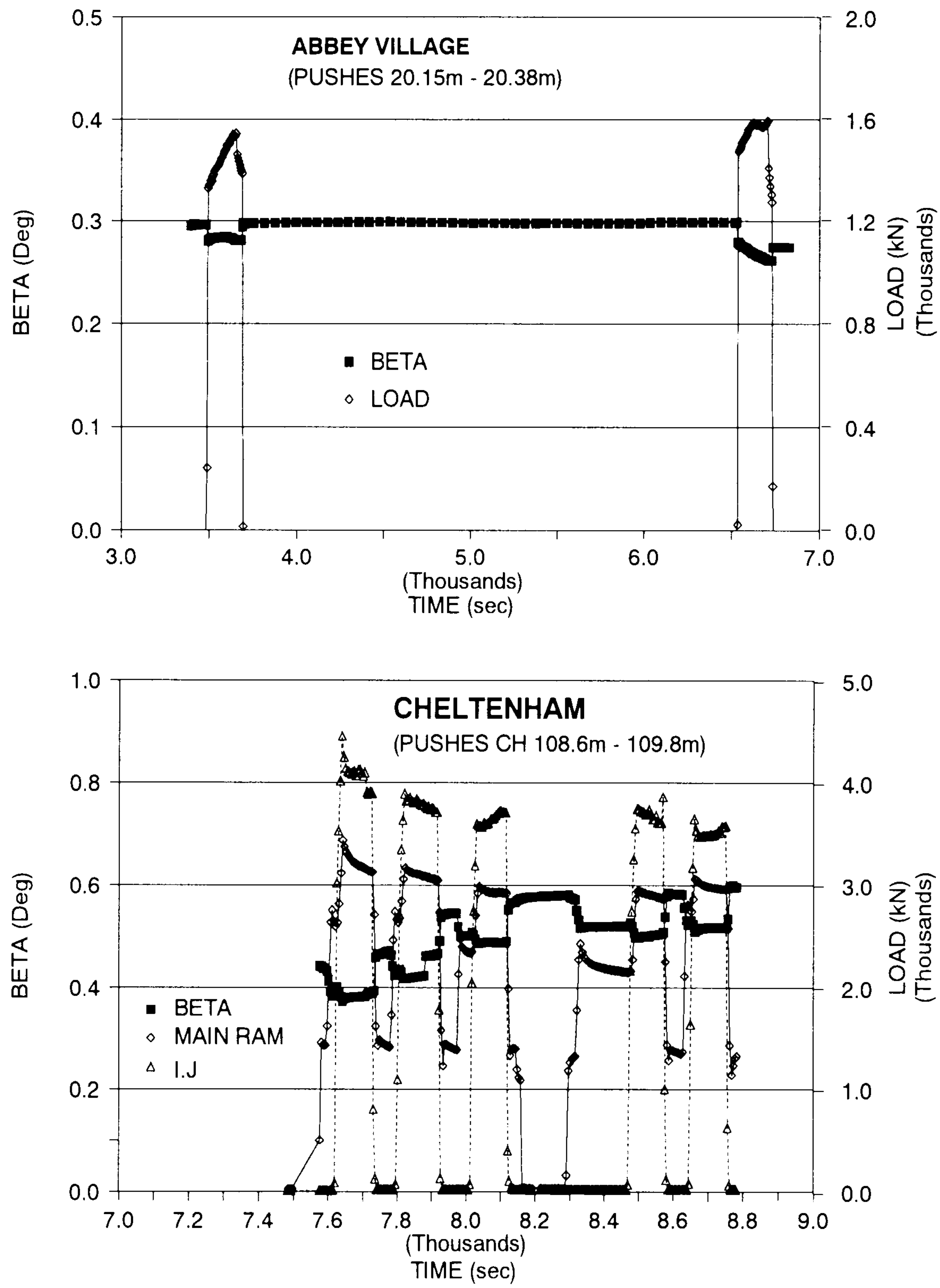


Figure 5.12 Variation in Beta due to application of jacking load; schemes 4 and 5.

5.2.4 Pipe end squareness audit

Additional angular deviations can occur at joints due to a lack of pipe end squareness, and in theory these need to be offset against the permissible misalignment angles. To evaluate the routinely obtained limits for this effect a number of pipes were measured at the various pipe manufacturer's works, covering the range of sizes used during the research. The results are detailed in Table 5.1.

Supplier	Spigot (β_{es})° A	Spigot (β_{es})° B	Socket (β_{em})° A	Socket (β_{em})° B	Maximum angle from BS5911 Part 120:1989	Pipe diameter (mm)
ARC	0.02	0.00	0.07	0.00	0.15	1200
	0.03	0.08	0.03	0.03	"	1200
(Spun process)	0.03	0.08	0.08	0.08	"	1200
	0.05	0.03	0.05	0.03	"	1200
Method of	0.02	0.02	0.02	0.02	"	1200
measurement CEN	0.08	0.03	0.03	0.03	"	1200
	0.05	0.09	0.00	0.09	0.14	1350
	0.02	0.05	0.02	0.05	"	1350
	0.00	0.02	0.00	0.02	"	1350
	0.03	0.02	0.07	0.02	"	1350
	0.04	0.05	0.04	0.05	"	1350
	0.00	0.02	0.00	0.02	"	1350
Spun Concrete	0.02	0.02	0.02	0.05	0.15	1200
	0.07	0.05	0.04	-	"	1200
(Spun process)	0.03	0.03	0.03	0.03	"	1200
Method of	0.07	0.07	0.07	0.06	0.13	1470
measurement CEN	0.08	0.04	0.03	0.00	"	1470
	0.07	0.04	0.07	0.04	"	1470
	0.05	0.01	0.05	0.05	0.15	1800
	0.04	0.02	-	0.02	"	1800
	0.01	0.01	0.05	0.01	"	1800
CV Buchan	0.08			-	0.15	1800
		0.09		-	"	1800
(Vertically cast)		0.09		-	"	1800
		0.06		-	"	1800
Method of measurement plumb line on roller line		0.09		-	0.14	1820
		0.08		-	0.14	1820

Notes 1. A & B refer to two planes at 90° to each other
2. - No reading taken

Table 5.1 End squareness audit.

The values demonstrate that pipes made by British manufacturers are well within tolerance for end squareness. It will be noted that an infinite combination of angles can occur when two pipes are joined and that the exact values were not determined at the instrumented joints. Although the worst combinations of angles typically produce values around 0.1°, it was observed in practice that the angular deviations correlate well with the alignment profiles suggesting that the end squareness effect is sufficiently small for it to be discounted in the field tests.

5.3 Load transfer at joints

The field work allows joint deflection angles to be correlated against joint packer compression and load distribution. Results at a selection of angular misalignments are presented in Figures 5.13 to 5.15. The plots have been grouped according to the observed stability of the tunnel bores. Schemes 1 and 4 were principally stable, scheme 3 was subject to large horizontal ground stresses and scheme 5 illustrated pipeline buoyancy. To explain the behaviour of the various examples it is necessary to consider the characteristics of the packing material used in the joints. Figure 5.16 presents the results of a series of uniaxial compression tests on dry Medium Density Fibreboard (MDF) which has been used throughout the field work. The choice of packing material was based on the recommendations of Milligan and Ripley (1989). Samples of 18mm thick MDF and the MDF-pressure cell composite have been subjected to twenty cycles of applied stress at 10, 20, 30, 50, 30, 10 N/mm^2 intensities. The plots show the responses on cycles 1, 5, 10 and 20. Examination of the results reveal several points of interest: the packer is permanently deformed as soon as it is loaded and unloaded once; after the first cycle the material illustrates little change in its stress/strain characteristics or permanent deformation unless the stress magnitude is increased; subsequent cycling at reduced stress levels exhibits a similar response to that at the maximum intensity. It is therefore important, although generally impracticable, when modelling the stress distribution in a joint to consider the previous stress history.

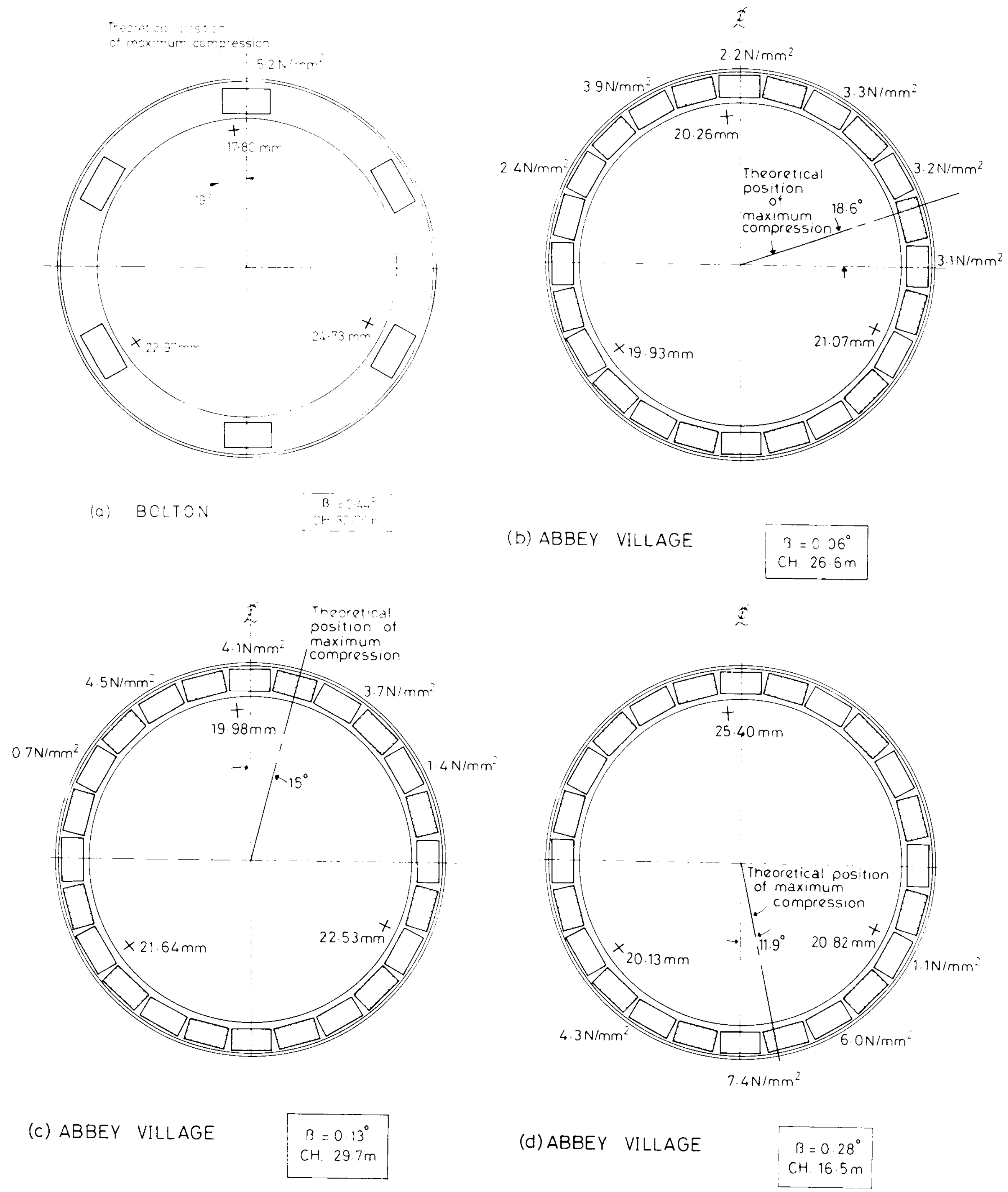


Figure 5.13 Relationship between measured joint angle and pressure distribution; schemes 1 and 4.

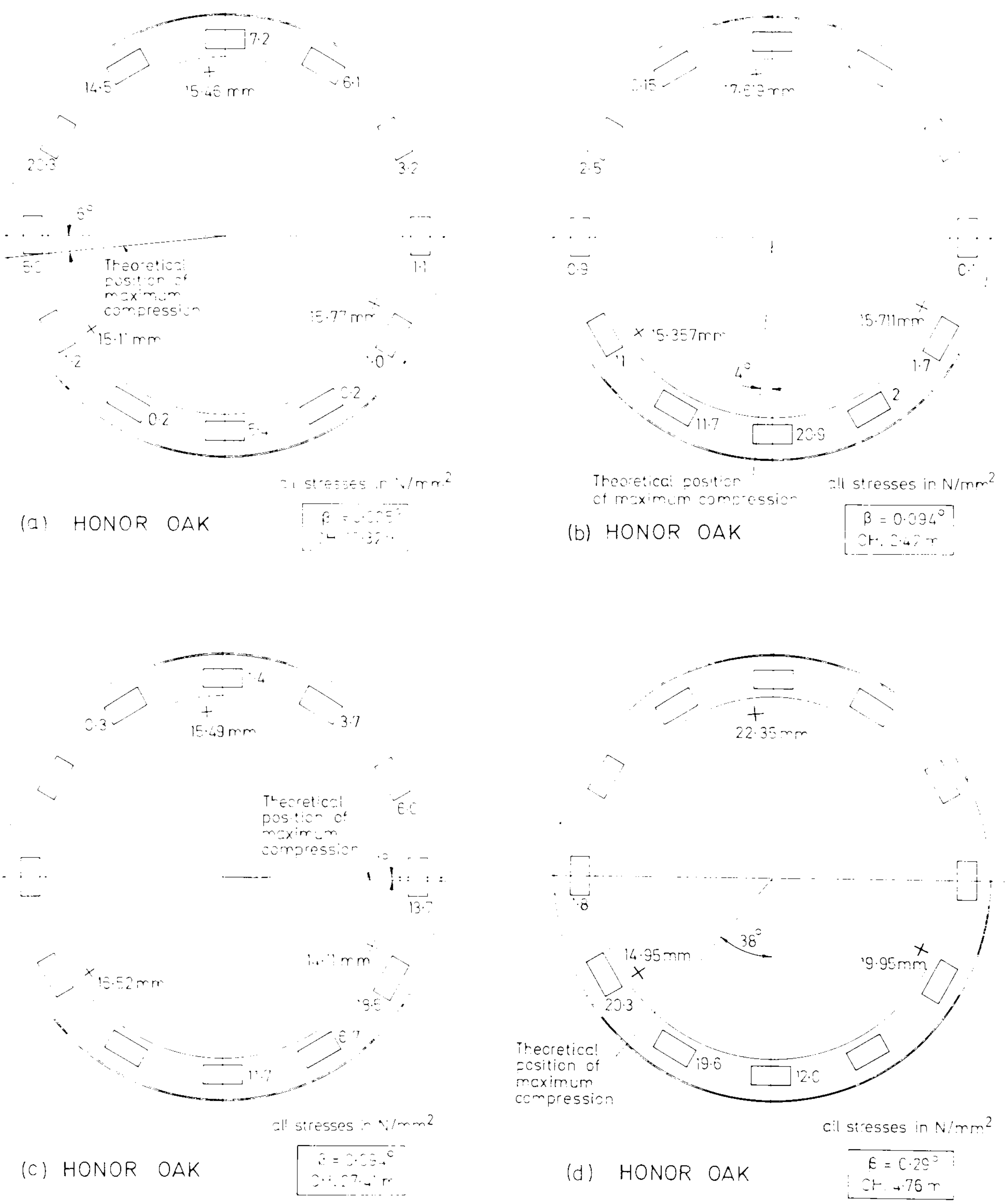


Figure 5.14 Relationship between measured joint angle and pressure distribution; scheme 3.

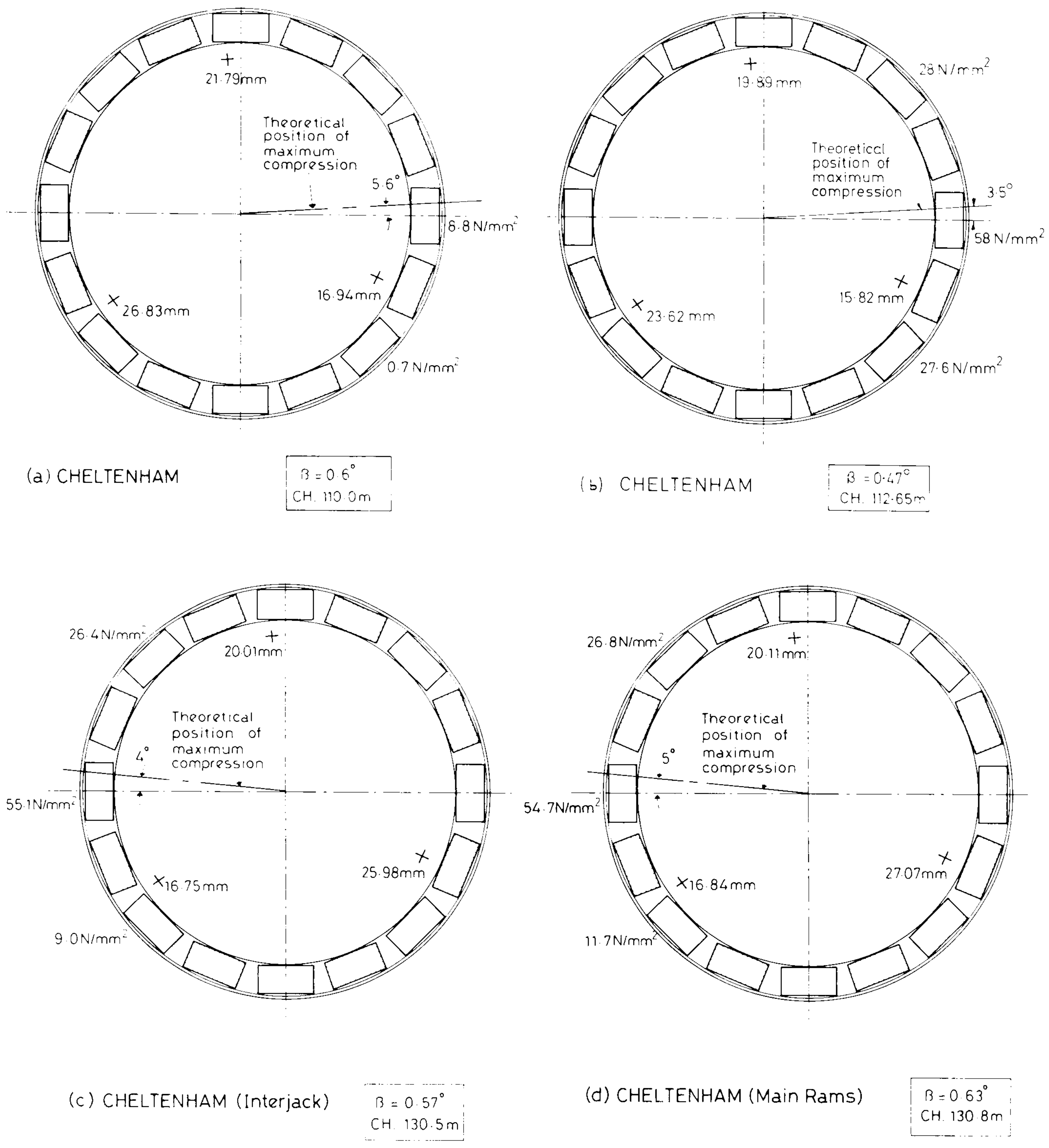


Figure 5.15 Relationship between measured joint angle and pressure distribution; scheme 5.

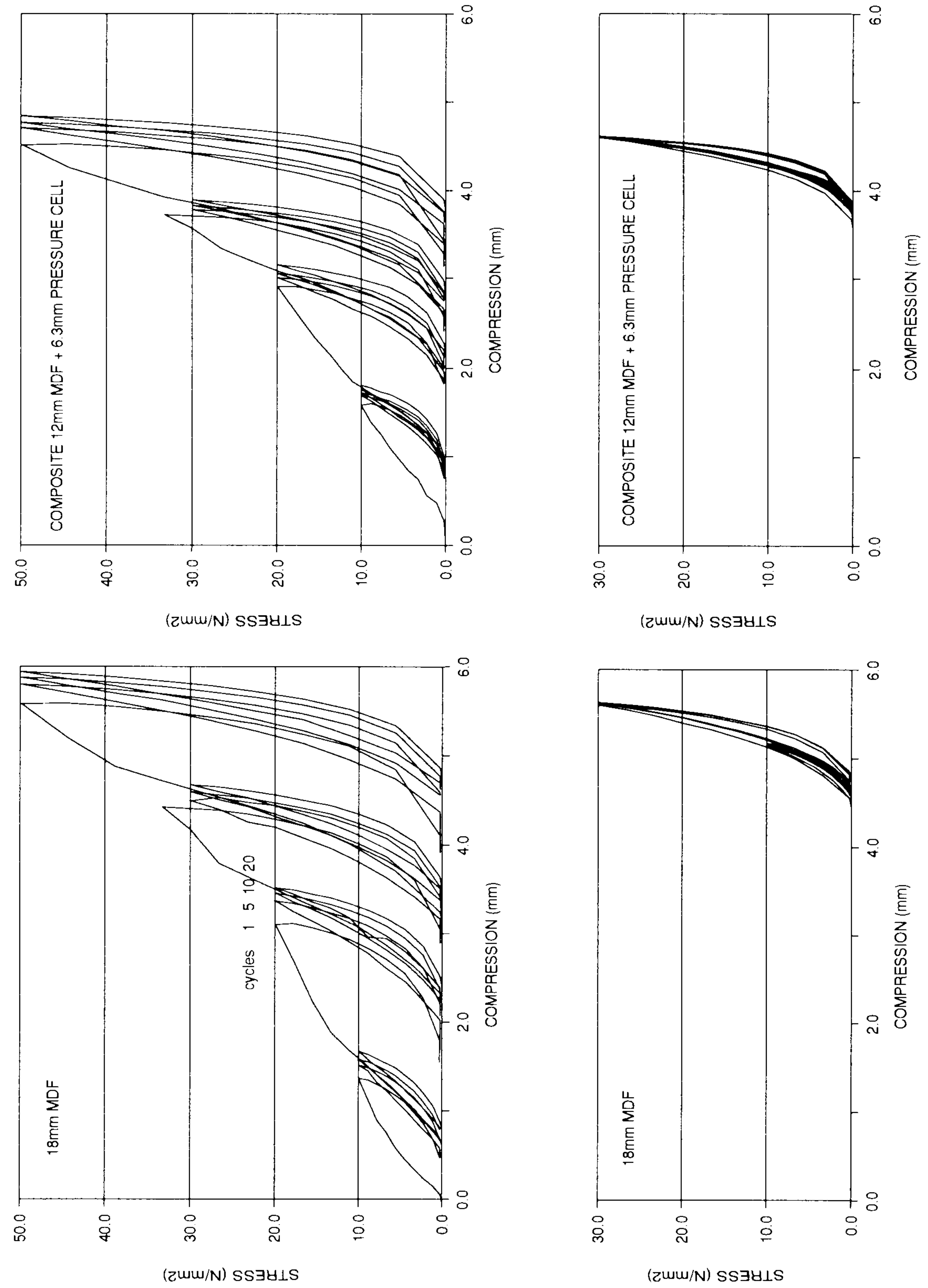


Figure 5.16 Uniaxial stiffness tests on joint packer material.

The plots of Figures 5.13a, d, 5.14d and 5.15b, c, d are taken at maximum angular deviations and illustrate close agreement between the measured stress distributions and the positions of maximum compression derived from the joint movement indicators. This is because the previous stress history of the packer has negligible effect when the packer is undergoing maximum compression. Localisation of stress is clearly demonstrated even with moderate deflection angles of 0.1°. At smaller misalignments, typically 0.025°, Figure 5.14a, the relationship between angular measurement and stress distribution tends to break down because the permanent compaction of the packer material results in redistribution of load. From the analysis of joint gaps the maximum compression is expected to occur at 6° below the left springing position. Inspection of the previous stress history of the joint, Figure 5.17, and consideration of the stresses imposed by the thrust ring (not shown in the Figure), indicate that the lower 5 pressure cells have registered large stresses prior to chainage 10.62m resulting in the major part of the load being transferred through the top left hand quadrant. It will be noted that the bottom pressure pad in scheme 3 indicates a larger compressive stress than would be expected from the general distributions. This may be the result of spoil being trapped in the bottom of the joint. In general stress levels in schemes 1 to 4 were smaller than 20N/mm^2 and did not precipitate pipe damage.

On scheme 5 the maximum angular deviation, Figure 5.15a, occurred under partial load conditions (i.e. closing the interjack). Although the localisation of stress was severe the stress levels were small. At larger jacking loads, (i.e. interjack engaged), localised crushing of a number of joints was observed at chainages 110m and 130m. The damage was sustained in standard joints fitted with 80mm wide by 18mm thick plywood packers. Stresses in the instrumented joints, containing 100mm wide by 12mm thick MDF to satisfy the requirements of the research, peaked at 52 to 58N/mm^2 and did not inflict damage. Comparison of the theoretical maximum jacking loads for the different joint configurations using the design approach of section 5.4 with $\beta = 0.47°$ suggests that the MDF joint should carry 15% more load than the plywood which is probably the reason the instrumented pipe survived.

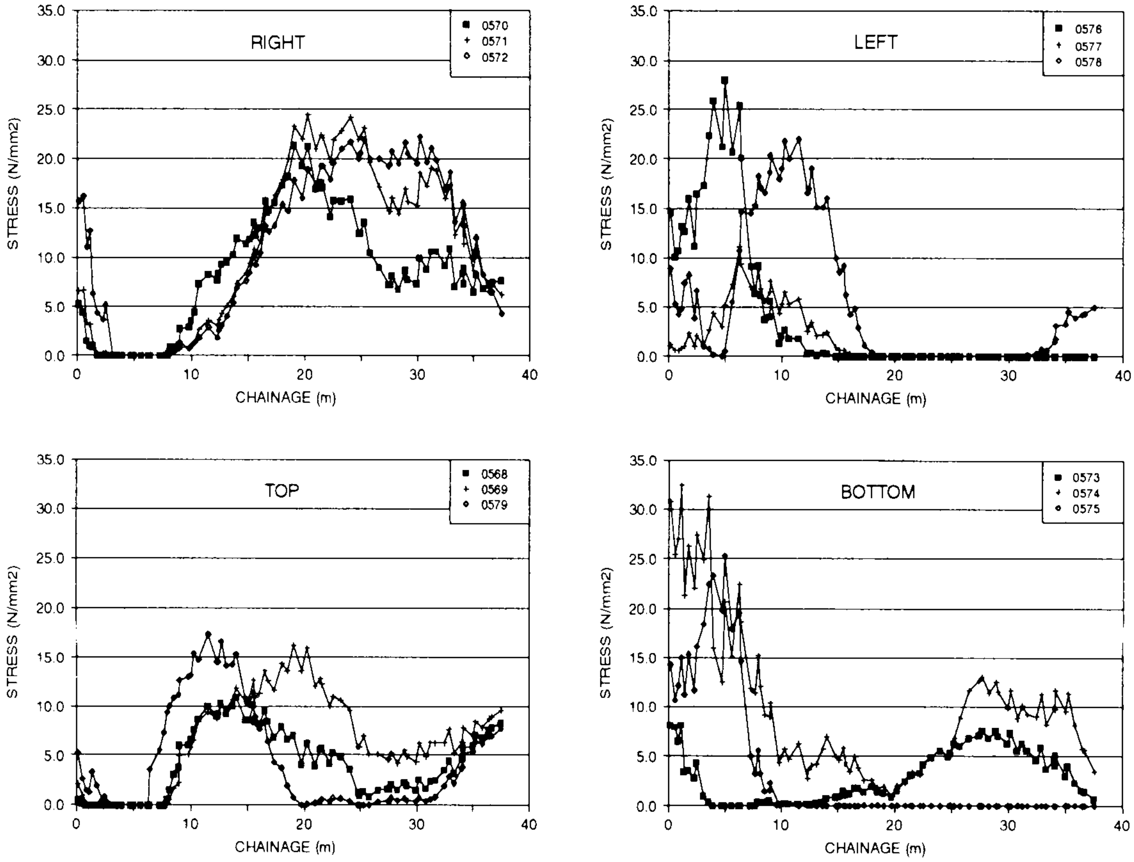


Figure 5.17 Variation in the rear joint pressure distribution obtained from scheme 3.

An important point to note from Figure 5.15 is the possible fatigue effect in pipes close to interjacks due to the increased number of large intensity stress cycles. This effect is dependent upon the contractor's interpretation of interjack usage.

5.4 Predicting pipe joint behaviour

Results connected with joint behaviour seem consistent with expectations and measured small deviations from line and level along the pipe string. The effects are essentially geometrical and related mainly to the accuracy of the drive, and are otherwise largely independent of the nature of the ground. Exact correlation of joint angles and contact stresses is complicated by the non-linear and stress-history-related behaviour of the packing material, as discussed in section 5.3. However the site instruments enable total joint loads to be accurately determined and packer material properties which take account of the previous stress history of the joint to be used . On this basis comparisons between measured and back analysed stresses using the Australian linear stress model, described in Figure 2.7, are presented in Figures 5.19 to 5.20. The appropriate packer stiffness is labelled E_p in the figures and the permanent packer compression is obtained by projecting the stiffness line back to zero stress. Curves linking maxσ_j/σ_{jo} and maxσ_j/σ_{jo}/z/R have been produced from the tables of Marks (1978), Figure 5.18.

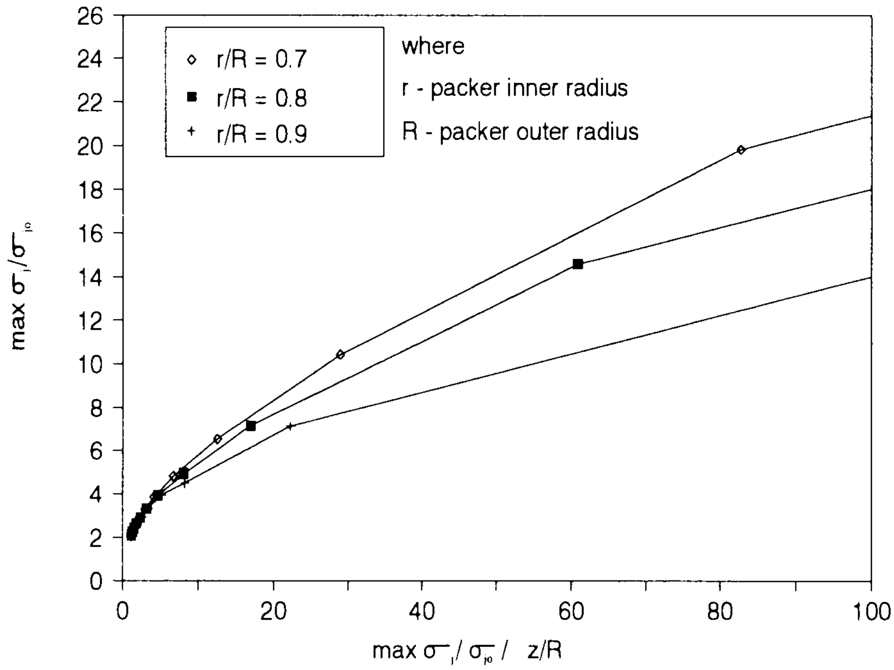
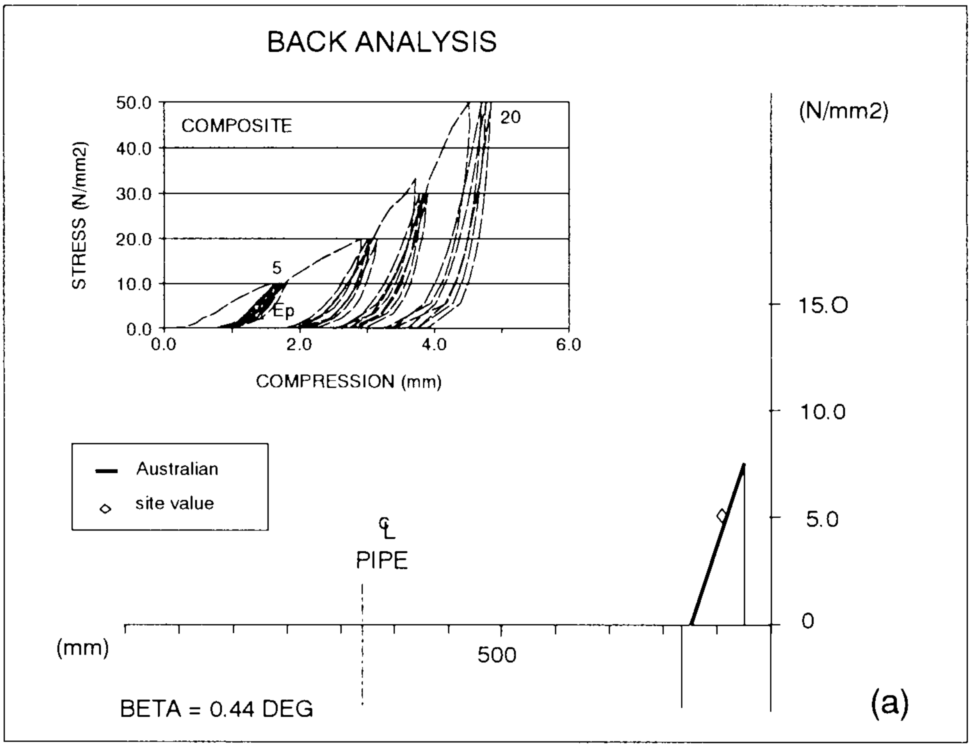


Figure 5.18

The comparisons show that the Australian approach can adequately match the measured stresses within the limits of the uncertainty of the data, and could be used to define allowable jacking loads on pipes for specified misalignment angles on the basis of the strength of the concrete and the stiffness of the packer defined over a suitable unload-reload stress range.



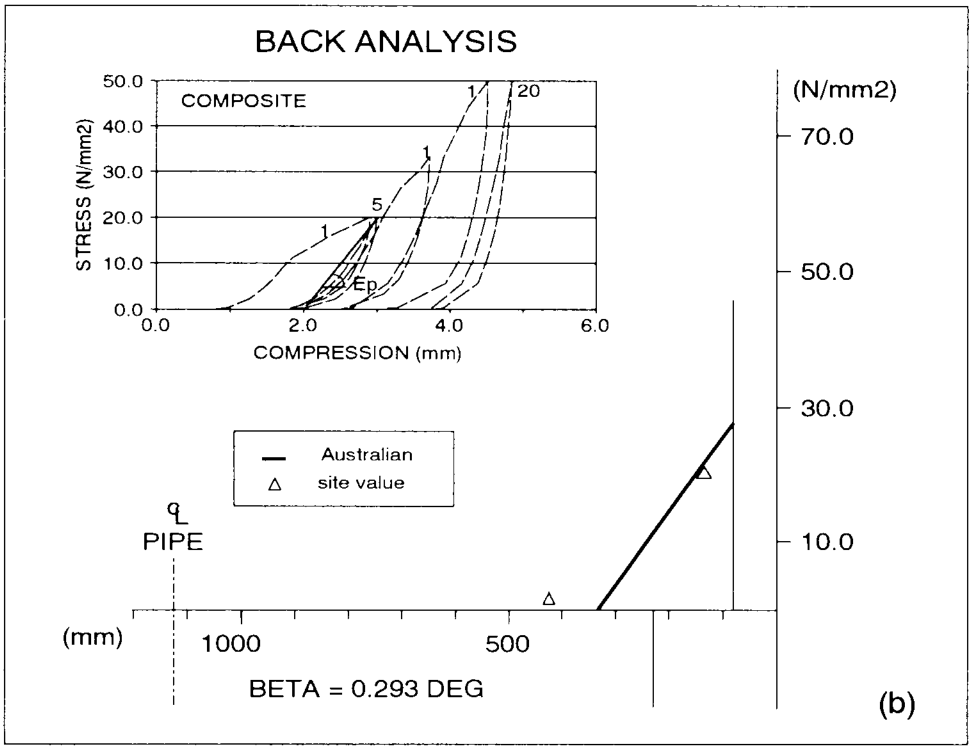
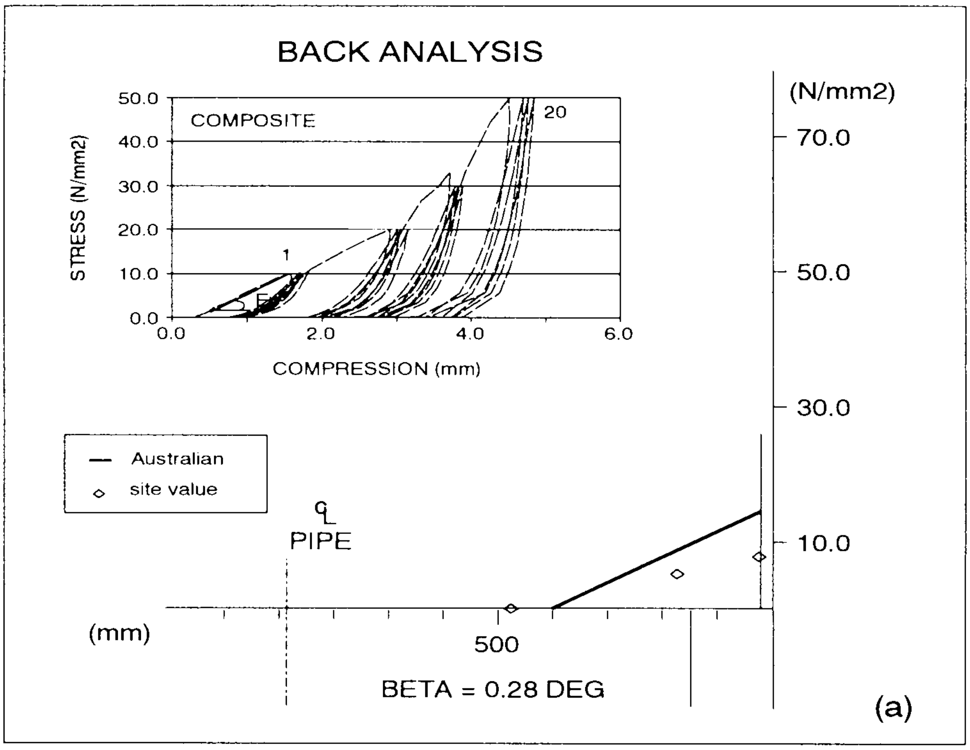


Figure 5.19 Comparison of measured joint stress distribution to predicted using a linear stress model based on the Australian Concrete Pipe Association method. (a) scheme 1 rear joint subject to a β value of 0.44°, and (b) scheme 3 rear joint subject to a β value of 0.29°.



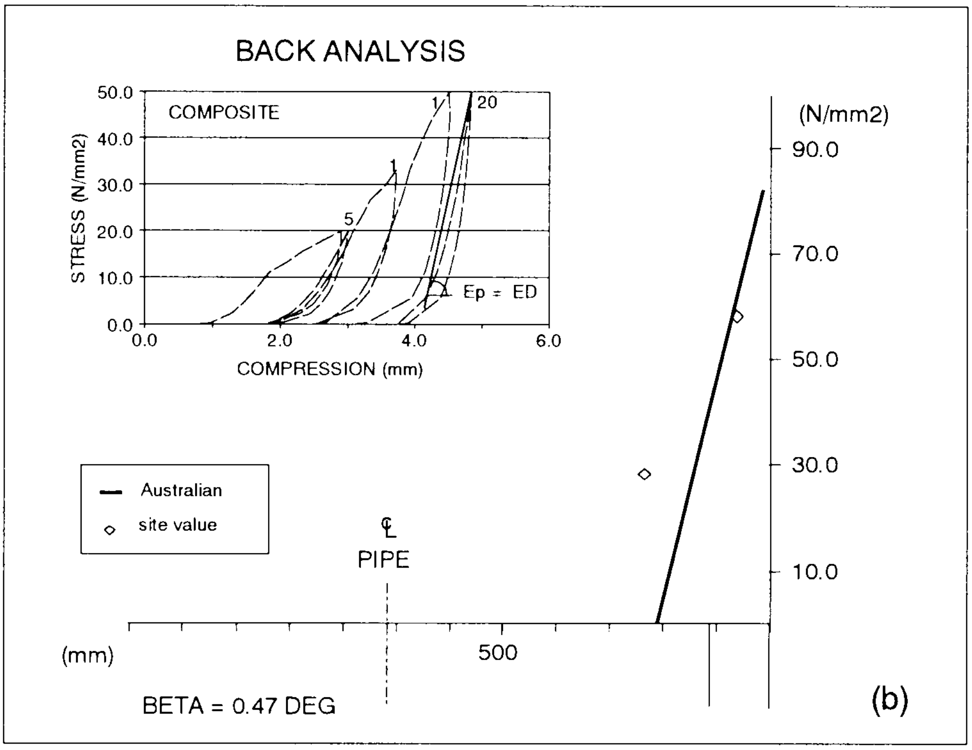


Figure 5.20 Comparison of measured joint stress distribution to predicted using a linear stress model based on the Australian Concrete Pipe Association method. (a) scheme 4 rear joint subject to a β value of 0.28°, and (b) scheme 5 rear joint subject to a β value of 0.47° and pipe damage.

5.5 Design approach

In adopting the Australian CPA model as a basis for evaluating allowable pipe end loads it is first necessary to define appropriate limits for the various parameters.

5.5.1 Permissible joint face strength

The field results have demonstrated that the steel banded joint detail can sustain average stresses over a cell area of 200x100mm equivalent to the characteristic cube strength of the concrete. The localisation of stress is generally severe with the peak stress rapidly reducing. This suggests that the joint face strength limits of BS 5911:Part 120 may be applicable. For design purposes two values of maximum permissible joint stress are considered:- $0.8f_{cu}$, which is the generally accepted maximum bearing stress in reinforced concrete under well confined conditions, and the lower limit of $70N/mm^2$ from the joint face strength test.

5.5.2 Packer material properties

Medium Density Fibreboard was used in the instrumented joints of all five schemes. Tests on other common construction materials were carried out by Ripley (1989). Exterior Grade Plywood has been selected for comparative purposes because it is currently the most popular material with pipe jacking contractors.

Stiffness

Figure 5.21 compares insitu stress and joint compression from a single cell on scheme 3 with the laboratory compression tests. The field data show that behaviour tends to that of precompressed material. For design purposes a linearisation of the reloading stiffness for pre-compressed material, such as that labelled ED in Figure 5.21 would be the safest. The value should be determined from packer compression tests using a maximum stress intensity of $0.8f_{cu}$. Using this approach values of $550N/mm^2$ and $700N/mm^2$ are obtained for MDF and plywood respectively.

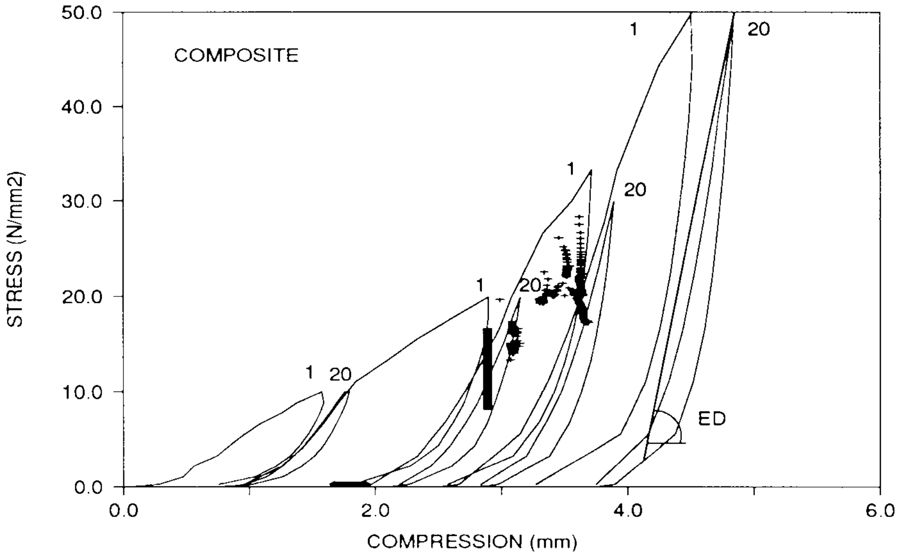


Figure 5.21 Insitu stiffness evaluation using data from scheme 3 and suggested design stiffness.

Permanent packer deformation

The permanent packer deformation at these high stress levels approximates to 0.3 times the original thickness for MDF and 0.45 times the original thickness for plywood.

5.5.3 Positioning in joint

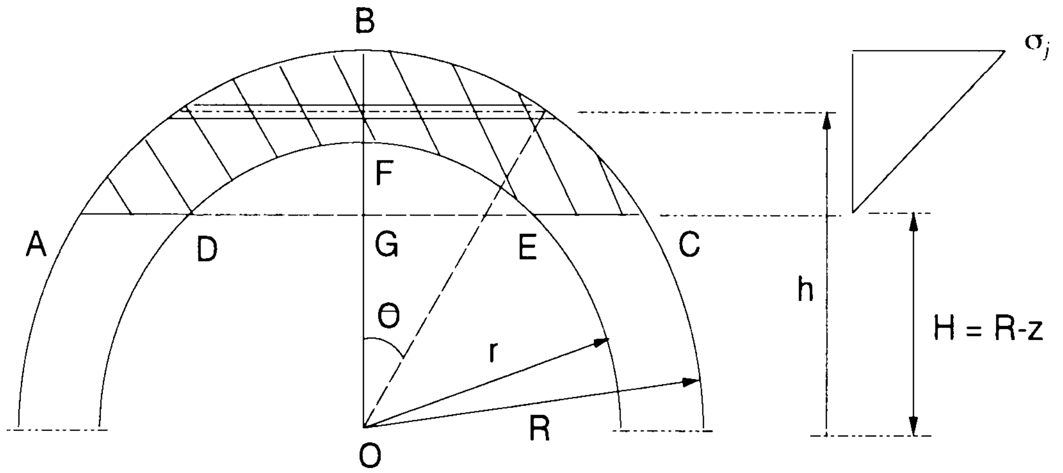
Ideally the packer should be made as wide as possible to maximise the available load area. However local bursting effects and large shear forces in joints (values up to 25 tonnes are indicated in Chapter 7) can cause joint spalling. To minimise the risk of superficial spalling, it is suggested that the packer is placed centrally in the joint with a minimum edge distance equivalent to the longitudinal reinforcement cover or 20mm which ever is the smaller. For the pipe size range in the research, typical wall thickness varied between 115mm and 225mm with the joint wall thickness some 15mm smaller. The range is clearly large although variations in thickness for the product ranges of a single manufacturer are much more comparable. Two packer widths are considered in this parametric study, 100mm to suit the research and 80mm which was frequently used by the participating contractors.

A typical set of calculations and the resulting design charts are produced as follows:

From the Australian linear stress approach

$$z = \frac{180}{\pi} \frac{a}{E_p} \frac{\max \sigma_j}{\beta}$$

Total force over hatched area for linear distribution of stress



Width of element = dh

Length of element = $2\sqrt{R^2 - h^2}$

Area of element = $2\sqrt{R^2 - h^2} \quad dh$

Stress on element = $\dfrac{\sigma_j (h - H)}{(R - H)}$

Force on element = $\dfrac{2\sigma_j (h - H)\sqrt{R^2 - h^2}\, dh}{(R - H)}$

Total force on area ABCEGDA = $\displaystyle\int_H^R \frac{2\sigma_j (h - H)\sqrt{R^2 - h^2}\, dh}{(R - H)}$

Similarly force on area DFEGD = $\displaystyle\int_H^r \frac{2\sigma_j (h - H)\sqrt{r^2 - h^2}\, dh}{(R - H)}$

Therefore force on area ABCEFDA = $F_{permissible}$ =

$$\frac{\sigma_j}{(R-H)} \left\{ \int_H^R 2h\sqrt{R^2-h^2}\,dh \;-\; \int_H^r 2h\sqrt{r^2-h^2}\,dh \right.$$

$$\left. -\; 2H\int_H^R \sqrt{R^2-h^2}\,dh \;+\; 2H\int_H^r \sqrt{r^2-h^2}\,dh \right\}$$

$$= \frac{\sigma_j}{(R-H)} \left\{ 2/3[\,(R^2-H^2)^{3/2} - (r^2-H^2)^{3/2}\,] - H\left[R^2\cos^{-1}\left(\frac{H}{R}\right) - r^2\cos^{-1}\left(\frac{H}{r}\right) \right] \right.$$

$$\left. + H^2[\,(R^2-H^2)^{1/2} - (r^2-H^2)^{1/2}\,] \right\}$$

When H > r $\quad F_{permissible}$ =

$$\frac{\sigma_j}{(R-H)} \left\{ 2/3[\,(R^2-H^2)^{3/2}\,] - H\left[R^2\cos^{-1}\left(\frac{H}{R}\right) \right] + H^2(R^2-H^2)^{1/2} \right\}$$

Consider the case of a 1200mm internal diameter pipe with a 12mm MDF packer 100mm wide positioned 20mm from the internal edge subject to a range of angular deviations.

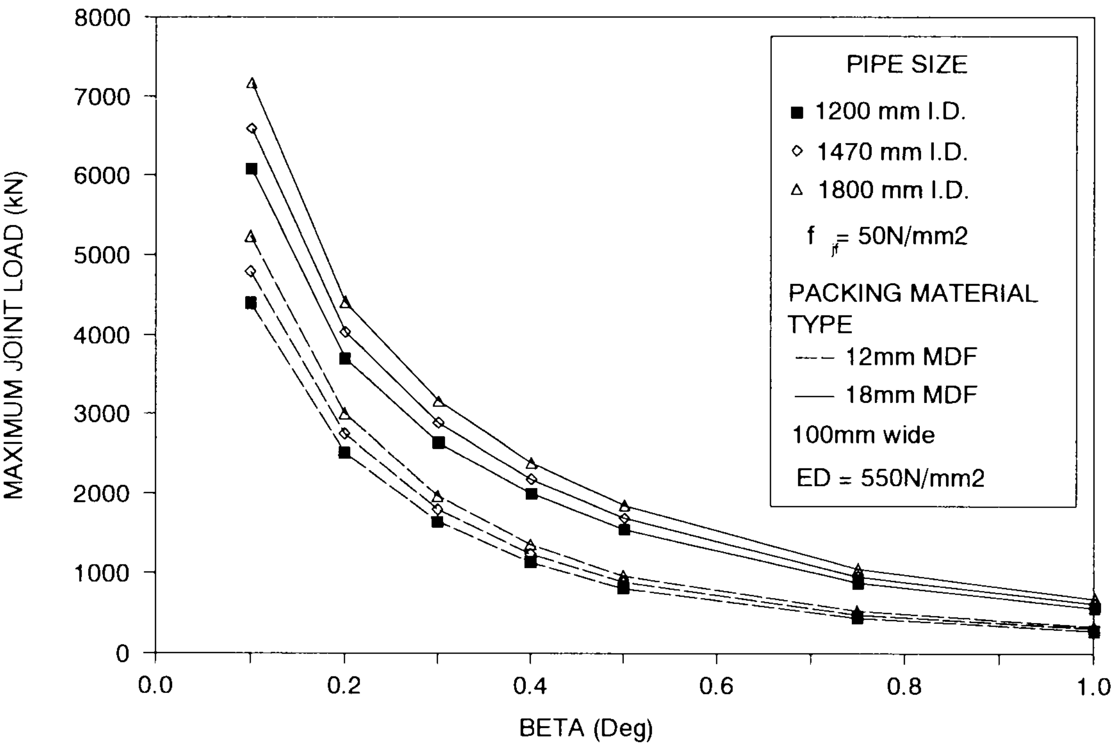
maxσ_j = 70N/mm^2

a_{12} = 0.7 x 12 = 8.4mm (composite joint assumed to be similar to 12mm MDF)

ED = 550N/mm^2

ß (deg)	z (mm)	R (mm)	r (mm)	H (mm)	$F_{permissible}$ (kN)
0.1	613	720	620	107	7830
0.2	306	720	620	414	4706
0.3	204	720	620	516	3302
0.4	153	720	620	567	2445
0.5	123	720	620	597	1870
0.75	82	720	620	638	1039
1.0	61	720	620	659	669

Calculations can be carried out for a range of pipe sizes, joint stress limits and packer material types and dimensions, resulting in design charts similar to those in Figures 5.22 to 5.24.



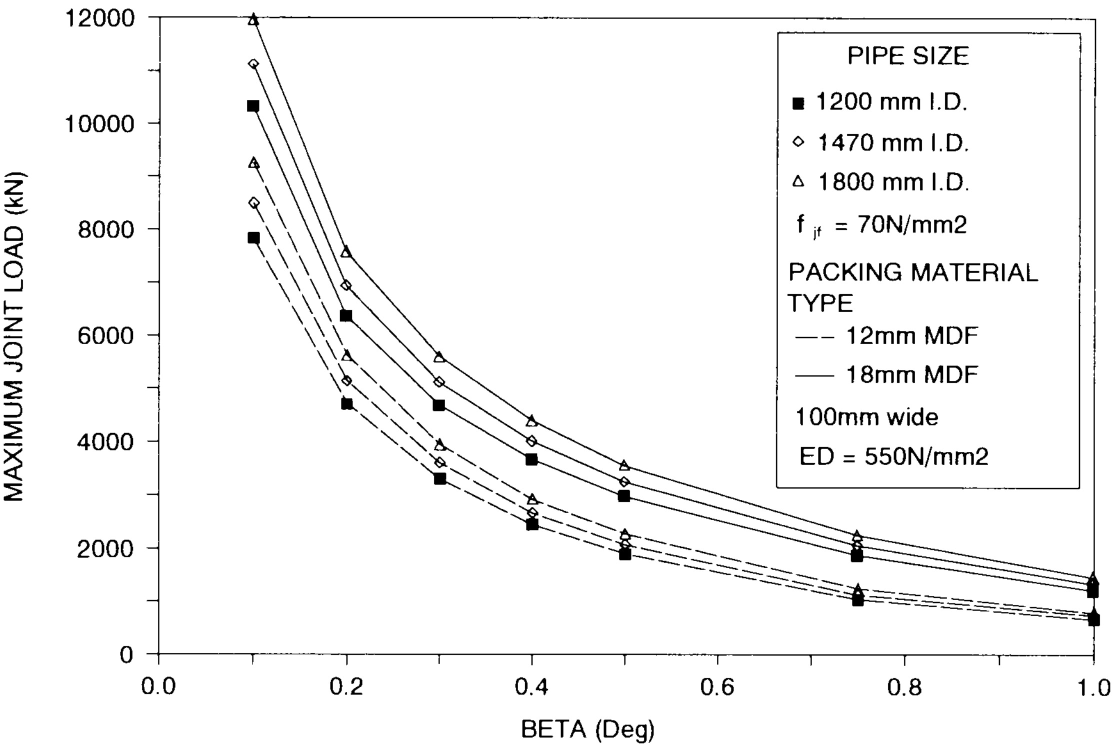


Figure 5.22 Permissible pipe end loading at various angular misalignments; 100mm wide Medium Density Fibreboard.

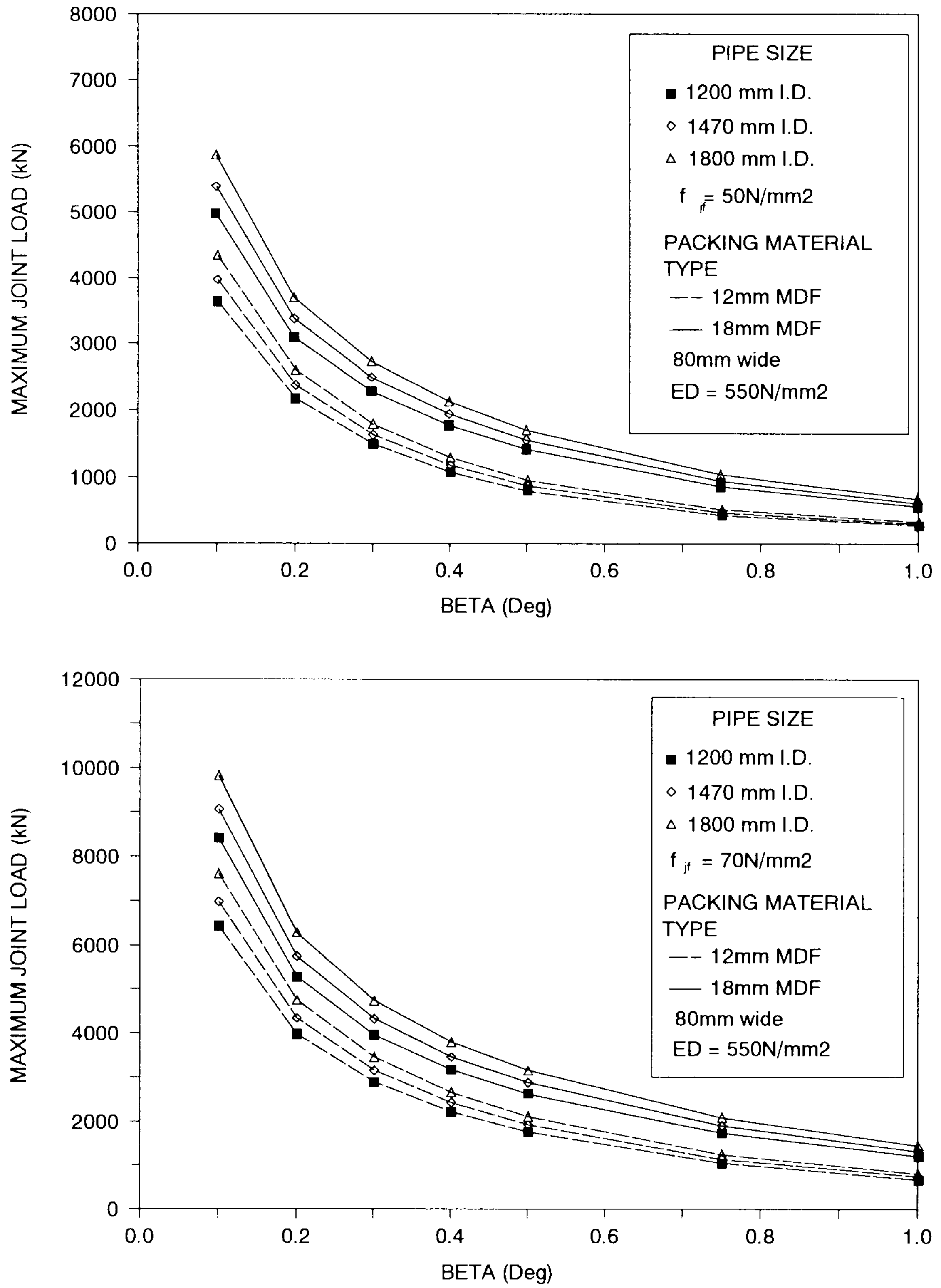
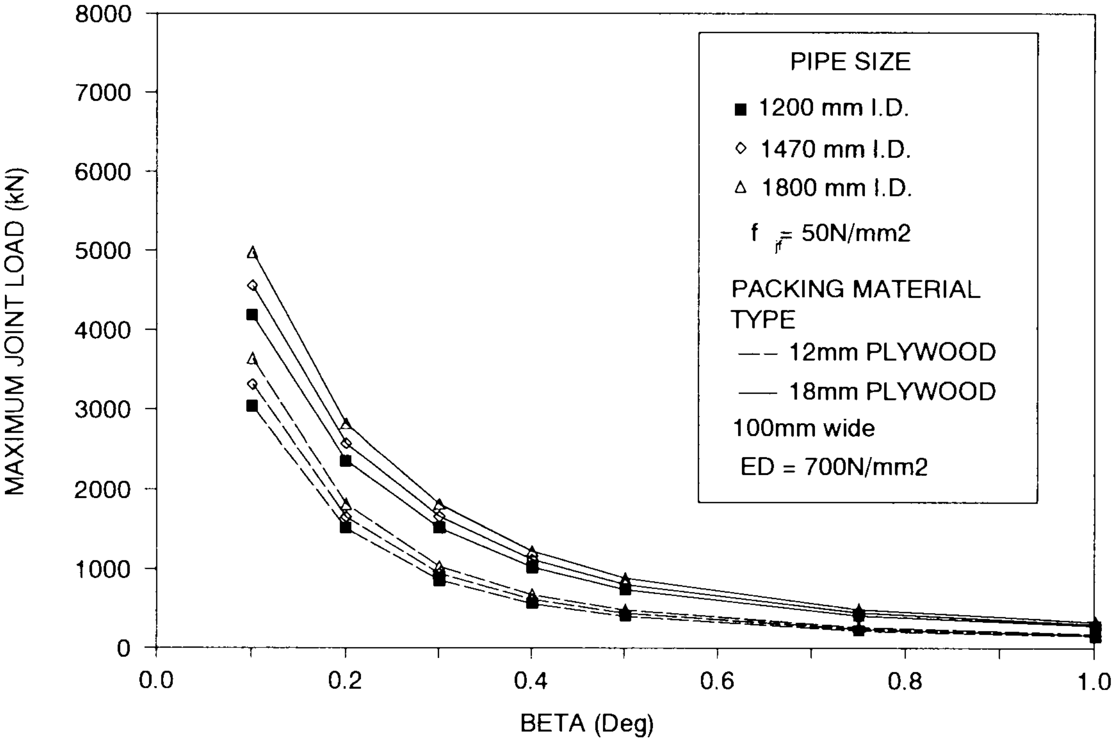


Figure 5.23 Permissible pipe end loading at various angular misalignments; 80mm wide Medium Density Fibreboard.



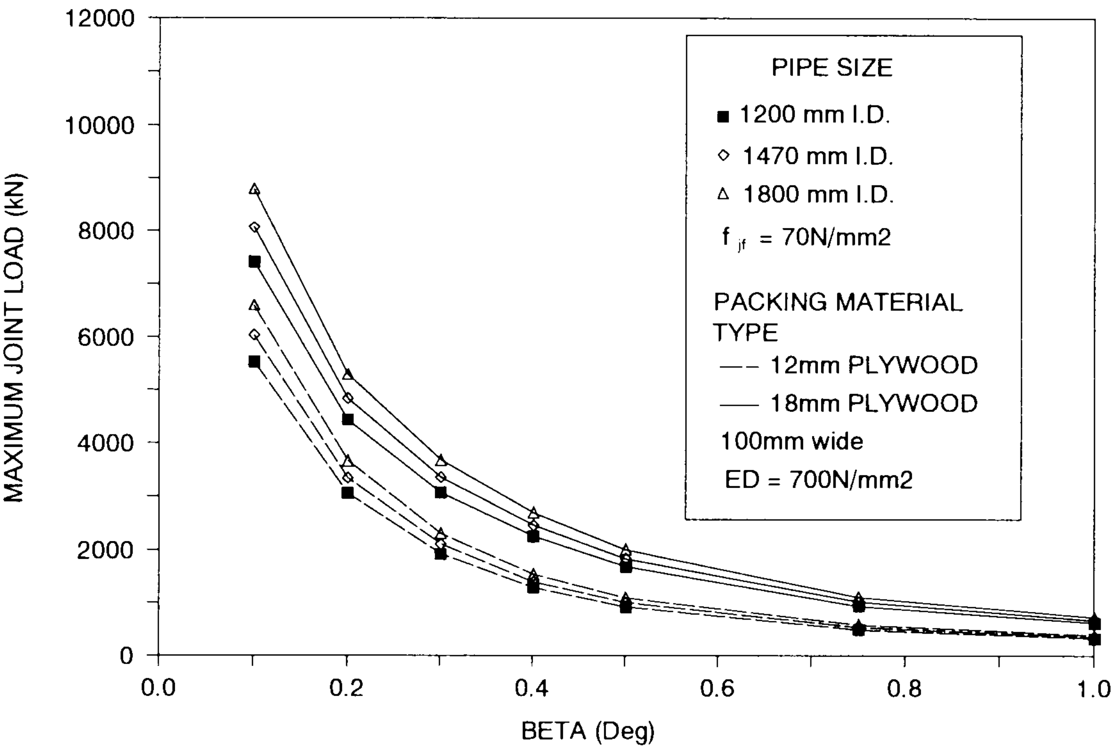


Figure 5.24 Permissible pipe end loading at various angular misalignments; 100mm wide Exterior Grade Plywood.

Examination of the design charts provides some indication of the influences of the various parameters used in the linear stress model. The distribution of stress on the end of a pipe is the same irrespective of pipe diameter and packer width. The observed increases in allowable jacking load with different pipe diameters and packer widths result from the increase in available end area. An 18% increase in allowable load is obtained if the pipe diameter is increased from 1200mm to 1800mm internal diameter, the percentage increase being independent of β. Increasing the packer width from 80mm to 100mm has less influence at larger values of β. Comparisons at 0.1° and 0.5° produce increases in permissible jacking loads of 21% and 4% respectively when MDF is used. Packer thickness is a key parameter. An increase from 12mm to 18mm produces a 38% increase in allowable load at 0.1° and 89% at 0.5°. Increases in concrete strength also produce significant increases in load carrying capacity at larger β values, 18% at 0.1° and 54% at 0.5°.

Use of plywood with all other parameters similar produces a 31% reduction in allowable load at 0.1° misalignment and halves the capacity at 0.5° compared to the response with MDF.

The most significant parameters for increasing allowable pipe end loads at moderate misalignment angles are therefore packer type (i.e. both stiffness and permanent compaction), packer thickness and concrete strength. A hypothetical case is presented in Figure 5.25. The material stiffness is half that of MDF, with an original thickness of 25mm and 80% recovery under maximum stress cycling. The maximum joint face stress has been limited to $0.8f_{cu}$ since the resulting stress distribution will probably have departed significantly from the highly concentrated joint face strength test configuration. Significant improvements in load carrying capacity over that given by 18mm MDF, which is the best of a number of wood based products, is illustrated. Clearly the packer should be considered as part of the pipe and fixed to it at the pipe works. There is obviously scope for the development of better, possibly polymer based, materials.

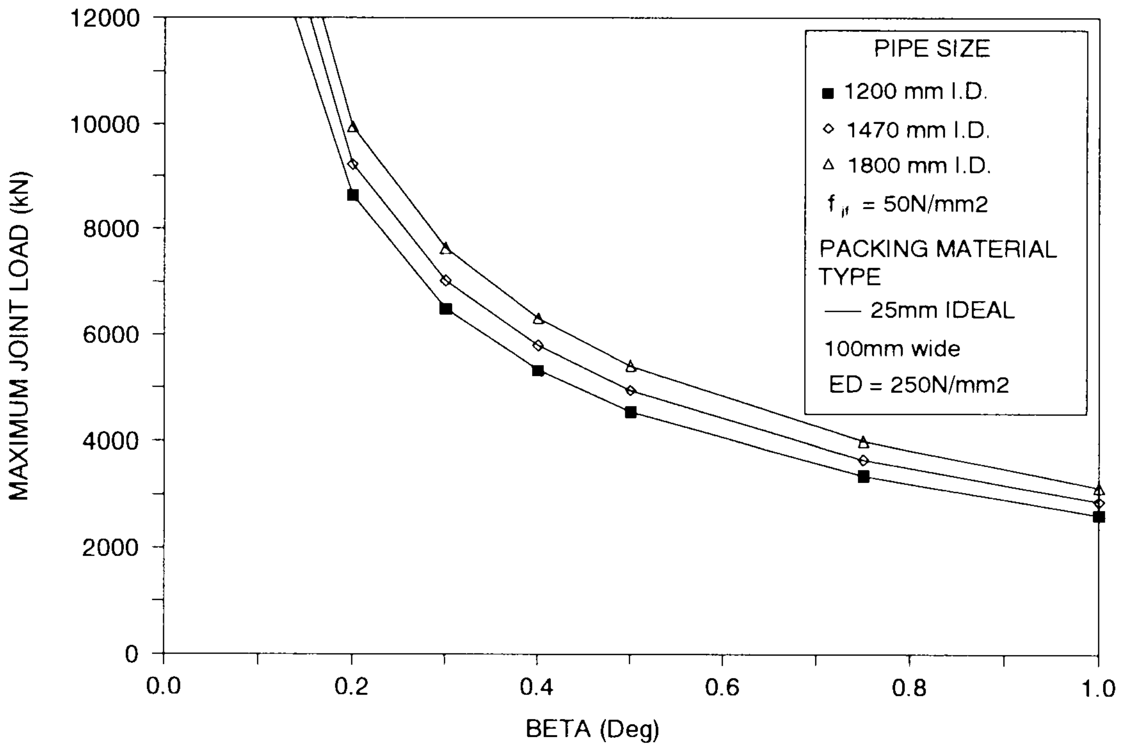


Figure 5.25 Permissible pipe end loading at various angular misalignments; 100mm wide idealised packing material.

Existing data to verify the suitability of the proposed design approach is scarce. The field data from scheme 5 is the only suitable direct comparison. The design value for $\beta=0.47°$ is 207 tonnes whereas a maximum measured value of some 380 tonnes was ultimately sustained by the pipe, albeit with local compressive failure. The approach appears to have an adequate inherent factor of safety at approximately 1.8.

CHAPTER 6:
SOIL-PIPE INTERFACE BEHAVIOUR

6.1 Introduction

The provision of sufficient jacking capacity is largely based on previous experience from schemes in similar ground conditions. In general, contractors design jacking systems with an excess capacity of 25-30% greater than the expected load to allow for misalignments or unexpected ground conditions. To improve the design situation, it is necessary to review the parameters which influence jacking loads and rationalise them in relation to theoretical methods and laboratory test data, for predicting ground pressures on pipes and appropriate friction coefficients. Comparison of theoretical values with actual jacking loads is essential to validate any design approach.

The jacking load depends upon the force required to push the shield into the excavation, commonly referred to as face resistance, and the frictional resistance along the length of the pipeline. The amount of resistance encountered at the face depends upon ground conditions and the measures required to support the face. It is predominantly related to the edge cutting resistance of the jacking shield and overall size is important as larger dimensions produce larger face resistances. Currently friction resistance can only be estimated within large limits, Table 2.2. The variability within basic soil types throws the credibility for "design" back onto the individual contractor's experience.

This chapter initially considers the total jacking resistance for each of the instrumented schemes. A detailed examination of the soil pipe interaction process follows with two sets of parameters being investigated: ground related including soil type, moisture content and grading; and construction related including pipeline misalignment, stoppage time and the use of lubricants.

6.2 Basic data

6.2.1 Jacking records

Progress and jacking force records for the five schemes are illustrated in Figures 6.1 to 6.5. The length of drive plotted along the horizontal axis does not include the length of shield or TBM which has been assumed to provide a constant face resistance as measured off the vertical axis. The average frictional resistance (jacking force divided by pipeline length) has been added to the graphs.

A prominent feature of the jacking record for scheme 1, Figure 6.1, is the change in average jacking resistance at chainage 35m from a uniform rate of 7.2kN/m to 29.8kN/m. The change in slope corresponds to a significant change in weather from hot dry conditions to torrential rain throughout the remainder of the contract, which appears to have affected the ground conditions to the depth of the pipe jack. Generally progress of 6m per shift was achieved. The jacking force increased from an initial face resistance of 120kN to a maximum of 1380kN. The effect of overnight and weekend stoppages resulted in increased restart forces, typically 200kN greater than at the end of the previous push. The weekend stoppage of the 18/19 August was preceded by tunnel flooding due to a fractured water main with the restart forces some 400kN larger and the post jacking resistance remaining significantly greater.

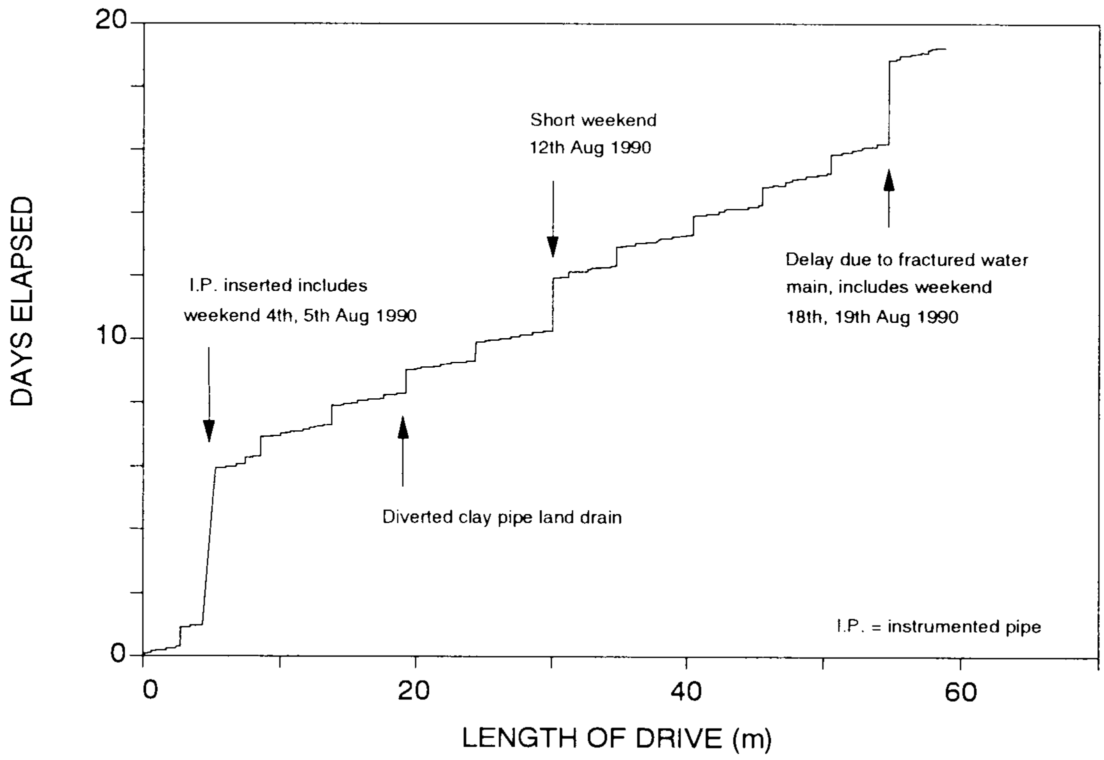
The jacking record for scheme 2, Figure 6.2, suggests that the main jacking resistance occurs at the tunnel face. Extending the average jacking resistance line backwards results in an intercept of 950kN. This high face resistance is the result of strong to slightly weathered mudstone being encountered in the pipe jack invert. The erratic nature of the jacking load probably reflects the variation in the extent of trimming of the excavation. This parameter could not be adequately monitored during this scheme. Progress was relatively slow at 3m per shift; a direct result of the difficulty of excavating the mudstone. The low rate of advance and in particular the eleven day stoppage while negotiating the disused mine shaft do not appear to have caused larger restart jacking forces. The average increase in jacking resistance was estimated as 8kN/m for the mudstone section with a larger value of 54kN/m through the boulder clay.

The pattern of jacking load build up for scheme 3, Figure 6.3,shows a relatively uniform increase of 54kN/m and a face resistance of 300kN. The blocky nature of the ground upon excavation made control of shield trimming difficult. The peaky time dependent response is particularly pronounced with each push illustrating an increase in resistance upon restart followed by a rapid decay to the average jacking resistance line. The magnitude of the oscillations increase with increasing drive length. The contractor changed from single to double shift working to reduce the length of stoppages after a 68% increase in jacking resistance after the weekend stoppage of 16/17 March. The continuous working phase resulted in little change in the ratio of peak to average jacking resistance suggesting that a substantial part of the "time factor" effect occurs within short periods of time. This effect is considered in greater detail in section 6.4.2

The jacking records for scheme 4 are presented in Figure 6.4. This scheme provided the first opportunity to monitor the effects of changes in the extent of shield trimming and the use of lubrication. Both of these factors will be considered in greater detail later in the chapter. Prior to lubricating, the jacking force increased at a uniform rate of 23.1kN/m. The well defined peaks at various positions along the drive are the result of increases in face resistance caused by changes in the extent of shield trimming. Initially the miner was trimming approximately 20mm with the shield to minimise overbreak. Upon request a number of pushes were carried out with reduced distances. The resulting large reduction in jacking resistance prompted the miner to revert to generally excavating to the outside diameter of the shield except when changes in alignment were necessary, and shield resistance was required to reset the steering jacks. The implications of surface settlement were negligible given the line of the jack through a green field site. Stoppages had little effect on the restart jacking loads over the non-lubricated section.

Lubrication was introduced at a drive length of 82m. For the first two days of injection, with a tunnel advance of 5m per shift, the bentonite lubricant had little effect on the jacking resistance. However, between chainages 90-102m the jacking resistance dropped by 15%. Close inspection of the tunnel face log indicates that boulder clay was present in the invert. At chainage 102m running sand entered the bottom of the face with the boulder clay horizon moving into the top half of the face. The running sand has probably increased the face resistance. The peaky response is likely to be the result of the miner burying the shield hood into the boulder clay to prevent the shield from tipping. The resulting increase in jacking resistance approaches the same value that would have been obtained by extending the average rate of increase prior to lubricating to the end chainage. This suggests that the lubrication has only been partially effective.

Scheme 5 was a mechanised drive; the jacking records are included as Figure 6.5. The scheme is characterised by a high face resistance of 1500kN and a low average frictional resistance of 17kN/m. The large rate of increase over the initial 25m is a result of the ground being un-lubricated because of concern over blowout so close to the jacking pit. Use of bentonite lubrication significantly reduces the rate of increase of the total jacking loads. The variability appears to be a function of the rate of pipe installation with the lower resistances, particularly between chainages 30m and 60m corresponding to an increase in daily productivity from 17.5m to 27.5m. Periods of inactivity and overnight stoppages result in increases in restart jacking forces as large as 25%. The effects of pipeline misalignment are clearly seen from the plot with an overall increase of 68% caused by the reverse bend between chainages 110m and 130m. Activation of the first interjack positioned 81m behind the machine has produced a banded response on the jacking record. The lower bound is the closing up of the rear of the pipeline into the interjack and the upper values are the combined resistance of the front and rear sections. An interesting feature is the much higher resistance between machine and interjack. This could be the result of less effective lubrication as the tunnel length increases and pressure losses along the injection system become more significant.



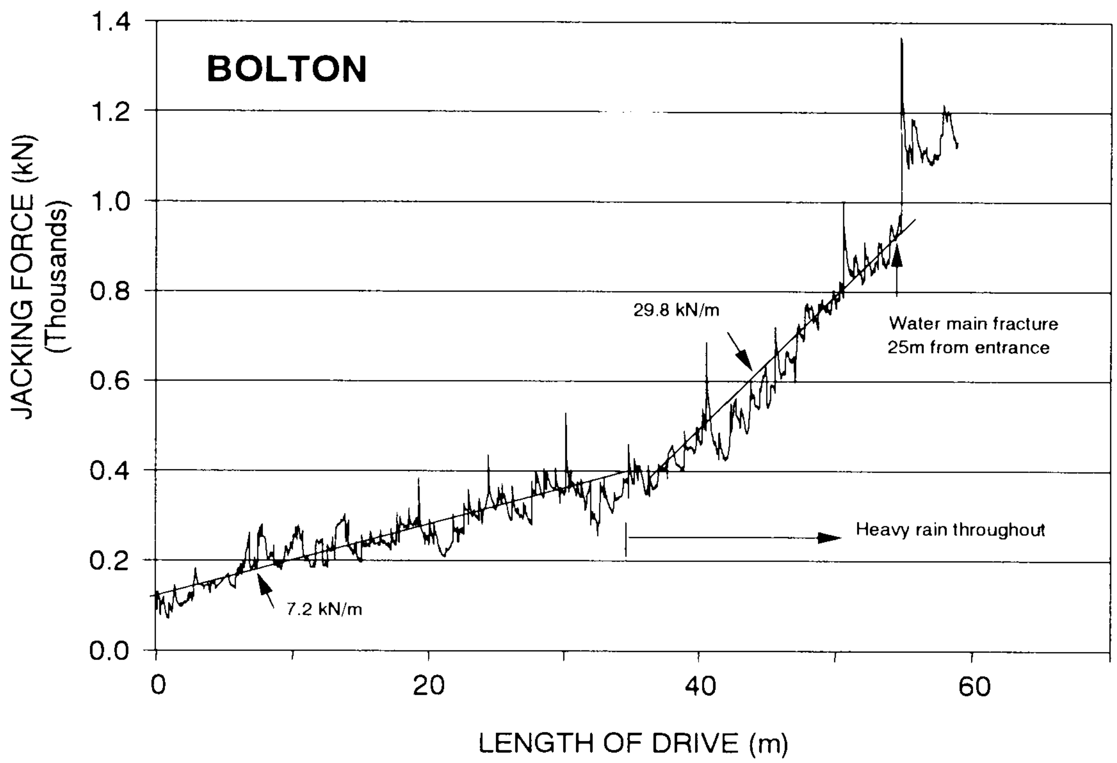
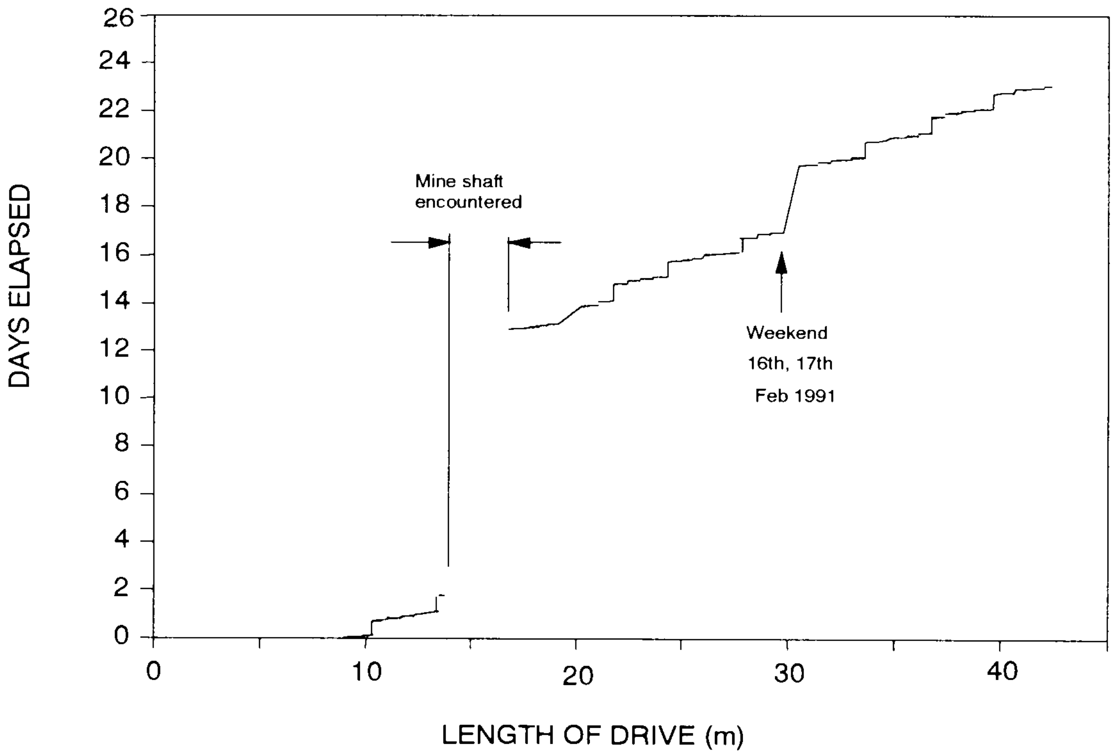


Figure 6.1 Jacking records for scheme 1.



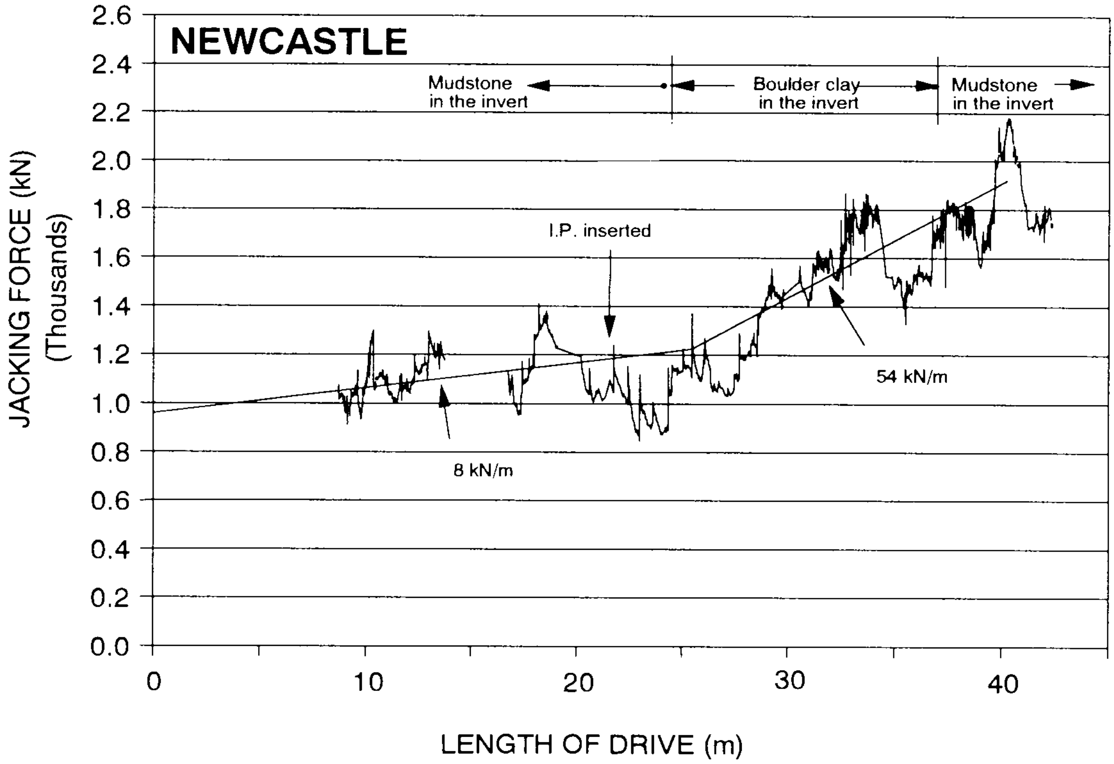


Figure 6.2 Jacking records for scheme 2.

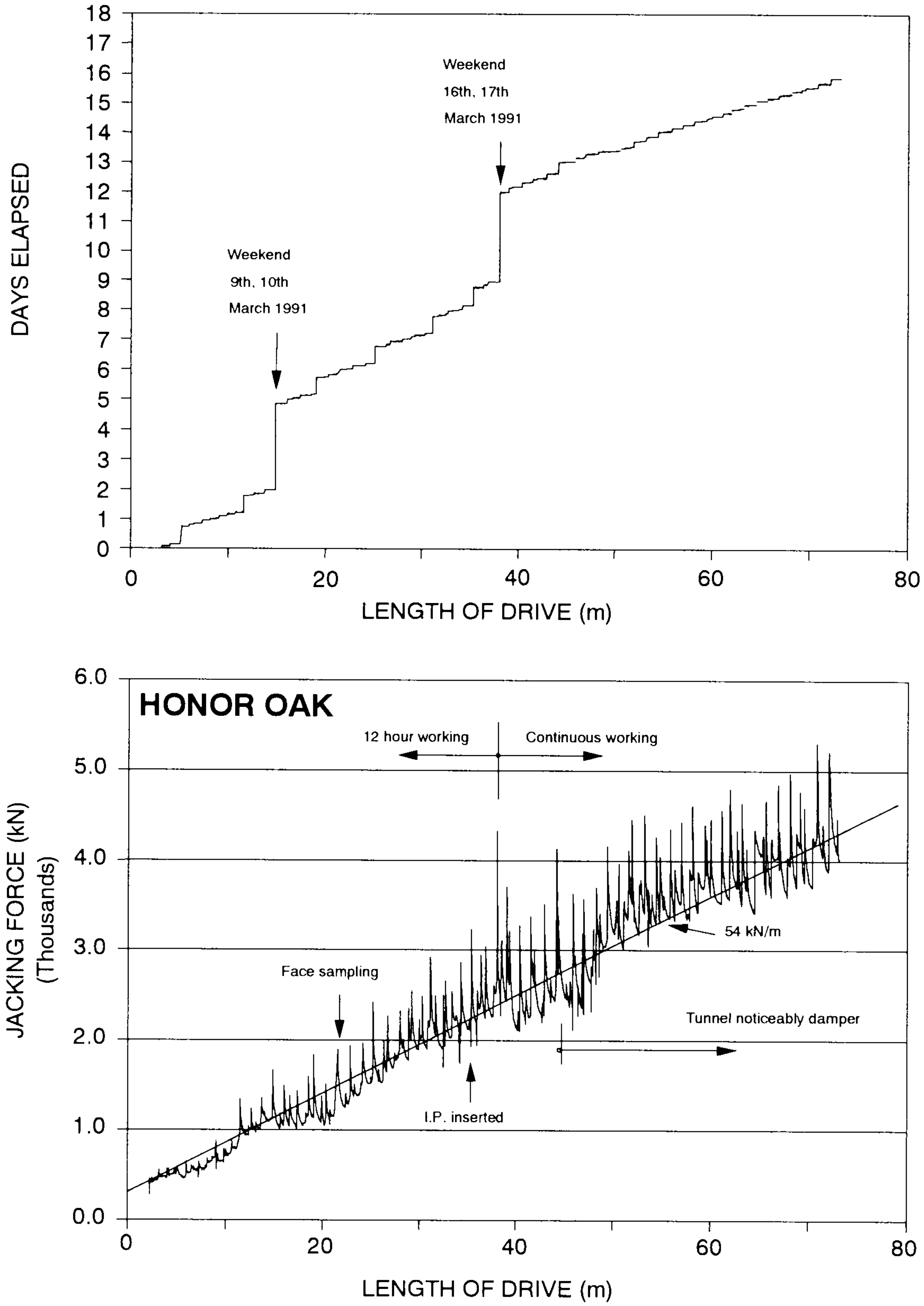
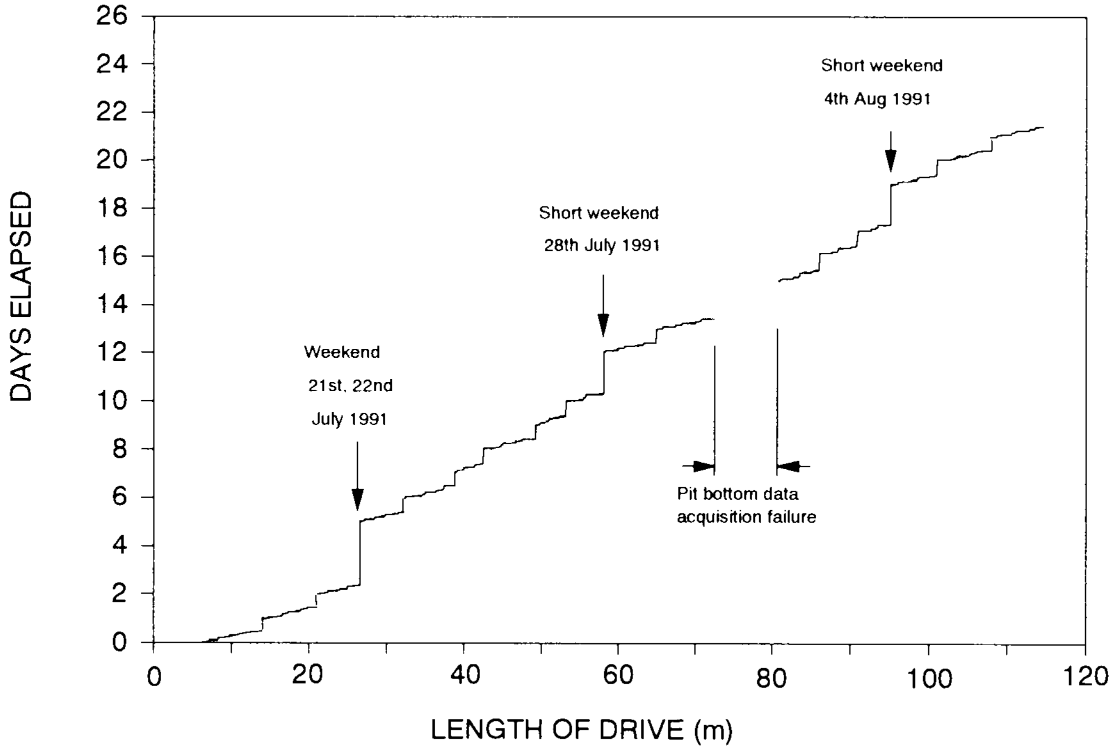


Figure 6.3 Jacking records for scheme 3.



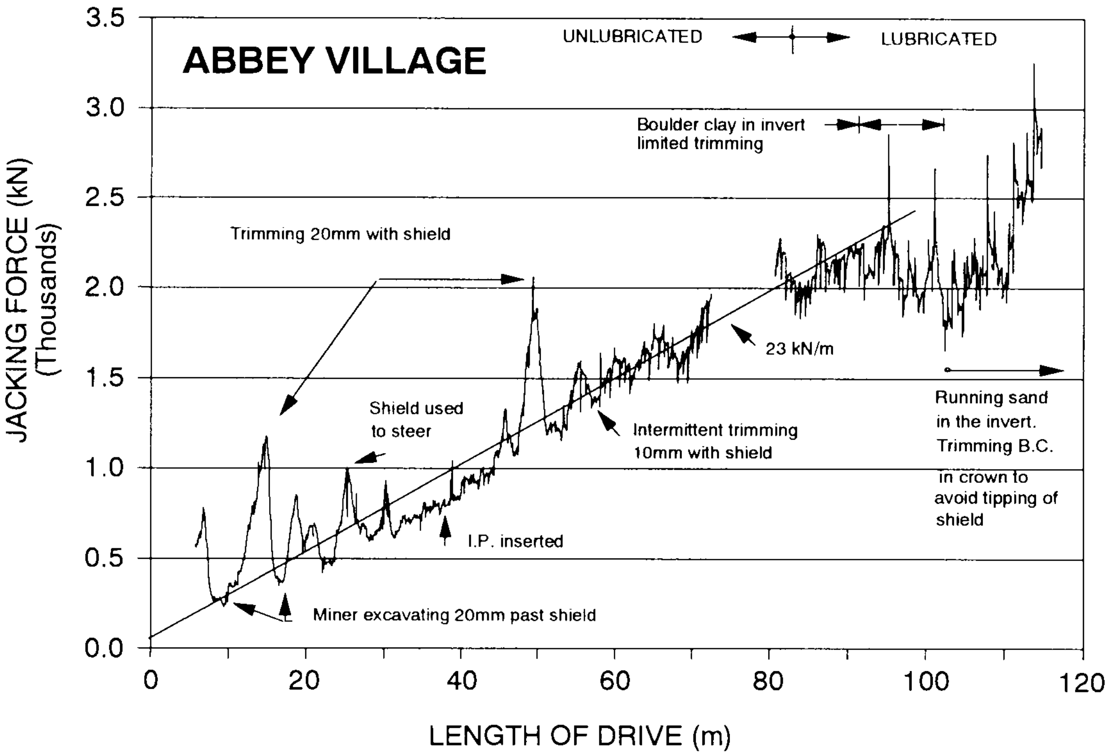


Figure 6.4 Jacking records for scheme 4.

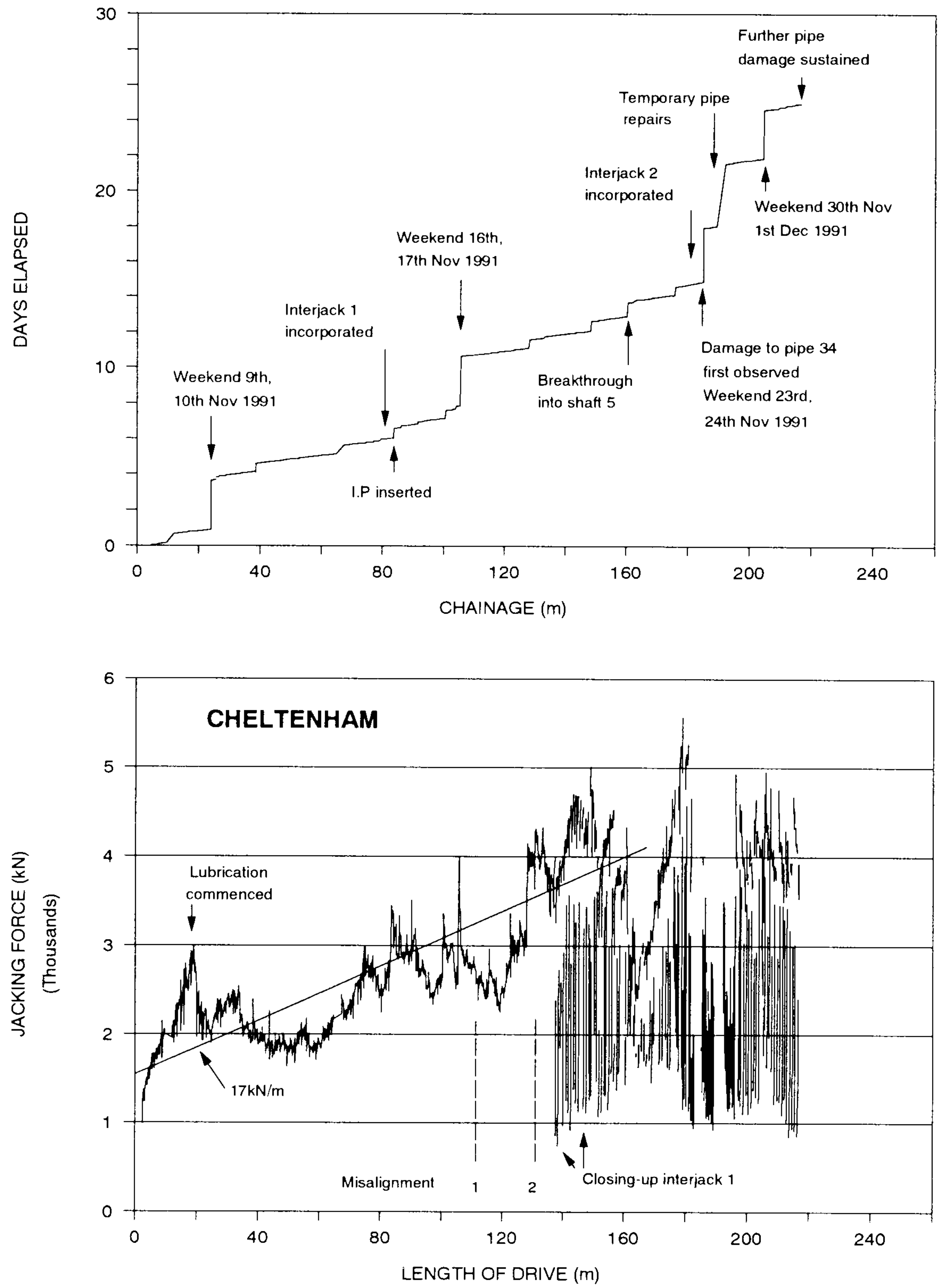


Figure 6.5 Jacking records for scheme 5.

6.2.2 Pipe jacking loads

Jacking records are generally the only information on ground related loading that the contractor has at his disposal. They allow values of nominal face resistance and average pipeline friction to be determined in different ground conditions. The values from the instrumented schemes are included as Table 6.1. Face resistance can be seen to account for a significant proportion of the total load. In cohesive ground the value varies from 6-9% although there is probably considerable variability in the value for scheme 3 because of the highly fissured state of the excavated material. Variability in face resistance is inherent in all hand drives and is clearly illustrated in scheme 4 were the value varies from 3% when the shield is only in contact with the base of the bore to 25% when trimming approximately 20mm all around the excavation. The large value of 27% for the machine drive is a function of the slurry pressure used to support the face, while the value of 43% from the rock drive is a function of the mudstone in the invert.

Scheme		Measured face resistance		Measured average friction		Craig (1983) limits
		kN	% Total	(kN/m)	(kPa)	(kPa)
1	Dry Wet	120	9	7.2 29.8	1.5 6.2	5-18
2		950	43*	18.0	1.5	2-3
3		300	6-8	54.4	7.6	5-20
4	unlub lub	100-800	3-25	23.1 9.4	4.2 1.7	5-20
5	lub	1500	27	17	3.6	10-15■

* Value based on 40m monitored length. Total drive length 100m
■ Unlubricated Wet Sand

Table 6.1 Average face resistance and pipeline friction.

Comparison of the average friction values with typical friction limits, Craig (1983), show that the measured values are below or at the lower limit of the generally accepted ranges. This is a function of the competent nature of the cohesive ground in schemes 1 to 3, Figure 6.6, and the well controlled alignment of the drives. For the mainly non-cohesive drives of schemes 4 and 5 small support pressures, σ_T, (see section 2.3, Table 6.2 and assume φ' is 40° and the water table is 1m above the top of the pipe in scheme 5) of 10kpa and 14kPa respectively were required to prevent collapse of the material onto the pipes. In scheme 4 this was provided by capillary suction within the silty material leading to a relatively stable bore for up to five days after excavation. In the loose sand and gravel of scheme 5 the support was provided by fully lubricating the line with pressurised bentonite slurry at 50kPa.

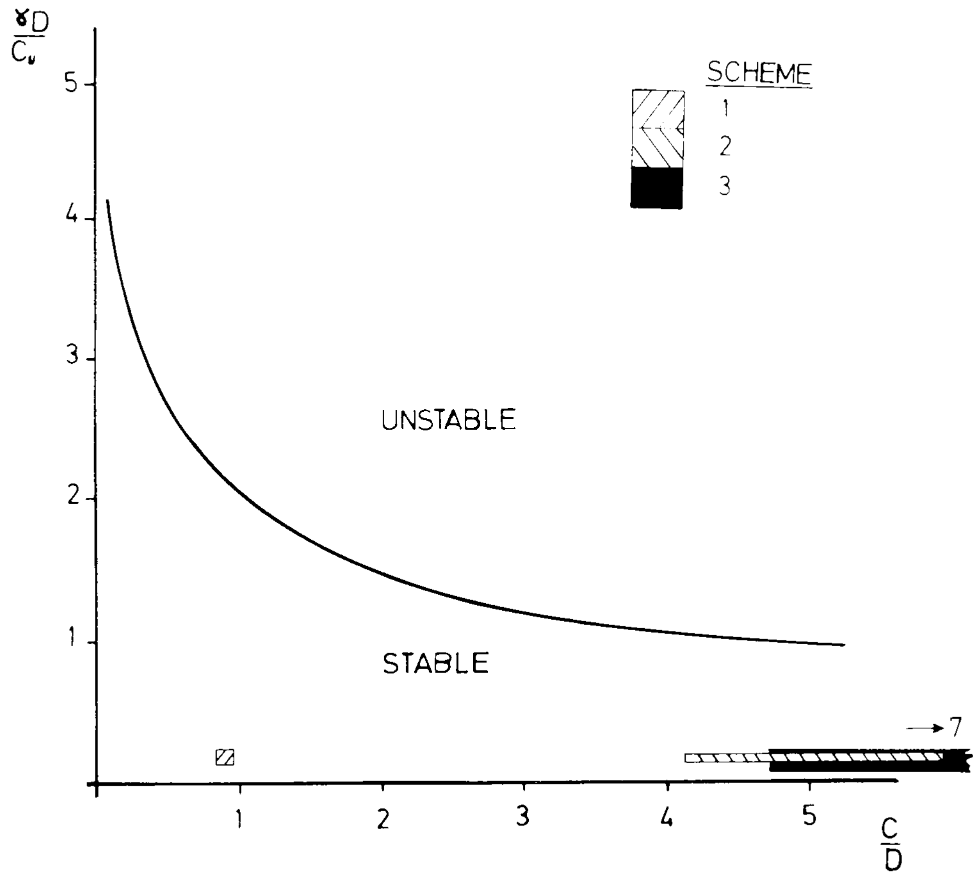


Figure 6.6 Stability of bores.

To improve the very approximate average resistances traditionally adopted in designing pipe jacks it is necessary to gain an understanding of the contact pressures between the pipes and the soil during jacking. The contact stress cells and pore pressure probes in the instrument pipe wall allow such measurements to be made.

6.2.3 Local interface stresses mobilised during jacking

Scheme 1

The contact stress cells throughout scheme 1 illustrate zero drift, Figures 6.7 and 6.8, due to moisture ingress. The instrumented pipe was positioned three pipes behind the shield resulting in the excavation being generally one day old when the pipe passed through. Contact between the pipe and soil was only recorded on the bottom of the pipe throughout the drive which is consistent with the overbreak in stiff soils at low cover depth remaining open and the pipe sliding along the base of the open bore. The total radial stress varied between -120kPa and 750kPa. These values were for extremely short periods with the majority of pushes lying between 0-250kPa. The larger values are probably the result of local cobbles pressing onto the cell. The negative values imply suction during the shearing process and generally correspond to the locations of negative pore pressures recorded during the wet site conditions, Figure 6.9. Shear stresses varied between -40kPa and 480kPa. The negative stress is an isolated value and is possibly due to recoil at the end of a push. The majority of shear stresses were between 0kPa and 150kPa.

The pore pressures during the dry weather conditions are generally smaller than the changes in total stress which suggests that the ground at the interface may be unsaturated even though the original material was saturated and/or the high permeability of the concrete allows a partially drained condition to exist during the jacking process. By contrast the behaviour during the wet conditions indicates closer parity between total stress and pore pressure suggesting undrained behaviour; the material having regained its saturated state and the local drainage path via the pipe interface being greatly diminished by the excess water.

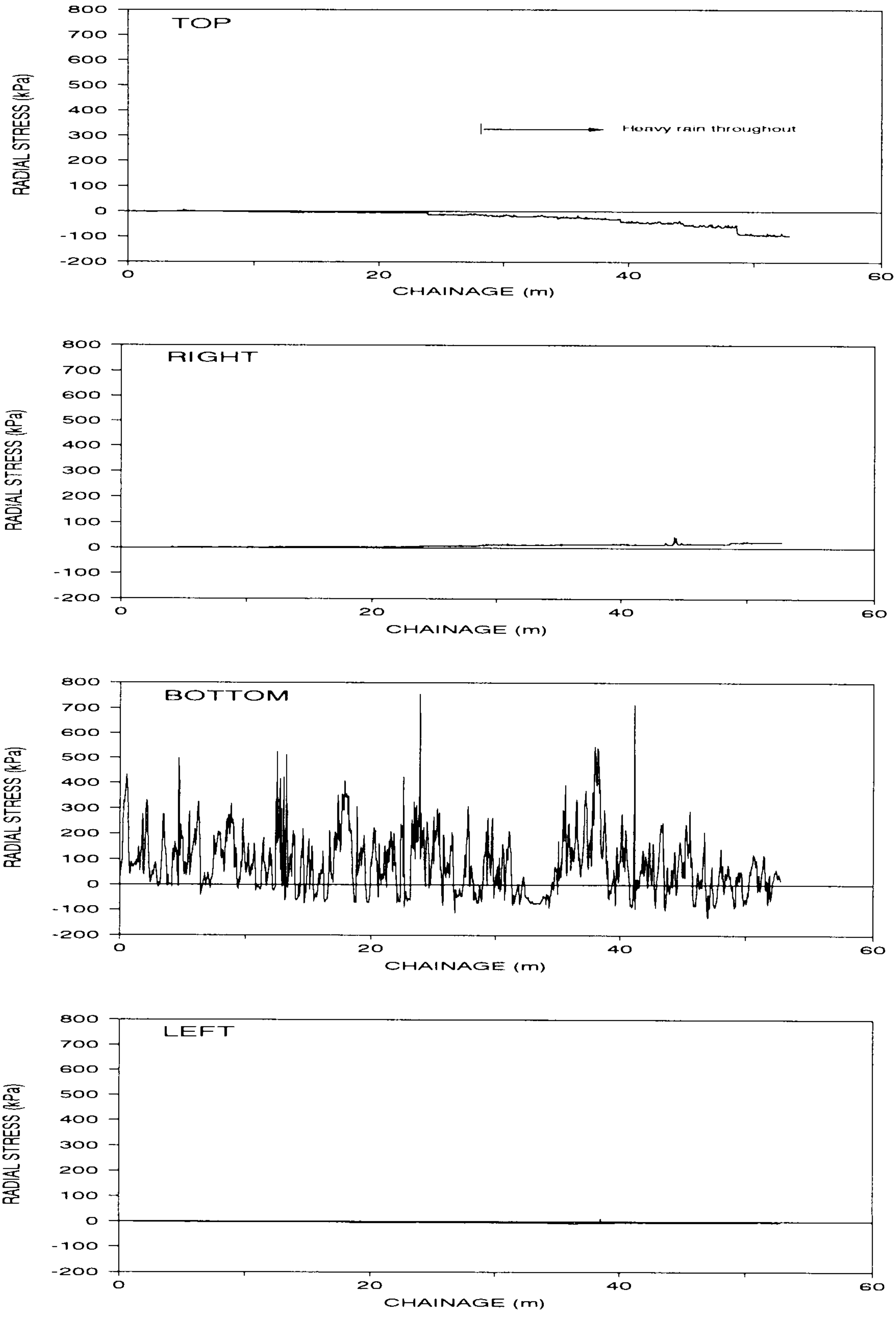


Figure 6.7 Variation in total radial stress during scheme 1.

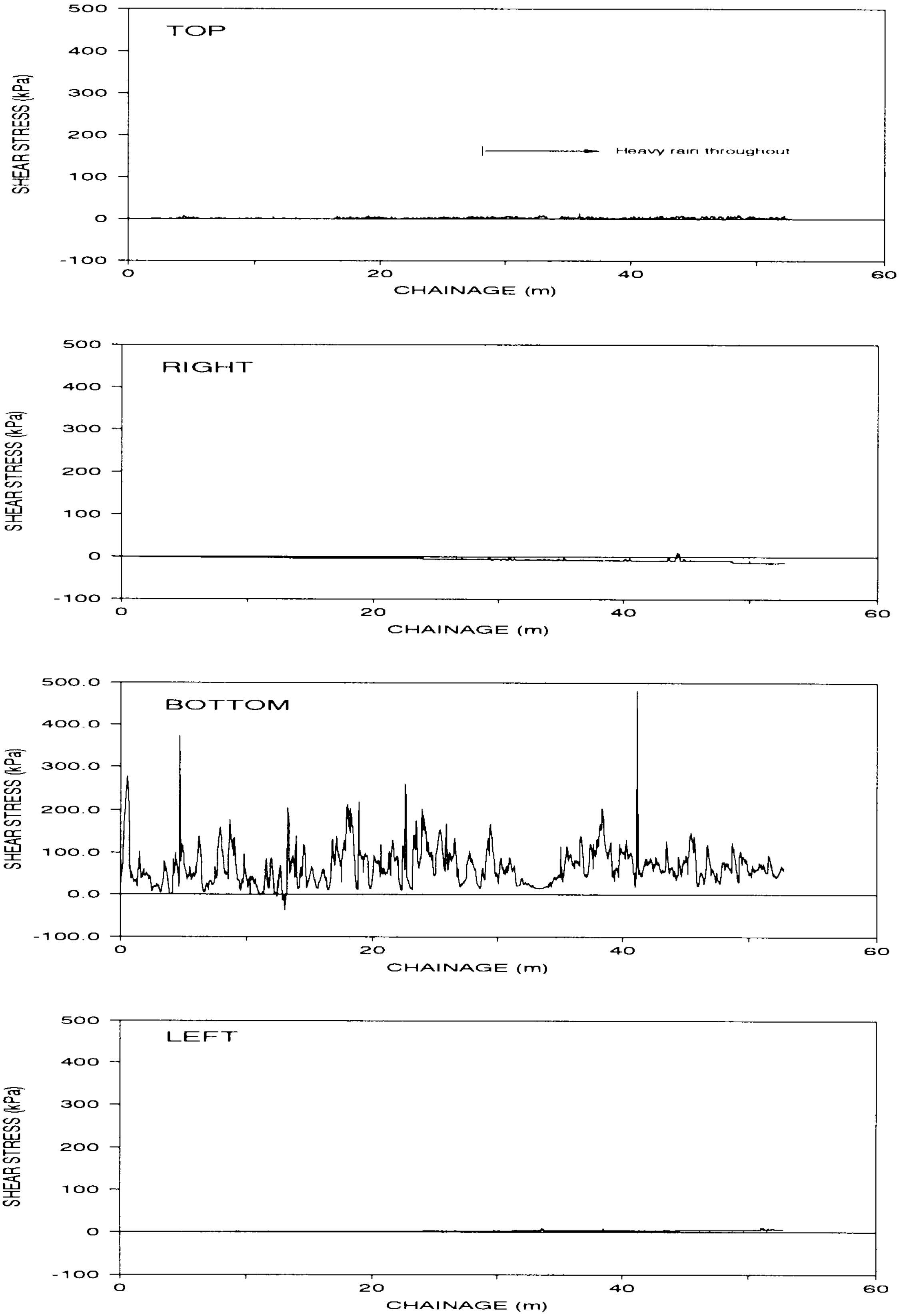


Figure 6.8 Variation in interface shear stress during scheme 1.

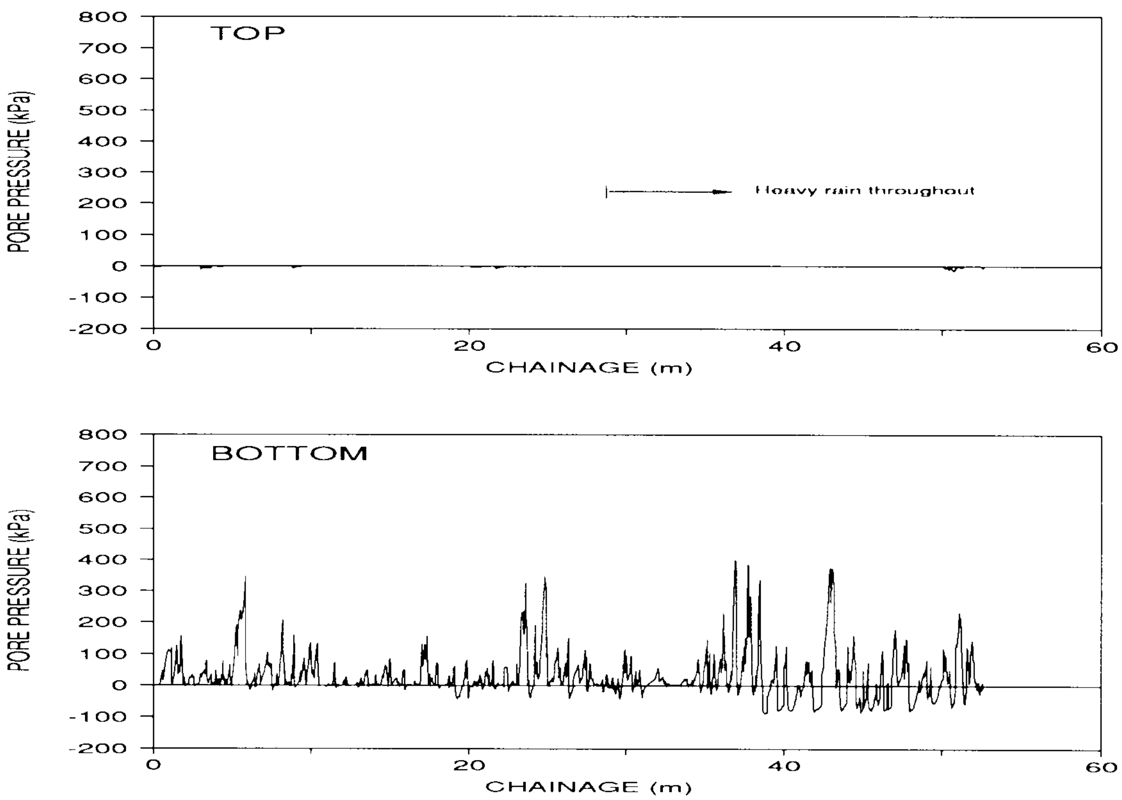


Figure 6.9 Variation in pore water pressure during scheme 1.

Scheme 2

The variations in local interface stresses for scheme 2 are included as Figures 6.10 to 6.12. The anticipated large stand up time of the overbreak led to a reduced instrument set being concentrated in the pipe invert. The instrumented pipe was inserted 22.5m behind the shield, passing through the excavation 16 days after the shield. It appears from the plots that the pipe was subjected to longitudinal rotation with the front contact stress cell attracting significantly larger stresses, prior to it being damaged. Total radial stresses approached 575kPa with corresponding friction stresses of 160kPa. However measurements of the exact positioning of the cells in the cored holes showed that the front cell was 1mm proud of the pipe surface and inclined towards the direction of movement while the rear cell was recessed by the same amount and inclined away from the direction of movement. The stiff nature of the ground has resulted in significant over and under-registration of load. The pore water pressure during pushes indicate suctions of typically 50kPa in the front cell. Large positive pore pressures of short duration were measured in the rear cell, the maximum value approaching 700kPa at chainage 6.1m which was the position were the lead contact stress cell was damaged. The large pressure caused the glue line around the probe to fail. Deep score lines across the contact stress cell suggest that a high spot may have resulted in concentrated loading.

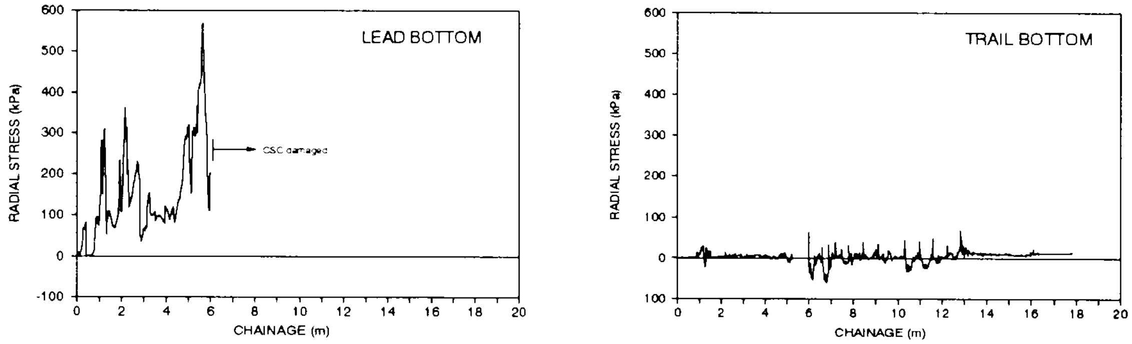


Figure 6.10 Variation in total radial stress during scheme 2.

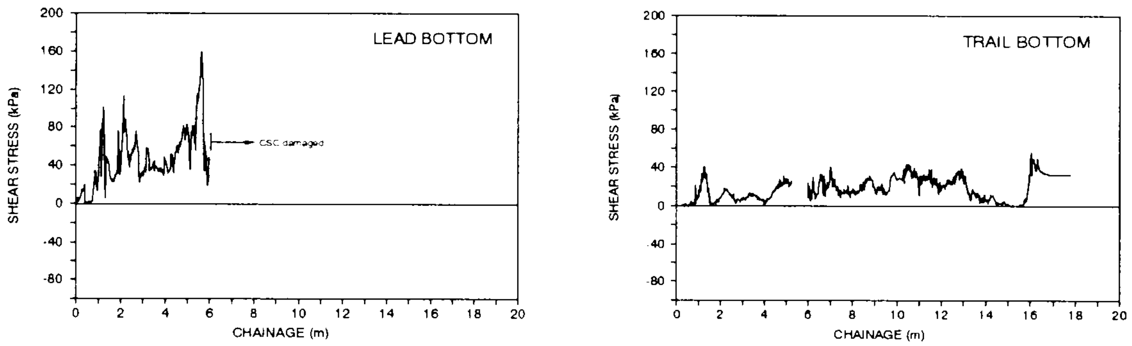


Figure 6.11 Variation in interface shear stress during scheme 2.

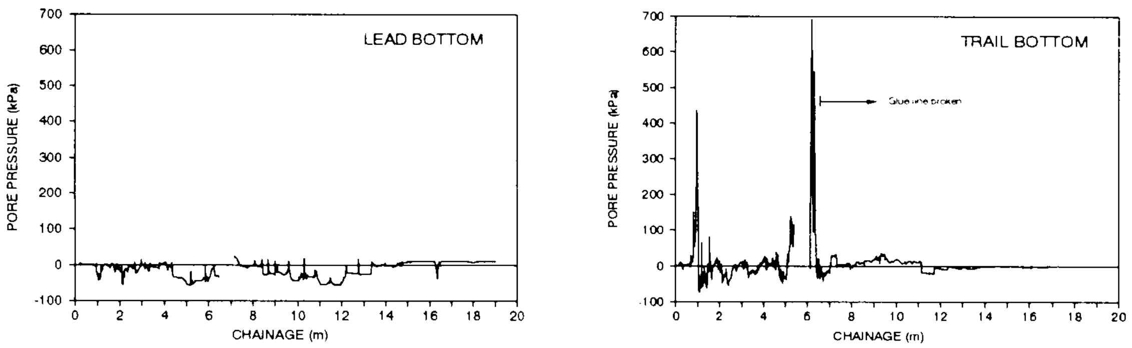


Figure 6.12 Variation in pore water pressure during scheme 2.

Scheme 3

The local interface stresses in the high plasticity clay of scheme 3 are presented in Figures 6.13 to 6.15. The instrumented pipe passed through the initial excavation 9 days after the shield. High lateral ground stresses exacerbated by horizontal deviations of the pipeline, Figure 5.4, were sufficient to break the glue lines on the contact stress cells at tunnel axis. The right cell was displaced under a radial total stress of 490kPa and a shear stress of 110kPa, approximately 1m into the drive. The left cell moved at chainage 4.5m when subjected to a radial total stress of 650kPa and a shear stress of 145kPa. It will be noted that the cells became wedged in the holes at depths of 32mm and 10mm respectively from the pipe outer surface and continued to record radial stresses up to 150kPa and complementary shear stresses up to -30kPa. The response of the bottom contact stress cell was faulty and this would have been picked up during calibration had scheme 3 not followed hard on the heels of scheme 2. Unfortunately the cell was subsequently crushed in service and could not be recalibrated or inspected to establish the reason for its abnormal response. The top cell was displaced by 3mm which probably occurred at chainage 10m. The maximum vertical total radial stress was 450kPa with a peak shear stress of 145kPa. The high failure rate of glue lines on this site was disappointing but certainly saved the earth pressure cells from being crushed. The method of fixity was modified for schemes 4 and 5 to prevent further premature movement.

The pore pressure data when considered together with the total stress data provides extremely limited information on the effective stress behaviour. The apparent dormant response of the top probe suggests that the cable was damaged during installation while the bottom and left axis probes were displaced during the drive. The only reliable data were obtained from the right axis instrument; the large pressures typically 400-900kPa imply that the pipe could be subject to squeezing from the sides. It is apparent from the plots that zero pore pressures were recorded during the first 3m of the drive. Visual observations suggest that the surface layer of the clay around the tunnel entrance may have dried out.

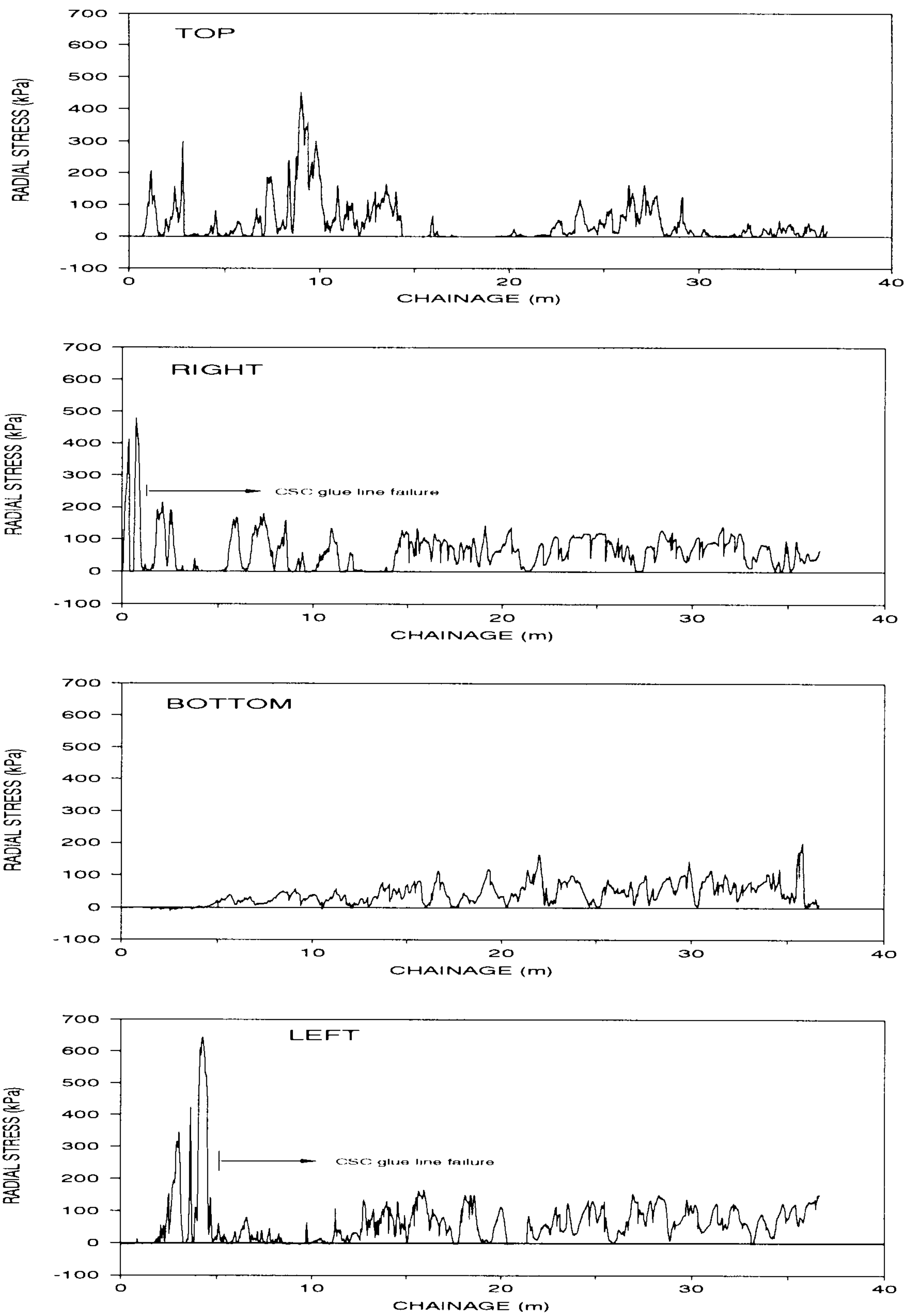


Figure 6.13 Variation in total radial stress during scheme 3.

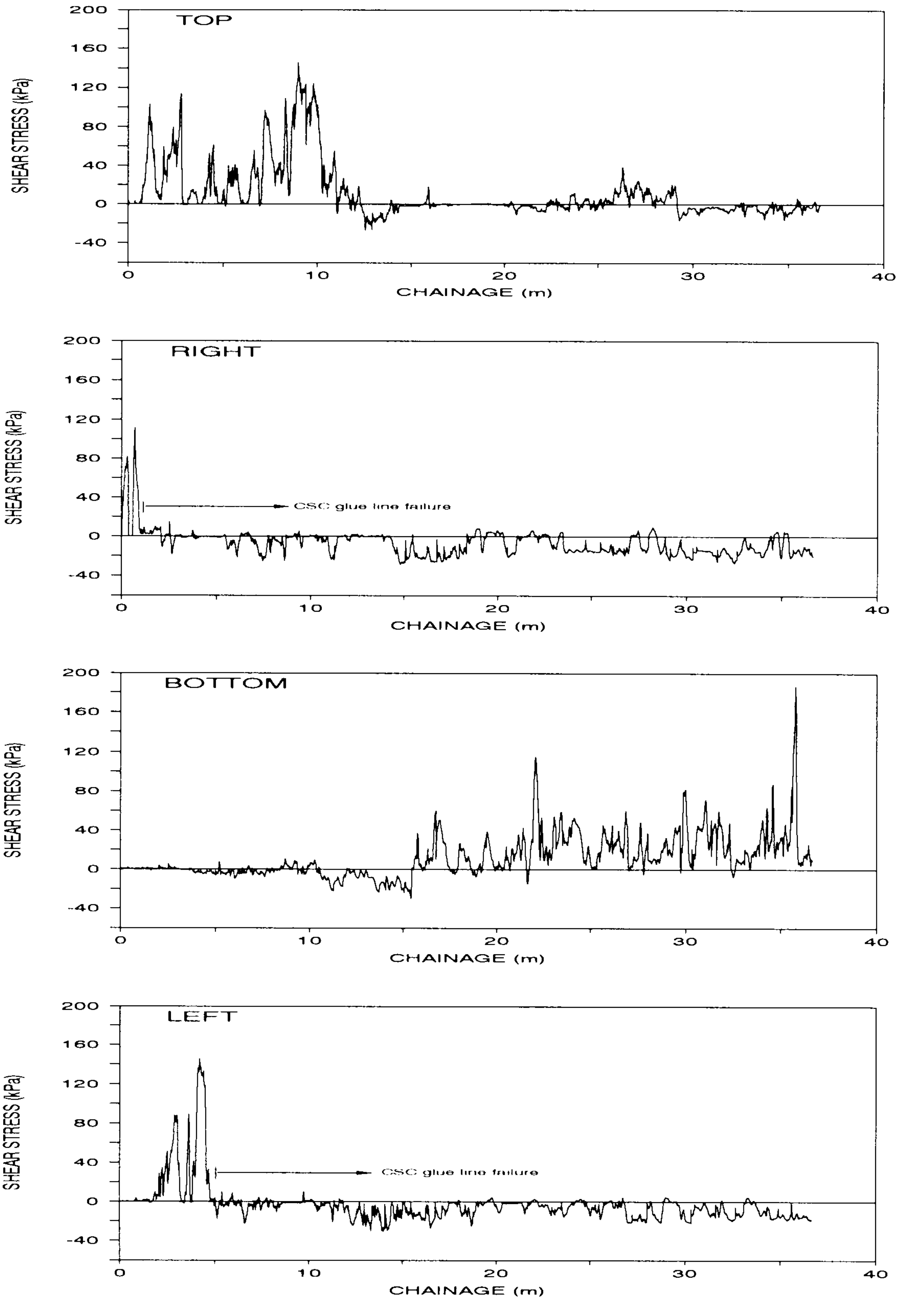


Figure 6.14 Variation in interface shear stress during scheme 3.

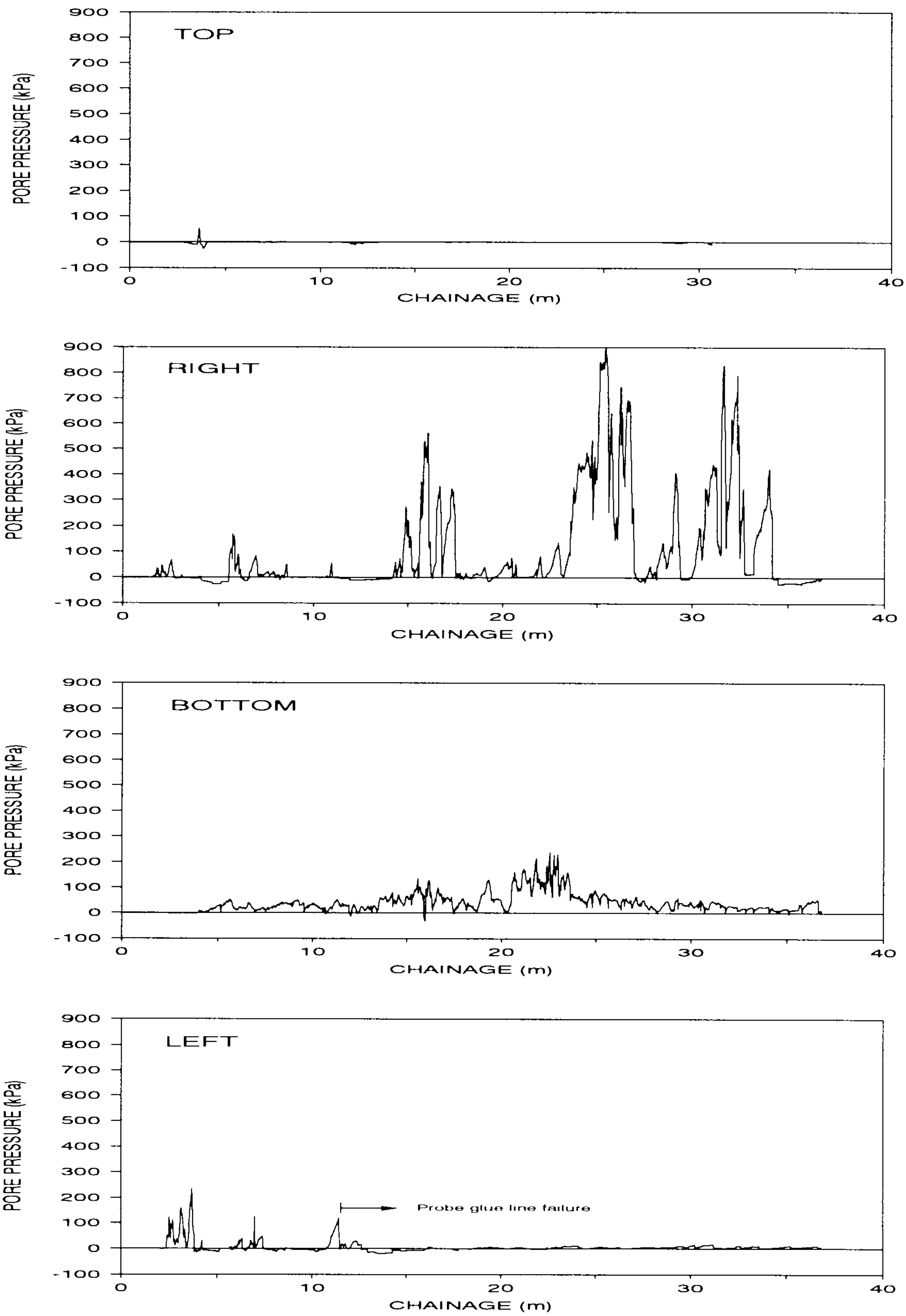


Figure 6.15 Variation in pore water pressure during scheme 3.

Scheme 4

Figures 6.16 to 6.18 illustrate the data recorded during scheme 4. The major stresses occurred on the base of the pipe although limited collapse of the silty sand around the pipe resulted in misalignment induced stresses being recorded at the crown and axis positions. The plots clearly show that peak values of total radial and interface shear stresses were obtained over short lengths of the drive with a lower overall average for the total length. The peak base values reached 300kPa and 160kPa respectively. The stresses at pipe axis and crown were generally smaller peaking at 150kPa and 80kPa. Closer examination of the plots indicates that the positions of peak stresses along the base agree well with the positions of maximum angular misalignment. Variations in ground conditions may also alter the interface stresses along the tunnel length. The detailed tunnel face log of Figure 4.8 indicates that the ground type at the base of the bore was predominently silty sand. The presence of a boulder clay band between chainages 58m-61m appears to have caused isolated peak radial stresses in the top and left hand contact stress cells. The alignment profiles of Figure 5.5 indicate that the pipe was passing through a vertical trough with the position of maximum deviation at chainage 60m. The stiffer response of the boulder clay compared to the loose sand has allowed the generation of large radial interface stresses. The partial lubrication over this section of tunnel appears to have reduced the ratio of shear stress to radial stress. Further details of misalignment and lubrication effects are covered in sections 6.4.1 and 6.4.3. Pore pressures along the base of the excavation were peaky reaching a maximum of 190kPa. Pore pressures in the disturbed material elsewhere around the pipe were small. The injection of lubricant into the void does not appear to have had a marked effect on the radial and shear stress profiles for the majority of the drive. The absence of large pore pressures along the base of the bore after lubricant injection suggests that the probe may have gone down.

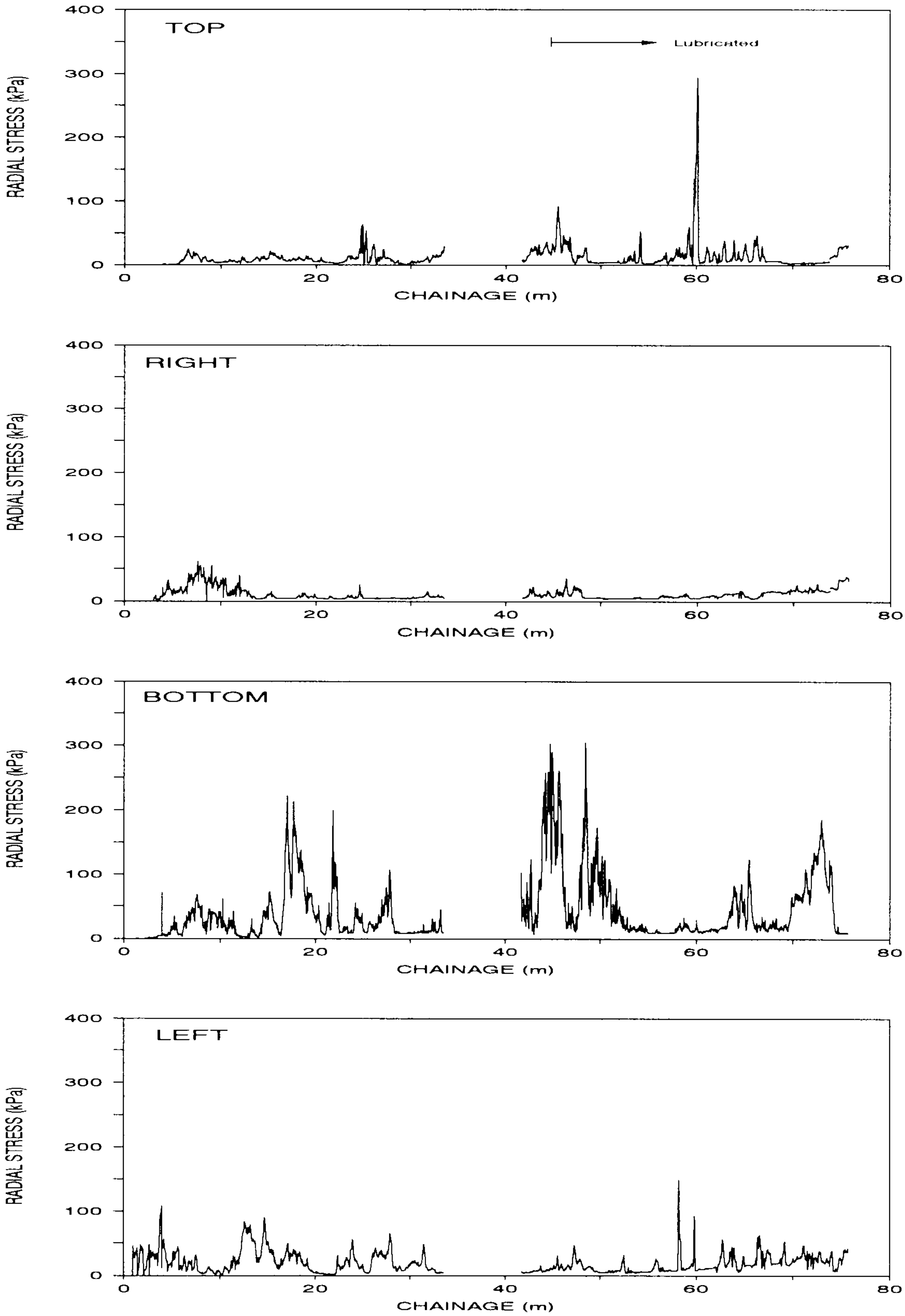


Figure 6.16 Variation in total radial stress during scheme 4.

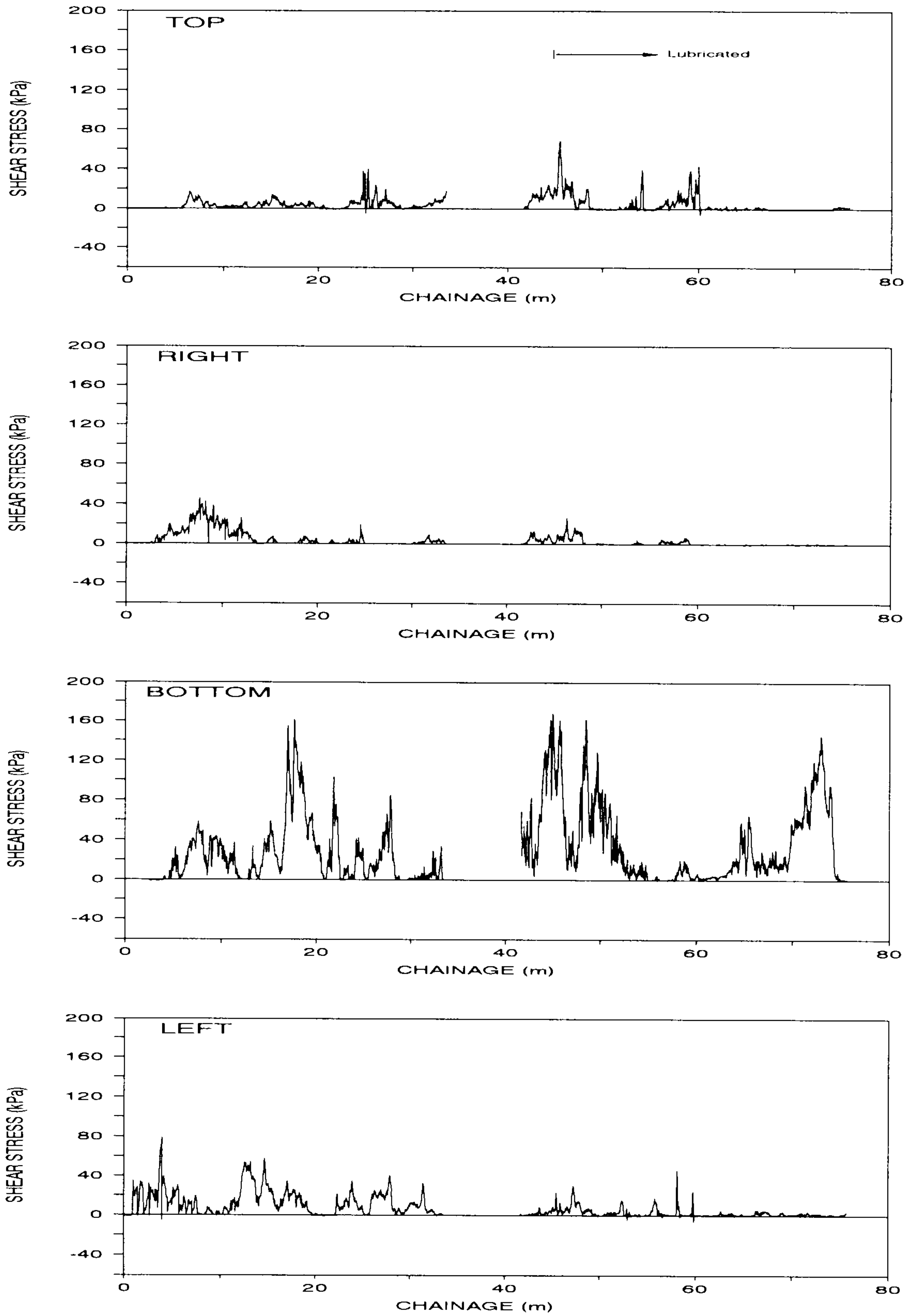


Figure 6.17 Variation in interface shear stress during scheme 4.

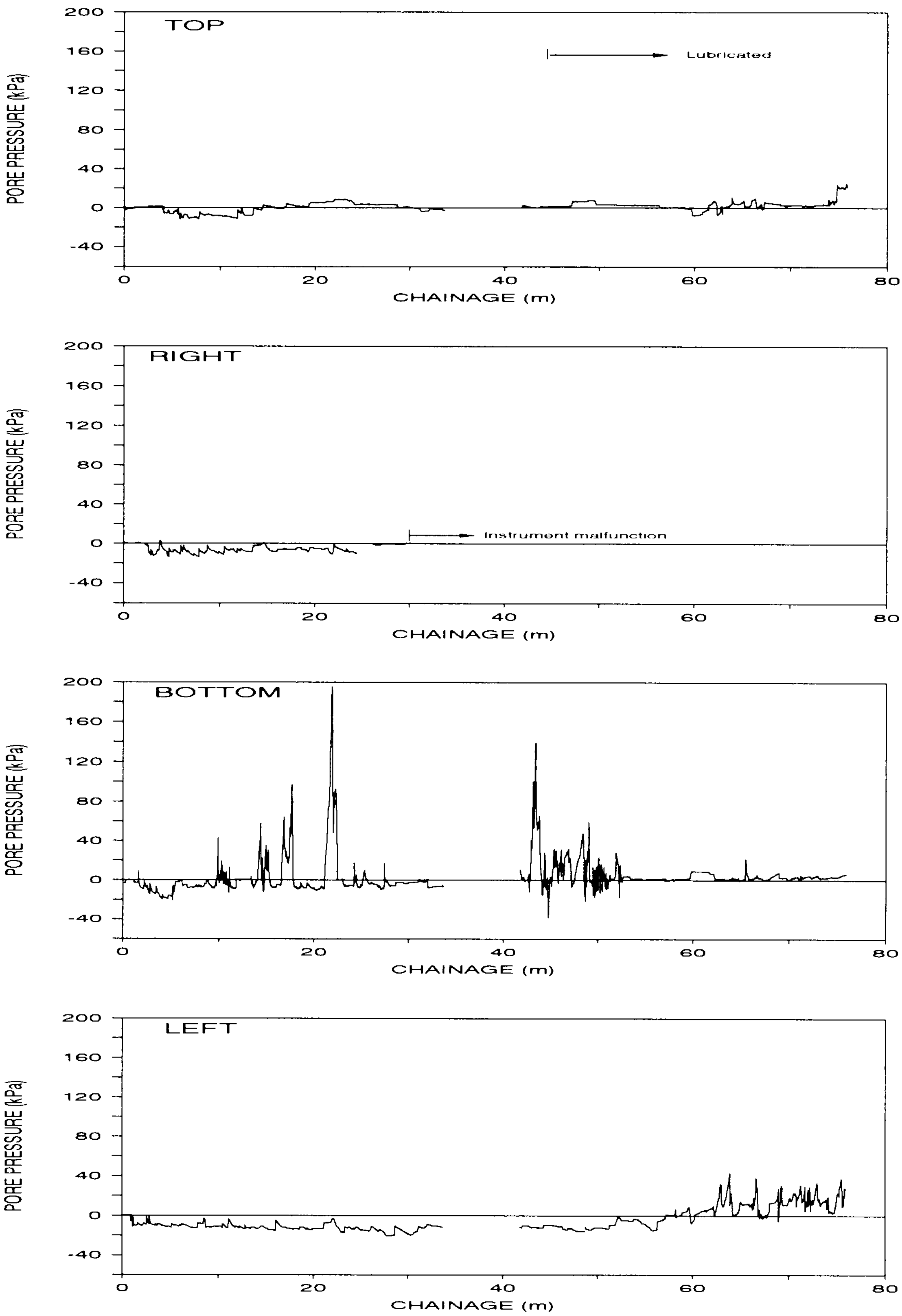
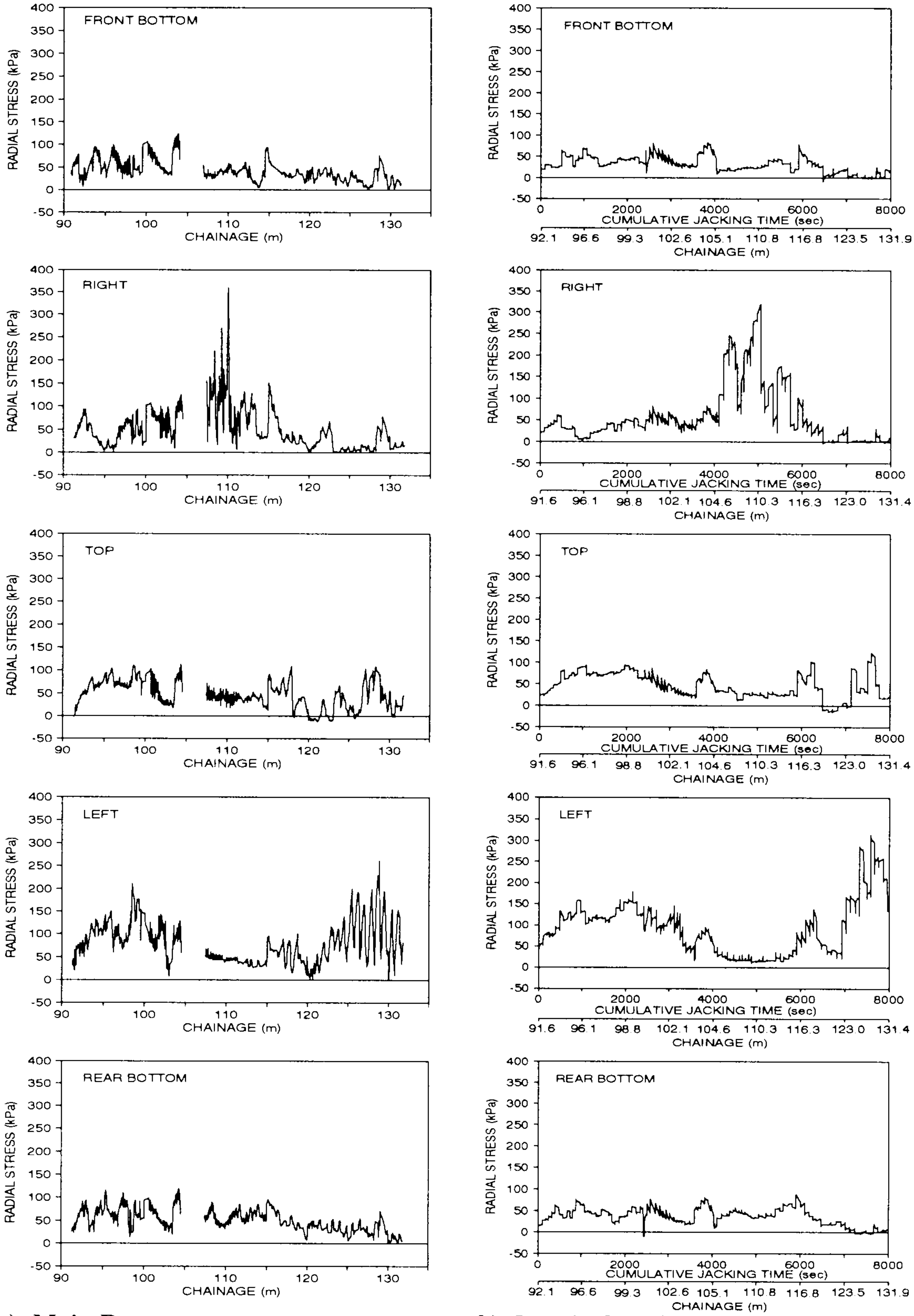


Figure 6.18 Variation in pore water pressure during scheme 4.

Scheme 5

The interface stresses along the length of tunnel containing the severe pipeline misalignments and pipe damage, chainage 90m to 132m of Figure 5.6, are presented in Figures 6.19 to 6.21. Stresses were mobilised when the whole pipe string was jacked from the thrust pit and also when the interjack, which was positioned one pipe in front of the instrumented pipe, was activated. An immediate conclusion that can be drawn from the plots is that stresses appear to be independent of the location of thrust. This is because the total load passing through the pipe, given the close proximity of the interjack, must be similar and tunnel alignment is clearly dictating the interface stress profiles. Maximum total radial stresses of 300-350kPa were measured at the tunnel axis with the average stress fluctuating about the 50kPa value. Pore pressures were generally very similar suggesting that pressurised bentonite slurry had formed a stabilised zone of soil around the pipe preventing the soil from developing large effective contact stresses. The effectiveness of the layer of bentonite gel between the pipe and soil is demonstrated in Figure 6.20 with shear stresses typically below 5kPa except at the positions of maximum misalignment, chainages 110m and 130m where stresses peaked at 15kPa. These values are supported by the average pipeline shear stress value of 3.6kPa, Table 6.1, obtained from the jacking record. Surprisingly there is no clear indication of a reversal of shear stress when activating the interjack. It is not known whether this is a function of the movement of pressurised fluid from the high pressure zones. The suggested buoyancy of the pipeline based on tunnel surveys, Section 5.2.1, is substantiated by the slightly larger total radial and shear stress values at the top of the pipe.



a) Main Rams **b) Interjack activated**

Figure 6.19 Variation in total radial stress (ch. 91.3m to 131.8m) on scheme 5.

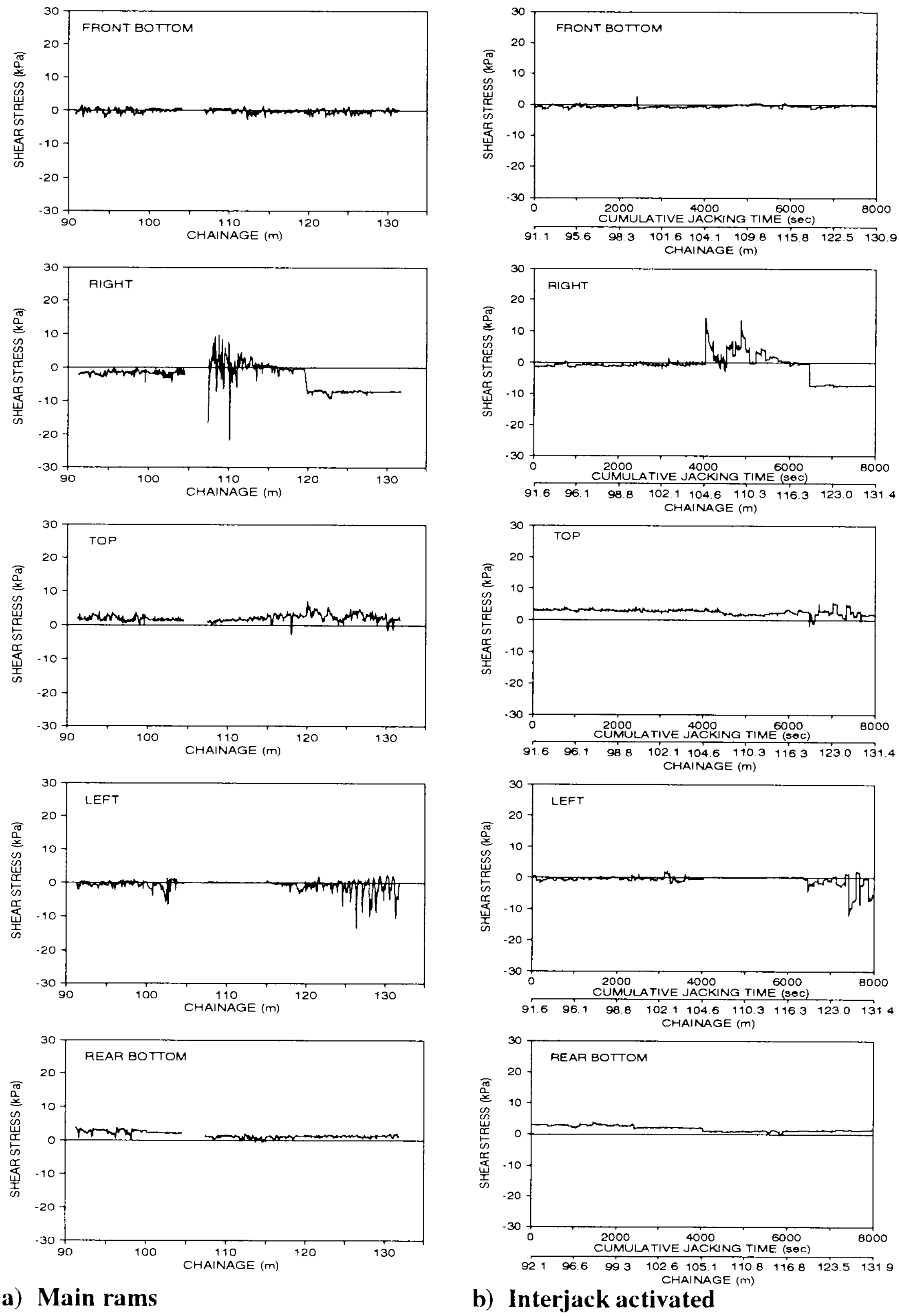
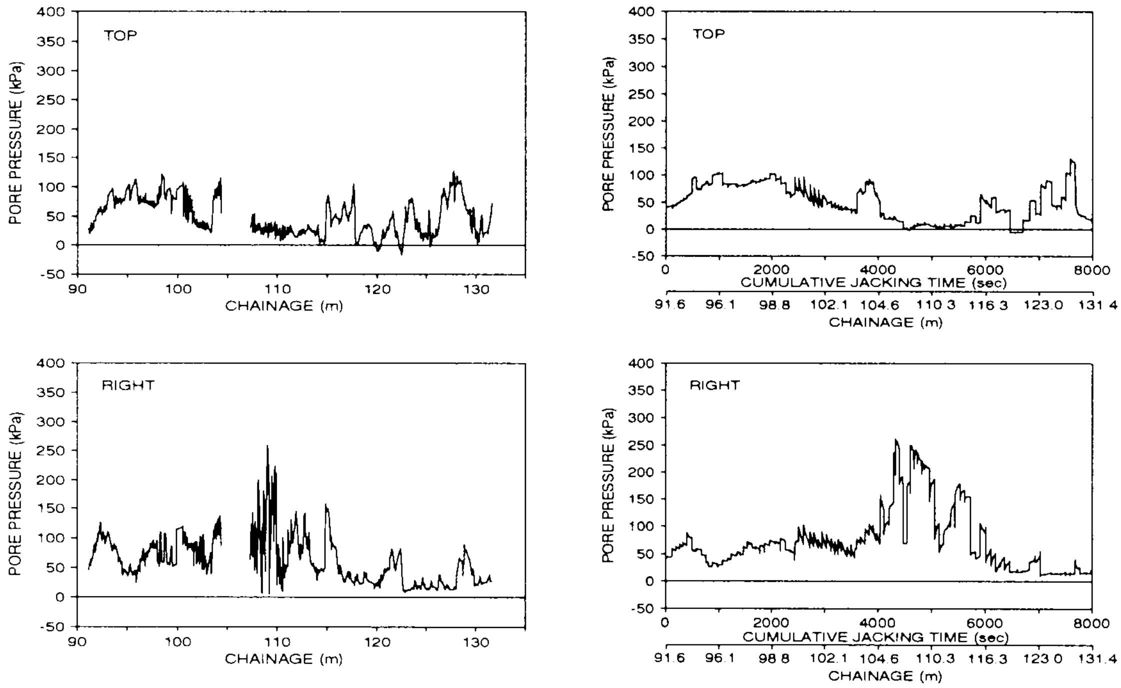


Figure 6.20 Variation in interface shear stress (ch. 91.3m to 131.8m) on scheme 5.



a) Main rams **b) Interjack activated**

Figure 6.21 Variation in pore water pressure (ch. 91.3m to 131.8m) on scheme 5.

A summary of the peak interface stresses has been reproduced in Table 6.2. The major contact stresses in the non-lubricated drives were mobilised at the bottom of the pipes, apart from scheme 3 through heavily overconsolidated London clay during which lateral stresses up to 650kPa were sufficient to damage the instruments on the tunnel axis. In the fully lubricated drive of scheme 5 the pressurised lubricant has caused the pipeline to float resulting in similar values at the top and bottom of the pipes. The various plots clearly show that peak values of radial, frictional and pore water pressures were obtained over short lengths, 1m-2m, of the drives with a lower overall average value for the total length. The peak skin friction values are nearly two orders of magnitude larger than the average friction values in Table 2.2. The generation of significant positive pore pressures during jacking is also demonstrated. To provide some feel for the magnitude of radial stresses the values are compared with full overburden loading, short term Terzaghi loading and elastic pipe self weight stresses based on the formula of Roark and Young (1976) for the contact width between a solid elastic cylinder resting in a cylindrical cavity:

$$b = 1.6(Pu*kd*Ce)^{1/2}$$

where
Pu = contact force per unit length
$kd = D_1*D_2/(D_1-D_2)$
D_1 = internal diameter of the cavity
D_2 = external diameter of the pipe
$Ce = (1-n_1^2)/E_1+(1-n_2^2)/E_2$
E_1 = elastic modulus of the soil
E_2 = elastic modulus of the concrete pipe
n = Poisson's ratios as for E

Scheme	γ	h	E_1	E_2	n_1	n_2	P_u	D_1	D_2
	(kN/m^3)	(m)	(MPa)	(GPa)			(kN/m)	(m)	(m)
1	22	1.5	48	40	0.2	0.2	17.7	1.554	1.530
2	22	11	120	40	0.2	0.2	23.0	1.724	1.700
3	21	16av	96	40	0.2	0.2	35.3	2.304	2.280
4	18	7	144	40	0.2	0.2	24.2	1.804	1.780
5	19	6	144	40	0.2	0.2	-2.2*	1.450	1.430

Properties used to determine radial stress predictions shown below

Scheme	Australian CPA based on Terzaghi loading (kPa)	Full overburden (γh) (kPa)	Pipe Self Weight (kPa)	Radial (kPa)			Shear (kPa)			Pore Pressure (kPa)		
				Top	axis	bottom	Top	axis	bottom	Top	axis	bottom
1	0	33	59	5	10	550	5	10	250	-5	-	400
2	0	242	96	-	-	550	-	-	160	-	-	700
3	0	336	79	450	650	-	150	150	-	-	250	250
4	88	126	102	100	100	300	60	60	60	10	10	190
5	76	114	-42*	110	350	110	5	15	4	120	260	-

Notes: * Net bouyancy force
- No instrument reading

Table 6.2 Maximum local interface stresses

Comparison of the theoretical and measured values demonstrates that induced earth loading during jacking can be larger than the most severe design case of full overburden, which may have implications for pipe design. Adopting full overburden for predicting jacking resistance would however seriously over estimate the required jacking capacity. The more generally accepted Terzaghi loading which is the basis of most European design approaches results in much smaller short term load intensities which approximate more closely to the average radial loading, but significantly underestimate the peak radial loading caused by pipeline misalignment. In adopting the Terzaghi approach and using short term material properties many pipe jacks through cohesive ground, except in very soft clays, invariably indicate negative radial loading, i.e. self supporting, which is consistent with the stability checks in section 6.2.2. In such cases pipe self weight contact stresses may be an appropriate estimate of radial loading. Contact has been assumed to occur on the bottom of the pipes in schemes 1 to 4, although it is suspected that the pipes in scheme 3 were subjected to horizontal squeezing. In scheme 5 contact was assumed to occur at the crown due to pipe buoyancy. The local interface plots vary above and below the self weight values suggesting that the local variation in the excavated surface and the changes in pipe end load positions due to pipeline misalignment result in uplift or down thrust of the pipes. It will be seen in section 6.3.2 that the pipe self weight model in a well controlled drive can provide a good approximation to total jacking resistance.

6.2.4 Detailed response of individual pushes

The compressed nature of the interface stress plots makes it difficult to evaluate the ground behaviour during a single push. Typical pushes have been selected from the first four schemes and are presented in Figures 6.22 to 6.30. A number of observations can be made:

a) There is excellent agreement between the responses of radial, shear and pore pressure profiles during pushes providing increased confidence in the validity of the readings.

b) Stresses vary rapidly over short distances of less than 300mm and are probably a function of local variations in excavation profile. The drives through cohesive ground exhibit rapid peak and trough (typically zero stress) responses while a much smoother fluctuation is apparent in the non-cohesive material.

c) Stress path plots indicate frictional relationships for the non-cohesive drives of schemes 2 and 4. Surprisingly the total radial stress plots for the London clay at pipe axis also suggest a friction material response (the effective stress plots could not be constructed). The stress paths for scheme 1 are however more erratic which may be a function of the high stone content of the boulder clay. During the dry weather a frictional response is demonstrated with an apparent change to an undrained constant volume behaviour after the period of tunnel inundation.

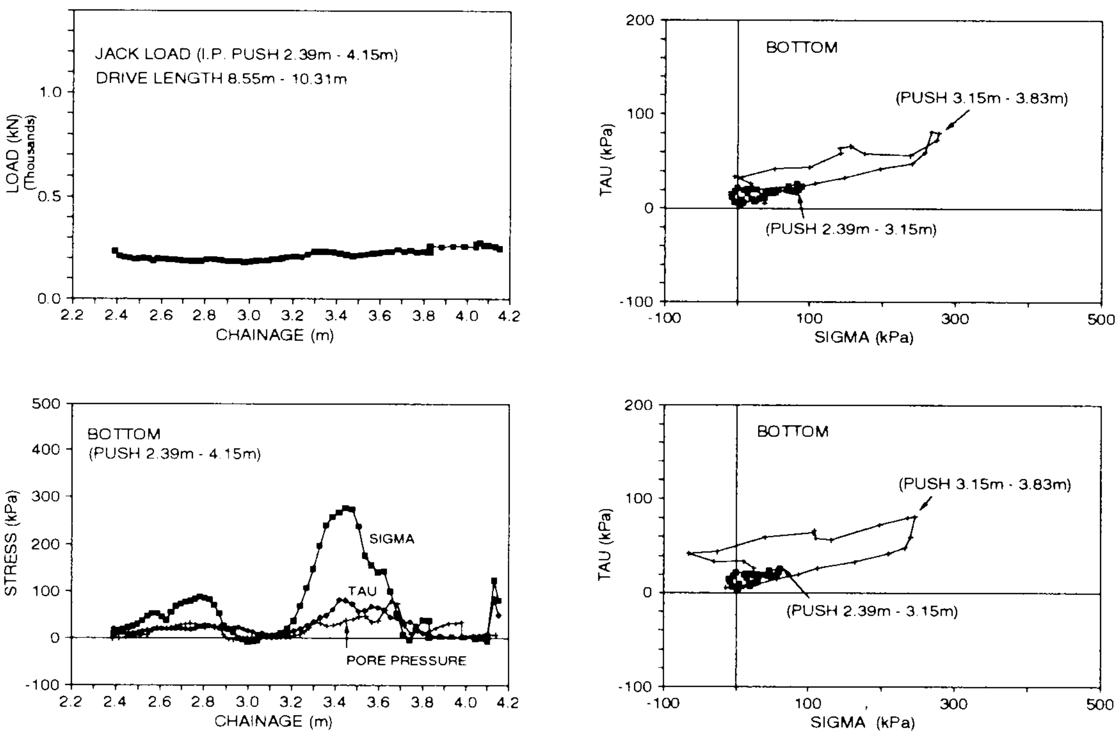


Figure 6.22 Detailed response of pushes between chainages 2.39m and 4.15m on scheme 1.

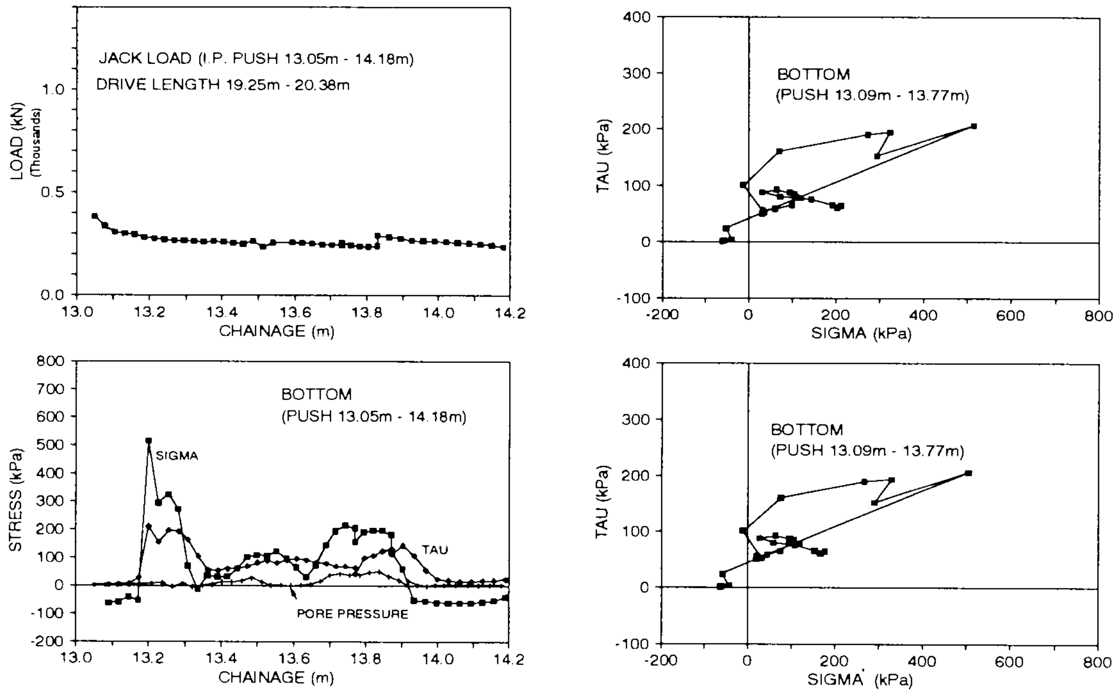


Figure 6.23 Detailed response of pushes between chainages 13.09m and 14.22m on scheme 1.

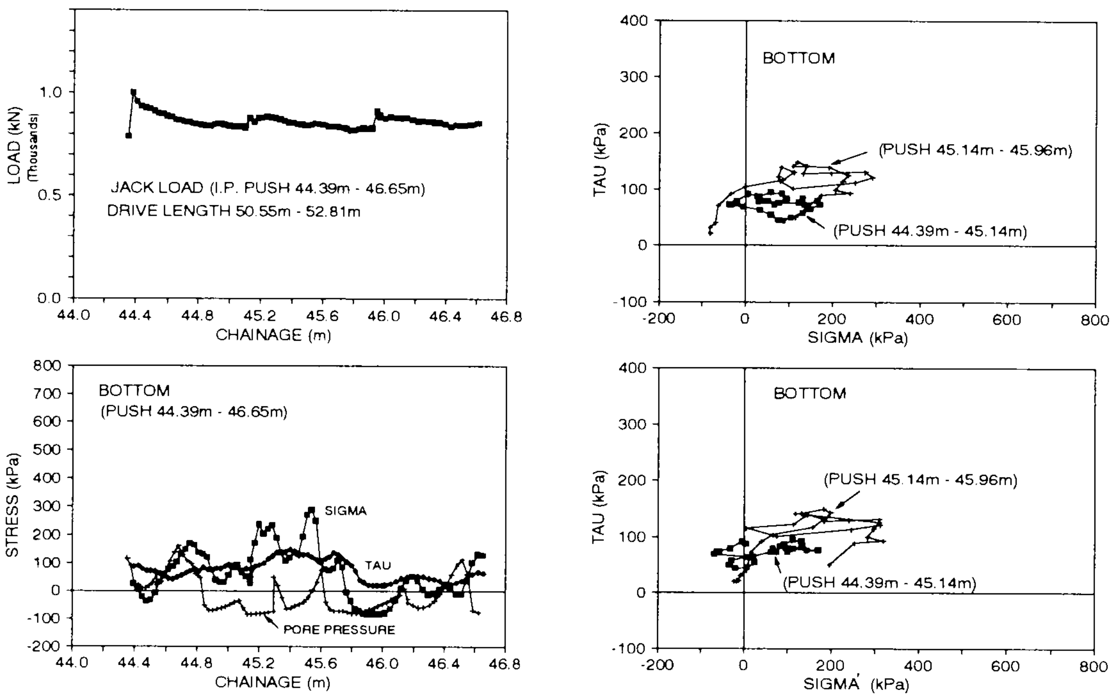


Figure 6.24 Detailed response of pushes between chainages 44.39m and 46.65m on scheme 1.

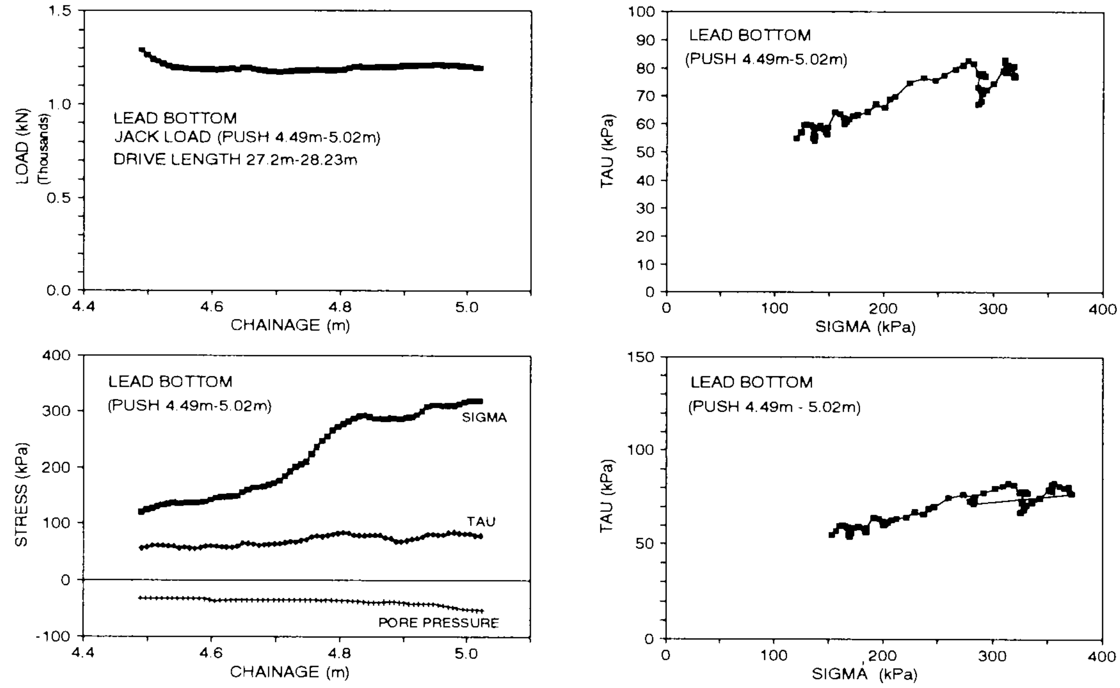


Figure 6.25 Detailed response of pushes between chainages 4.49m and 5.02m on scheme 2.

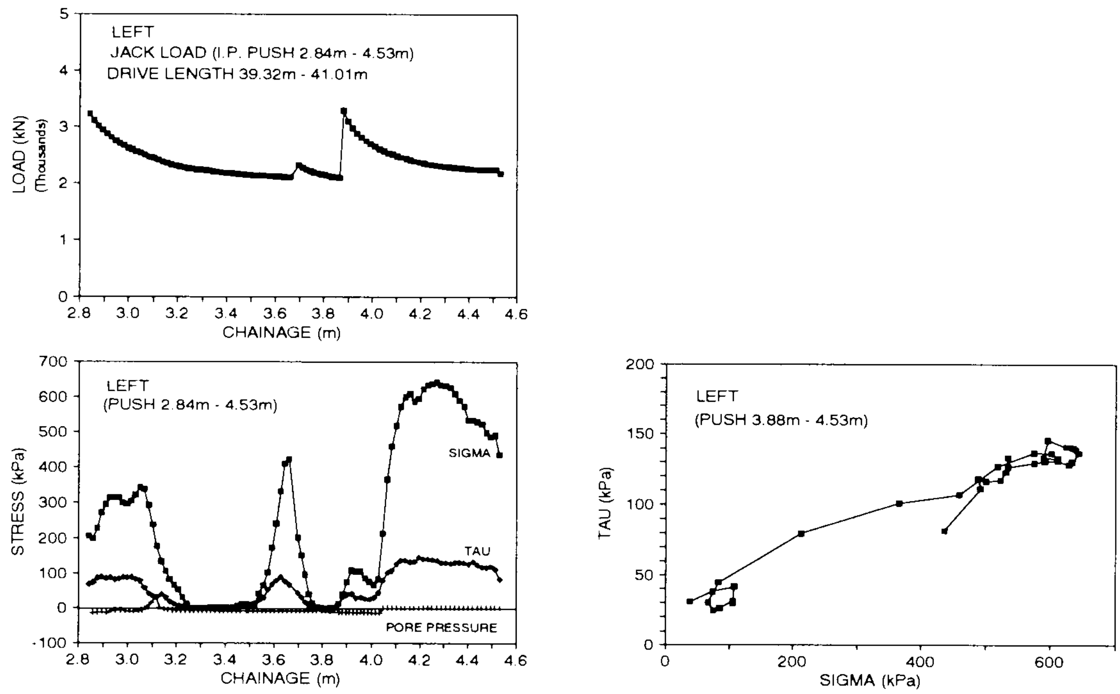


Figure 6.26 Detailed response of pushes between chainages 2.84m and 4.53m on scheme 3.

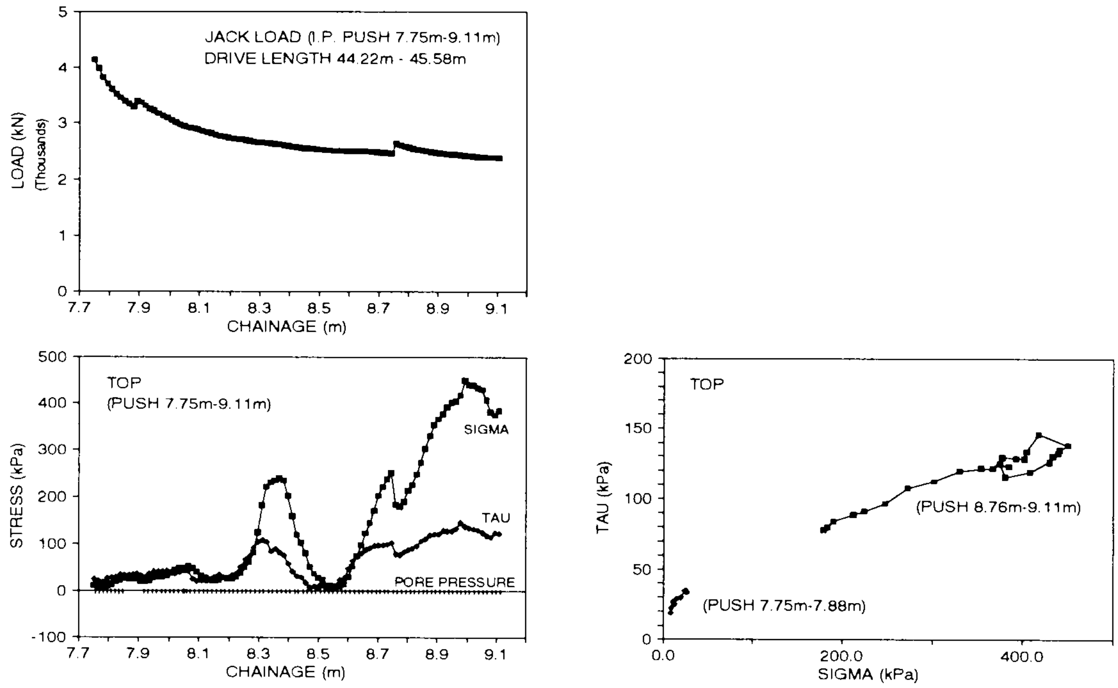


Figure 6.27 Detailed response of pushes between chainages 7.75m and 9.11m on scheme 3.

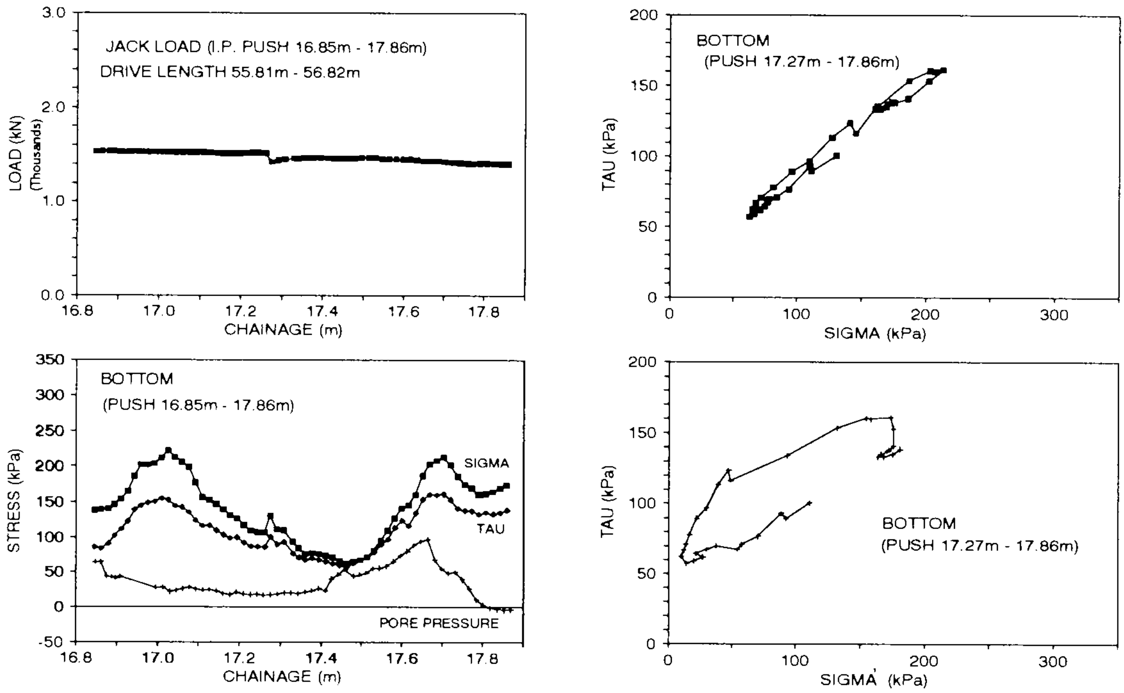


Figure 6.28 Detailed response of pushes between chainages 16.85m and 17.86m on scheme 4.

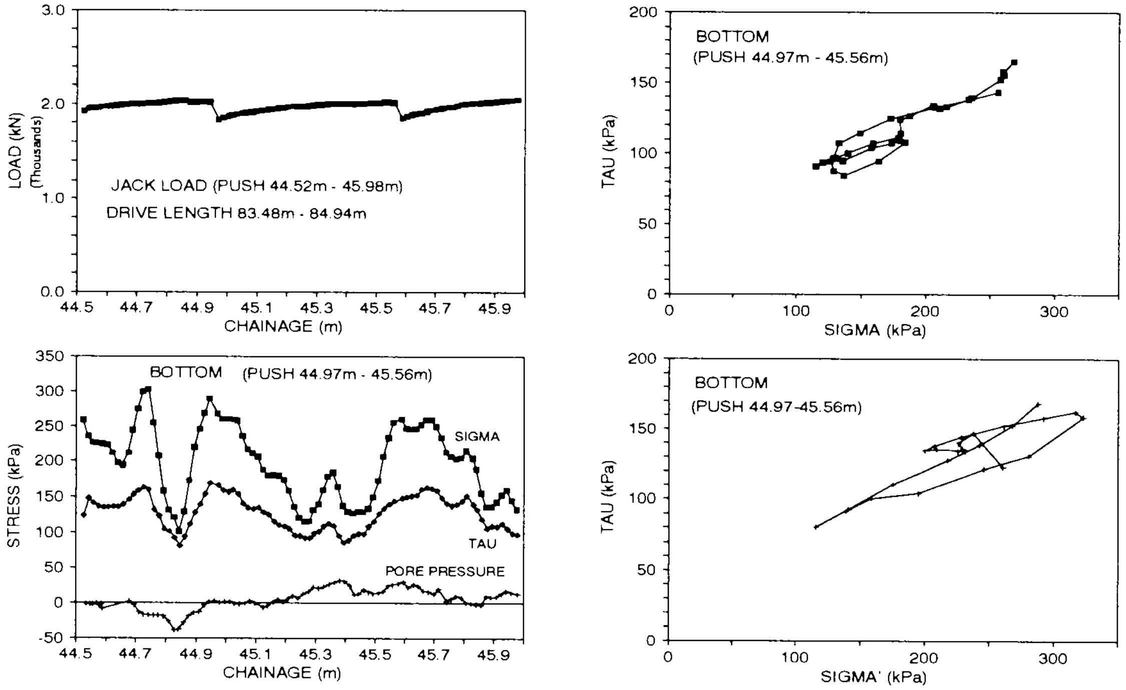


Figure 6.29 Detailed response of pushes between chainages 44.97m and 45.56m on scheme 4.

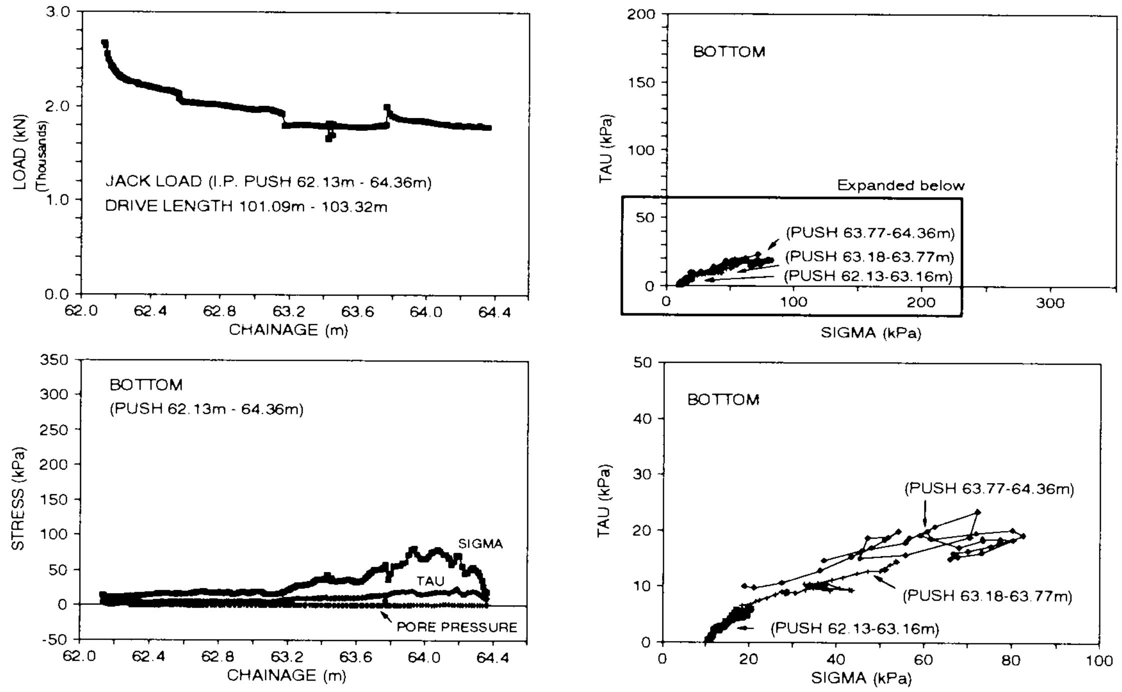


Figure 6.30 Detailed response of pushes between chainages 62.13m and 64.36m on scheme 4.

6.3 Ground related factors

6.3.1 Soil-pipe friction coefficients

To establish whether there is a fundamental difference in interface behaviour between cohesive and non-cohesive soils plots of shear stress against total and effective radial stress during jacking have been compiled for all pushes on schemes 1 to 4 and part of scheme 5.

The responses for scheme 1 are included as Figure 6.31 and relate to the bottom of the pipe. Considerable scatter is illustrated, although best line fits to the plots indicate only small differences between the total and effective stress behaviour. This suggests that the material at the interface may be unsaturated and/or a partially drained state exists at the pipe-soil interface with the concrete pipe acting as a local drainage path. Such a draining effect has been recorded during interface shear box tests between concrete and clay, Potyondy (1961), where reductions in moisture content of 7-10% were measured in the soil adjacent to the concrete. The resulting angle of skin friction during the dry weather δ' was 20.3°. The adhesion intercepts are probably unreliable due to zero drift in the instruments.

The increased moisture content of the ground and in particular within the interface zone during the latter part of the drive has resulted in a reduced angle of skin friction $\delta'$of 13.6°. This observation is supported by the laboratory interface tests of Potyondy which measured skin friction between concrete and a cohesive granular material similar to a glacial clay with three different moisture contents, Figure 6.32. It is however plausible that the reduction in shear strength of the clay caused by the increased moisture content has changed the mode of shearing with the skin friction being independent of the radial loading. The undrained constant volume model of Haslem (1986) would seem appropriate for this case.

The response for the rock drive of scheme 2 is included as Figure 6.33. A linear relationship is indicated producing an apparent skin friction angle δ of 17°.

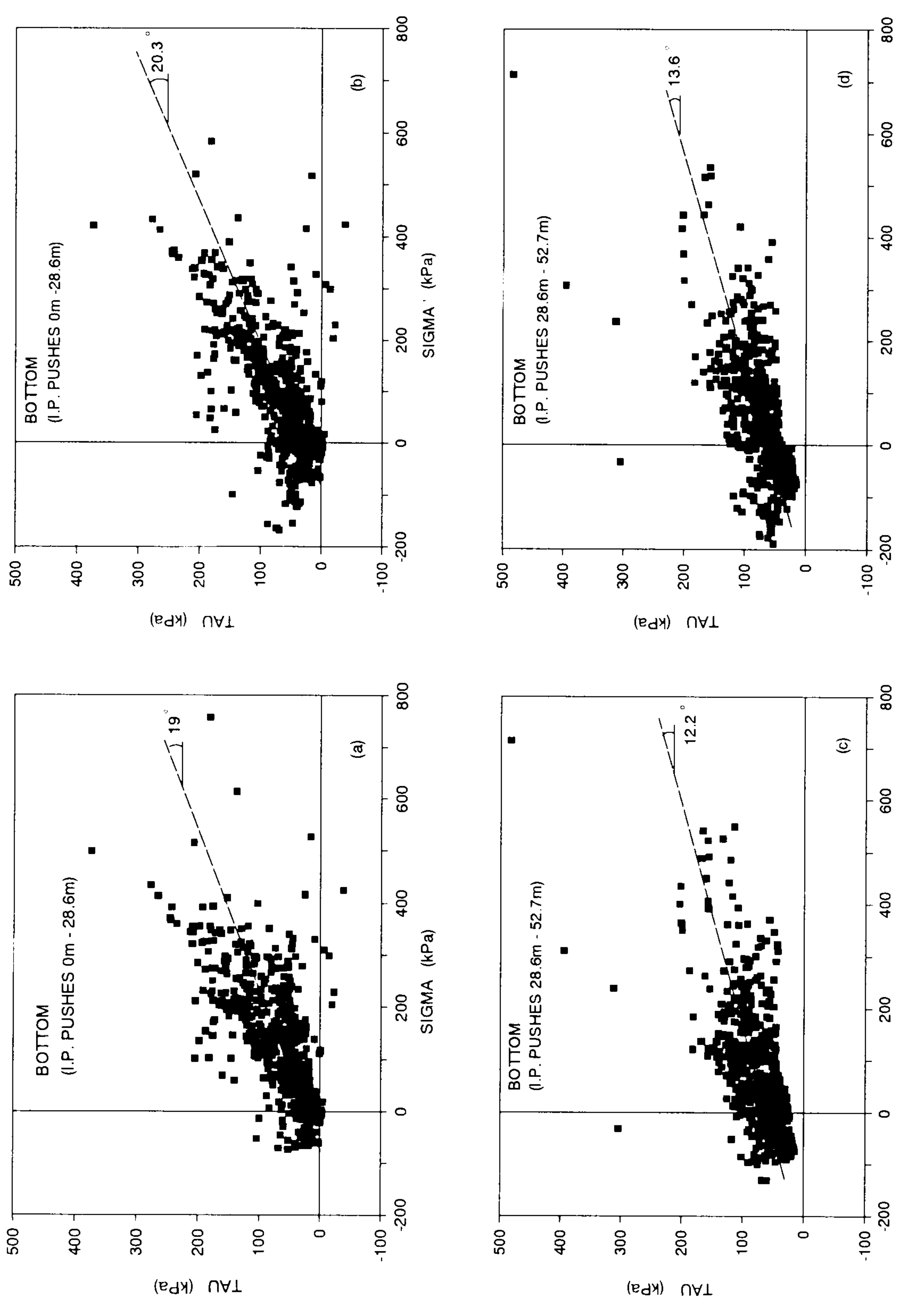


Figure 6.31 Shear stress/radial stress relationships during scheme 1; total and effective stress responses prior to heavy rain (a & b), and after wetting (c & d).

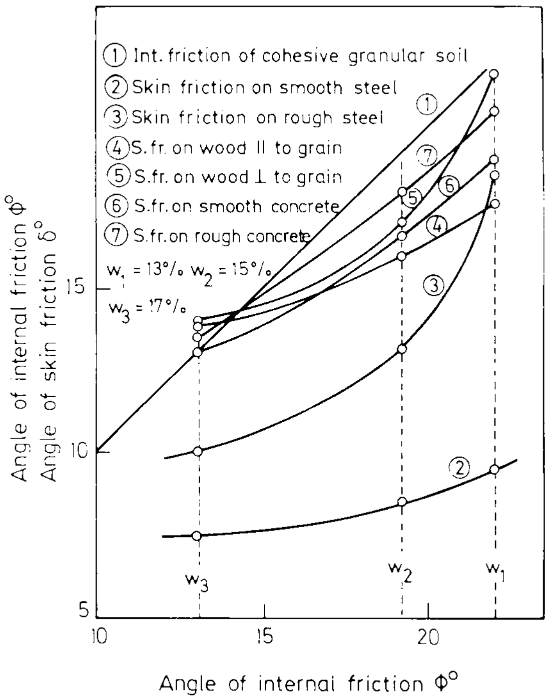


Figure 6.32 Effect of moisture content on δ/φ ratio of cohesive granular soil.

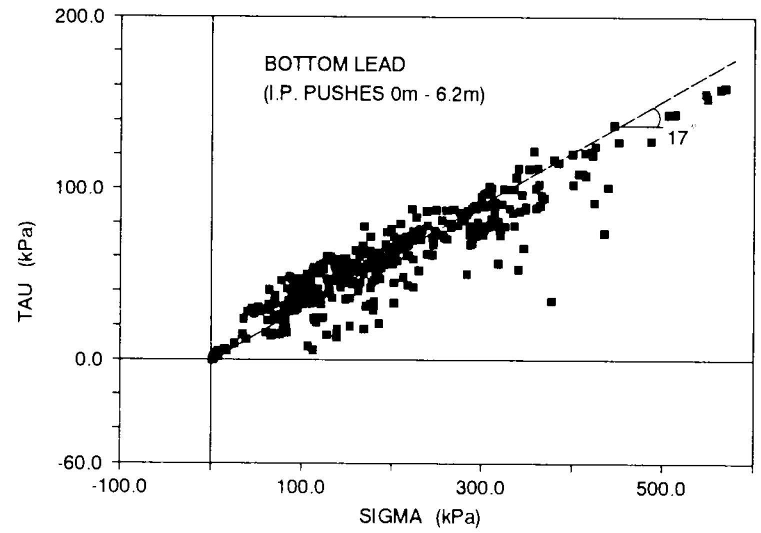


Figure 6.33 Shear stress/total radial stress relationship from the lead cell during scheme 2.

The scatter plots for scheme 3, Figure 6.34, are more difficult to interpret. Responses up to the time when glue lines failed are presented. A fundamental difference is observed between the axis and crown positions. Both axis instruments produce a reasonably linear frictional relationship with a skin friction angle δ of approximately 13° and δ' of 11.1°. These values are within the initial 4.5m of the drive and may have been affected by surface drying. The response at the crown is non-linear and approaches a limiting "adhesion" value of 120kPa. Closer inspection of the tunnel axis plots also indicates the possibility of a limiting interface shear stress of 120kPa. The reason for the marked increase in the crown interface friction angle, 26° (over the initial 1.6m), is not known. Instrumentation of a further pipe jack in London clay, with a full suite of contact stress and pore pressure readings, would be a valuable addition to the existing data base, and may provide some answers.

Shear stress against radial stress plots for the silty sand of scheme 4 are included as Figures 6.35 to 6.36. The data have been split into pre and post lubrication phases. Effective stress plots have only been produced for the bottom contact zone because the remaining locations were fully drained. There is very good agreement between the resulting skin friction angles δ of 33.4° and 34.3° at crown and axis positions prior to lubrication. These are slightly smaller than the 37.7° measured at the bottom of the pipe and probably reflect the loosened state of the collapsed material on to the pipe. The relationship between shear and radial stress appears to be stress level dependent at stresses in excess of 100kPa. The effective stress plot for the base of the bore shows considerably more scatter but produces a similar skin friction angle to that obtained from the total stress plot. The introduction of lubricant has clearly changed the interface behaviour. This factor is discussed in detail in section 6.4.3 together with the shear stress-radial stress plots from the fully lubricated drive of scheme 5.

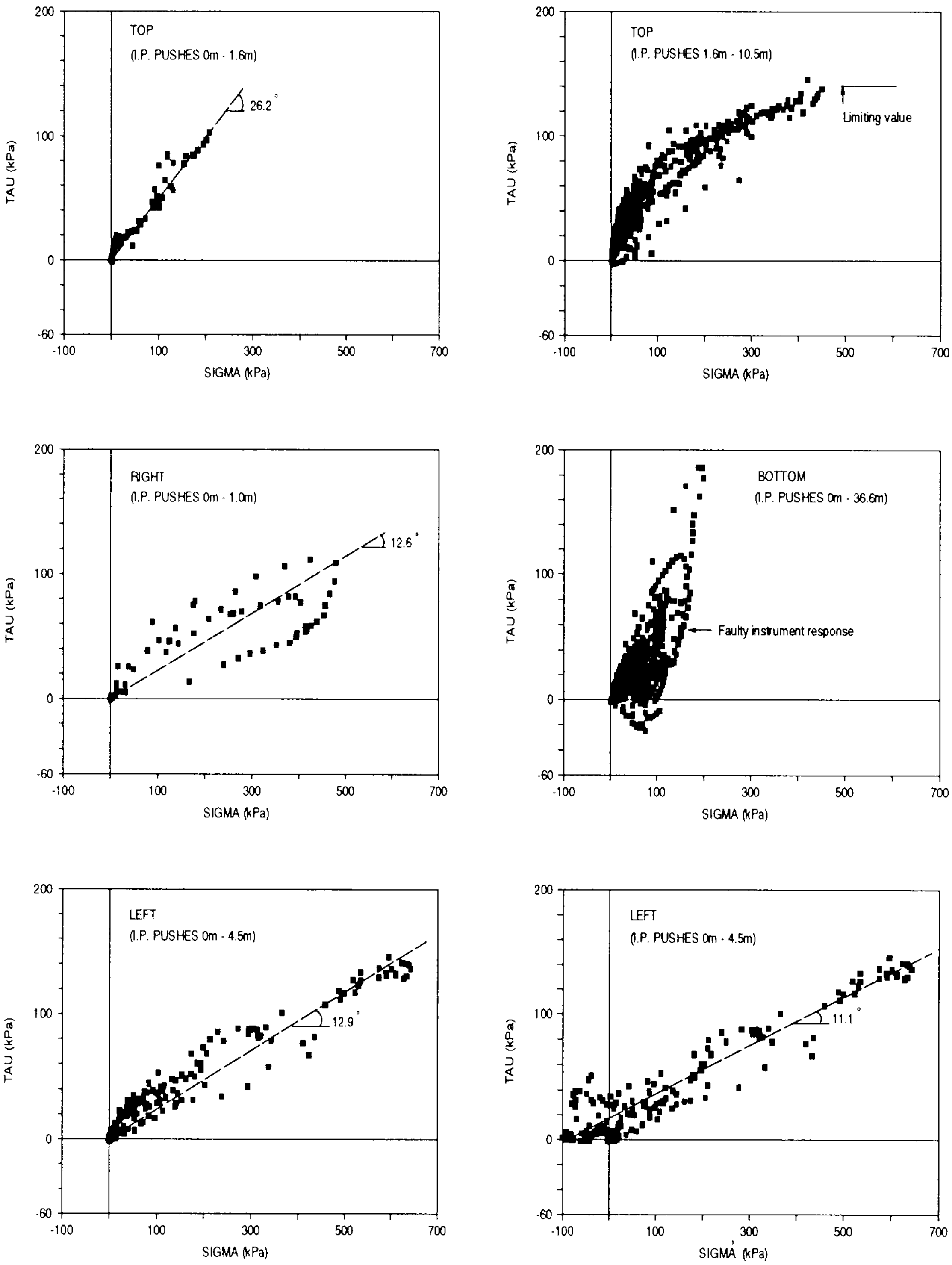


Figure 6.34 Shear stress/radial stress relationships during scheme 3.

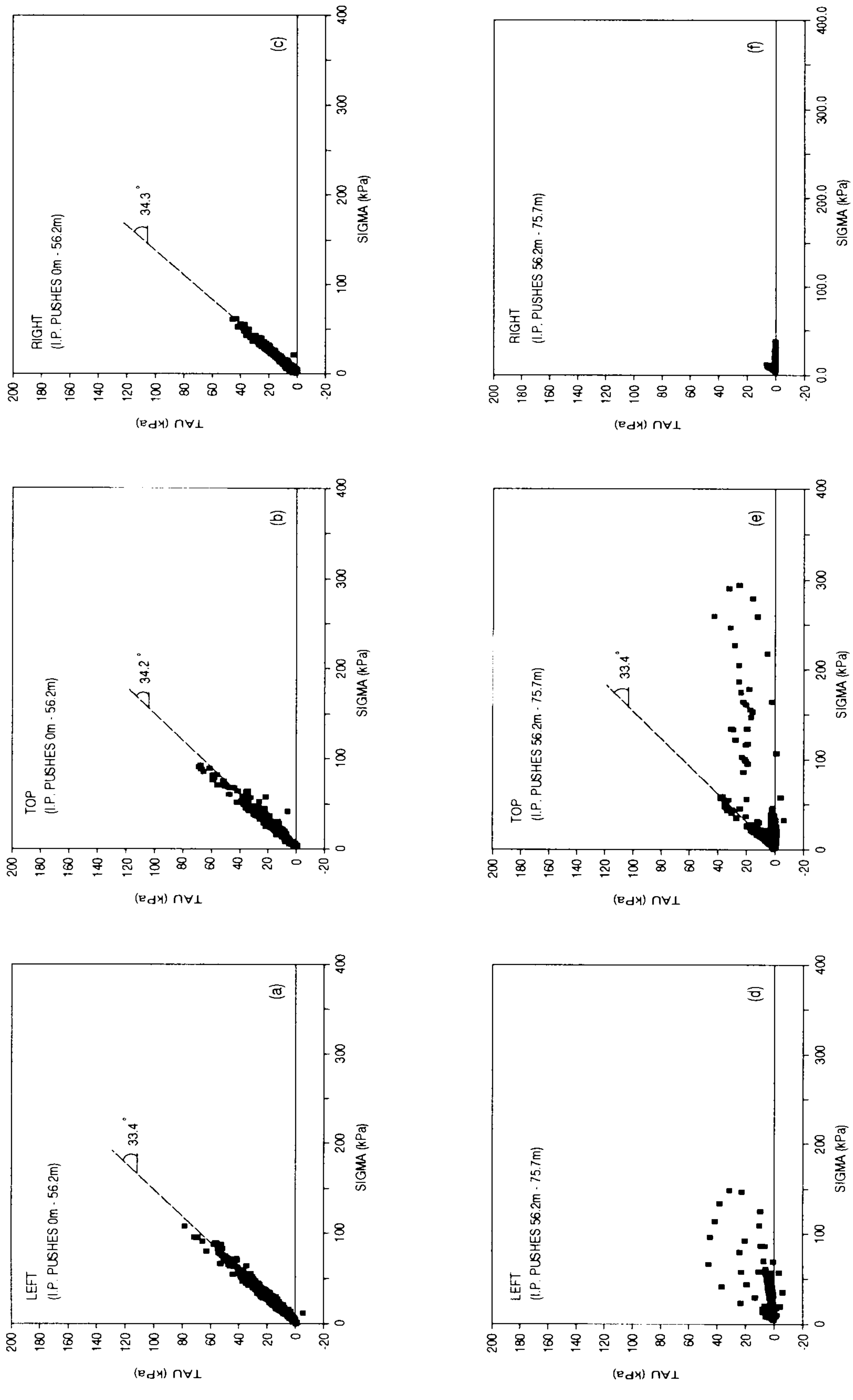


Figure 6.35 Shear stress/total radial stress relationships during scheme 4; prior to lubrication (a)-(c), and during lubrication (d)-(f).

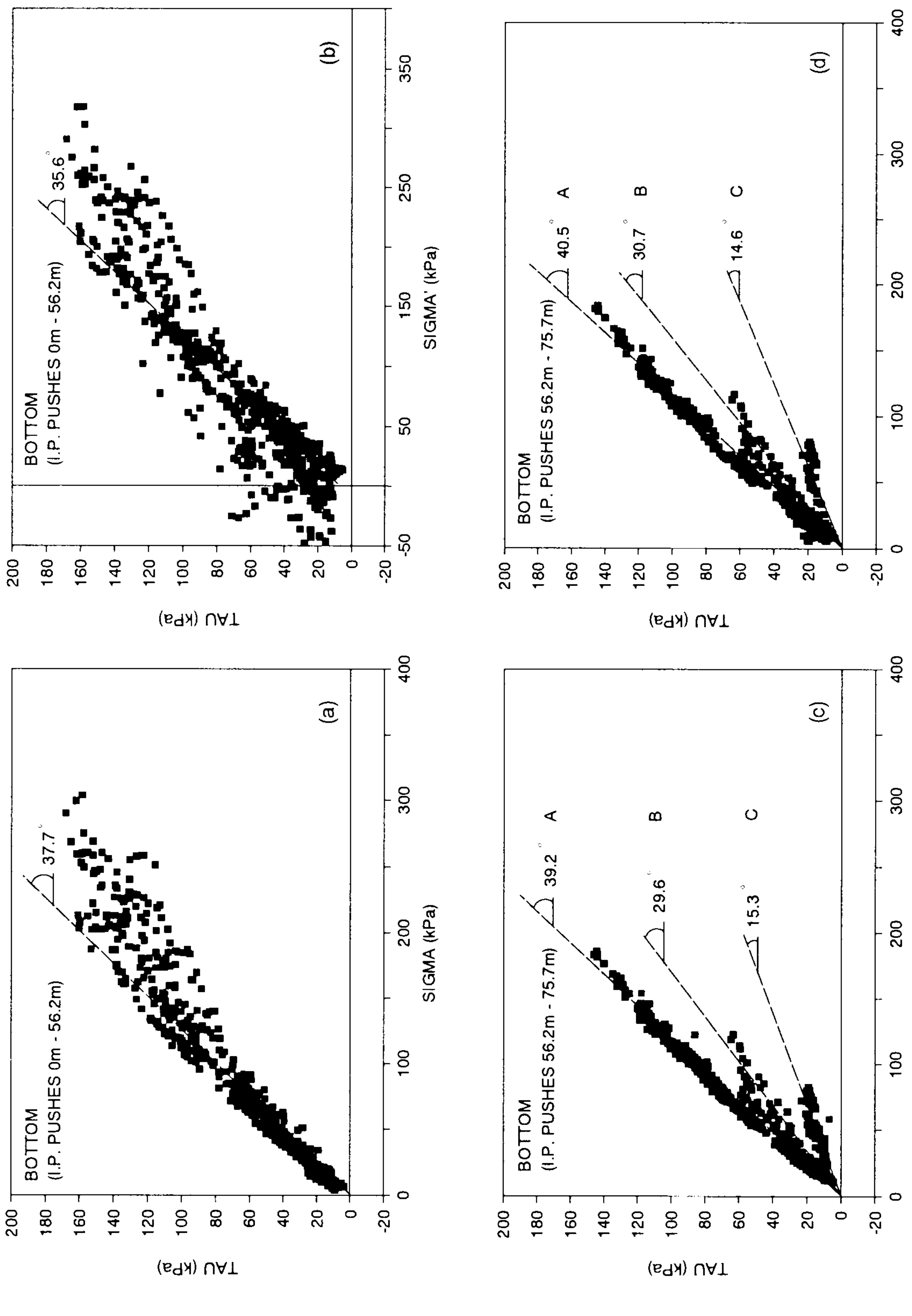


Figure 6.36 Shear stress/radial stress relationships during scheme 4; total and effective stress responses on the bottom of the pipe prior to lubrication (a & b) and during lubrication (c & d).

The measured skin friction angles fall within the typical ranges for the different soil types as illustrated in Table 6.3. Given the degree of scatter of the field results there is evidence to suggest that the repeated passage of pipes subjects the interface to large shear displacements causing the frictional behaviour in cohesive ground to be controlled by residual strength mechanisms and in non-cohesive ground by critical state conditions. An important point to note in Table 6.3 is the use of skin friction coefficients for critical state and residual conditions which strictly apply to peak angles. Only limited information has been found on this parameter, although it is likely that the error introduced at critical state conditions is small. The residual skin friction for London clay is particularly subjective. The field measurements do however compare favourably with residual skin friction angles obtained from ring shear interface tests between roughened stainless steel and London clay, Figure 6.44, which illustrates a typical angle of 11°.

	Internal angle of friction ϕ' °				Predicted δ' °			Field	
Scheme	Peak	Critical State	Ring shear ϕ'_R	Skin friction coeff-icent	Peak	Critical State	Residual	δ°	δ' °
1	33av	30 (Bolton 1979)	25.3 (Lupini et al 1981)	0.84 (Potyondy 1961)	27.7	25.2	21.3	19	20.3
2	35 (Bolton 1979)	25 (Bolton 1979)	12 (Lupini et al 1981)	0.84 (Potyondy 1961)	29.4	21	10	17	-
3	25 (Bolton 1979)	22 (Bishop et al 1971)	9.5 (Bishop et al 1971)	0.68 (Potyondy 1961)	17.0	15.0	6.5	12.7	11.1
4	47 (Bolton 1979)	32 (Bolton 1979)	-	0.87 (Potyondy 1961)	40.9	27.8	-	37.7	35.6

Table 6.3 Predicted and measured coefficients of skin friction

6.3.2 Pipe self weight friction

Comparison of the pipeline self weight friction with measured average frictional resistance from the jacking records is presented in Table 6.4. Reasonable agreement is obtained for schemes 1, 2 and 4 which are through relatively stable bores. The larger recorded values are probably a function of limited ground closure onto the pipe and the effects of pipeline misalignment. A greater understanding of the imposed radial loading from the highly plastic overconsolidated clay is necessary to obtain closer agreement for scheme 3.

Scheme	Field skin friction δ	W tan δ (kN/m)	Av friction (kN/m)
1	19°	6.1	7.2 29.8
2	17°	7.0	8
3	13°	8.8	54.4
4 unlub lub	37.7° 15°	18.7 6.5	23.1 9.4

Table 6.4 Pipe self weight friction

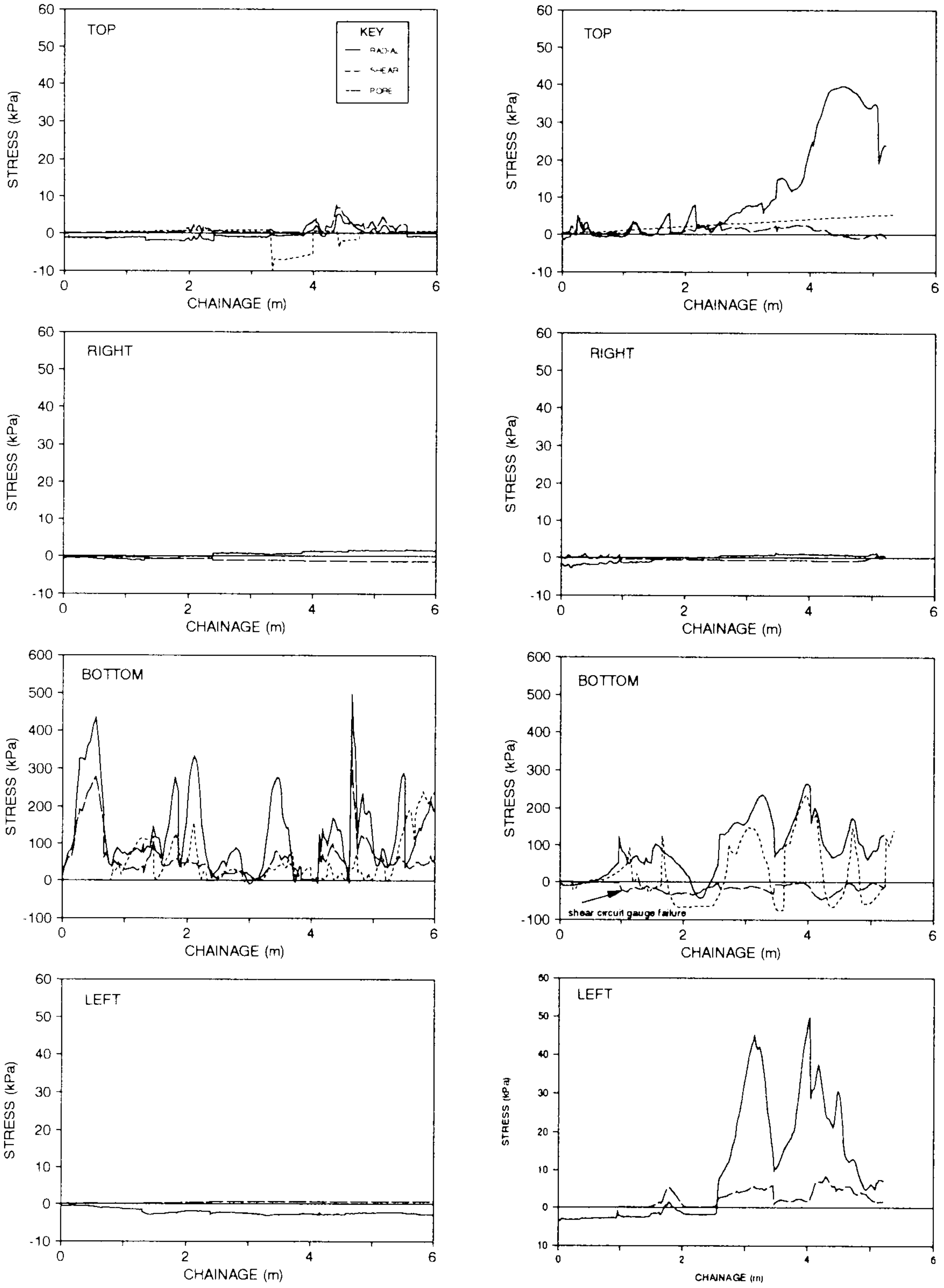
An interesting feature of the shallow tunnel of scheme 1 is the large increase in frictional resistance following the period of heavy rain and temporary inundation of the tunnel. Closer agreement is obtained if the undrained shear strength model of Haslem is used.

From Figure 6.31c, αs_u approximates to 80kPa;

contact strip width, b, based on E_{clay}=48MPa equals 0.3m;

$F = \alpha . s_u\ b$ = 24kN/m (compared to measured 29.8kN/m)

Insertion of the instrumented pipe close behind the shield provided the opportunity to push the pipe into the reception shaft and re-jack it two pipe lengths into the tunnel. Comparison of the initial and re-jacked radial loading substantiates that closure of the ground took place around the pipe, Figure 6.37, which almost certainly accounts for the above difference.



(a) Initial interface stresses **(b) Re-insertion interface stresses**

Figure 6.37 Interface stresses during initial 6m of the drive of scheme 1 and corresponding values after re-insertion of the instrumented pipe.

6.4 Construction related factors

6.4.1 Misalignment

Comparison of tunnel alignment and local interface stress profiles, Figure 6.38 illustrates excellent agreement between the positions of peak stress and maximum tunnel deviation. For scheme 4 the major deviations were in the vertical plane and enable a tentative plot of total radial stress against angular deviation to be produced, Figure 6.39. Although there is considerable scatter, larger angular deviations generally give rise to larger ground reactions.

Closer inspection of the various interface stress plots and tunnel alignment surveys suggests that at the positions of maximum deviation the pipelines try to straighten and act as post-tensioned tubes under load. Misalignments are maintained by the generation of large ground reactions on the inside of the series of large radius curves created by the corrections to line and level. This is contrary to the basis of Haslem's (1986) misalignment analysis which assumed that ground reactions would be generated on the outside of each curve. The difference in behaviour arises because Haslem assumed that a curved section of pipeline acted as a continuous member whereas in practice the small varying angular deviations at each joint produces complex local loading regimes on individual pipes. Two cases are illustrated in Figure 6.40. The first can occur at positions of rapid changes in direction, while the second corresponds to gradual changes in direction which approximate to large radius curves. It is important to note that the angular deviations β increase around these approximate curves, peaking at the point of maximum amplitude. The mode of loading in pipe 3b is supported by the field measurements, Figure 6.41; no evidence was found supporting pipe 3a loading. It should be noted from Figure 6.40 that a multitude of different ground reaction distributions occur as a pipe moves through the ground and that in many cases the central positioning of the contact and pore pressure cells is inadequate to record such data. Gravitational effects can also swamp ground reactions arising from vertical misalignment as illustrated in the interface plots for scheme 1 where contact is principally maintained along the base of the bore. Further analysis of pipe-soil interaction with particular emphasis on the resulting pipe strains, is included in Chapter 7.

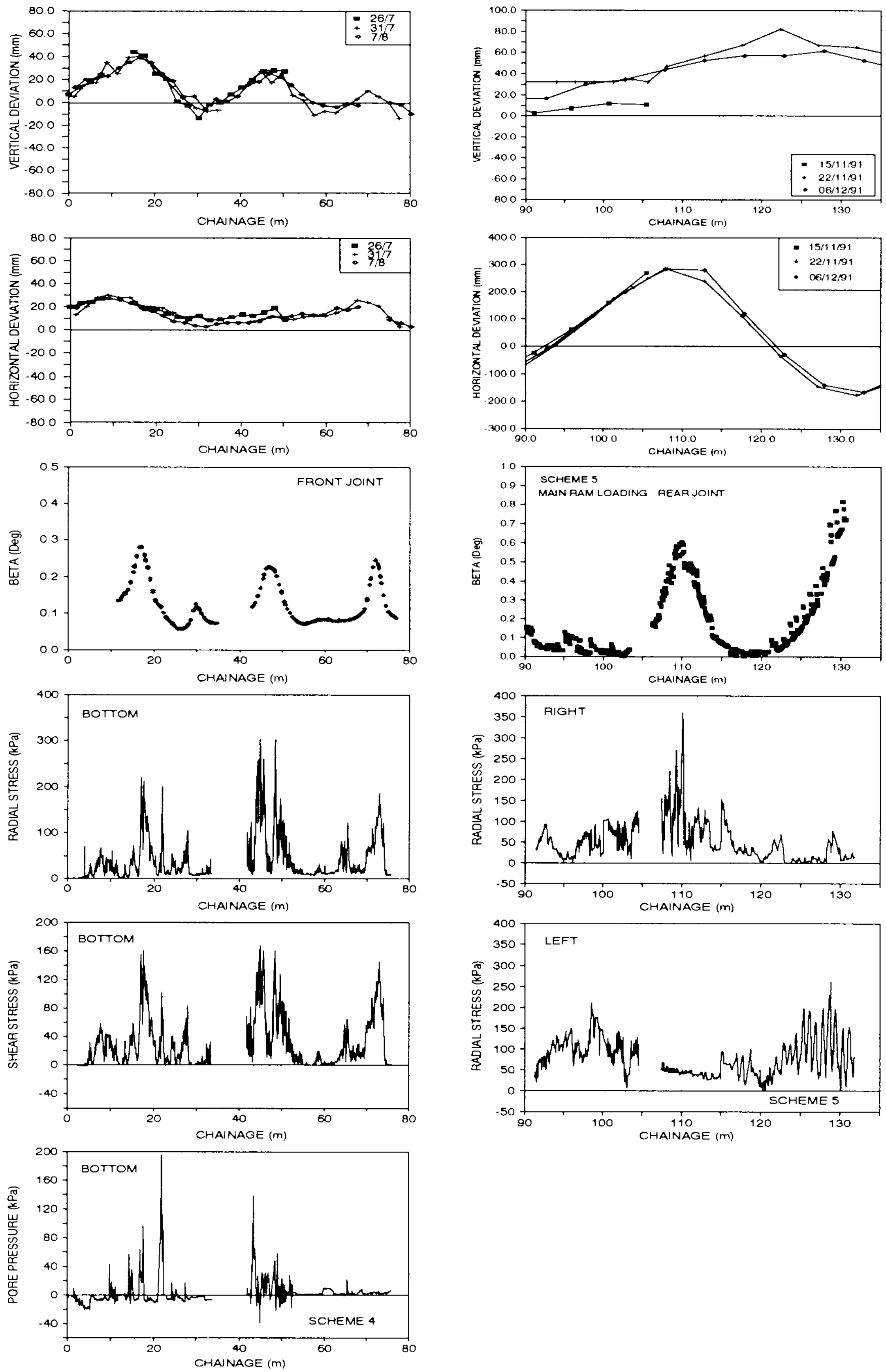


Figure 6.38 Comparison of tunnel alignment data and local interface stresses.

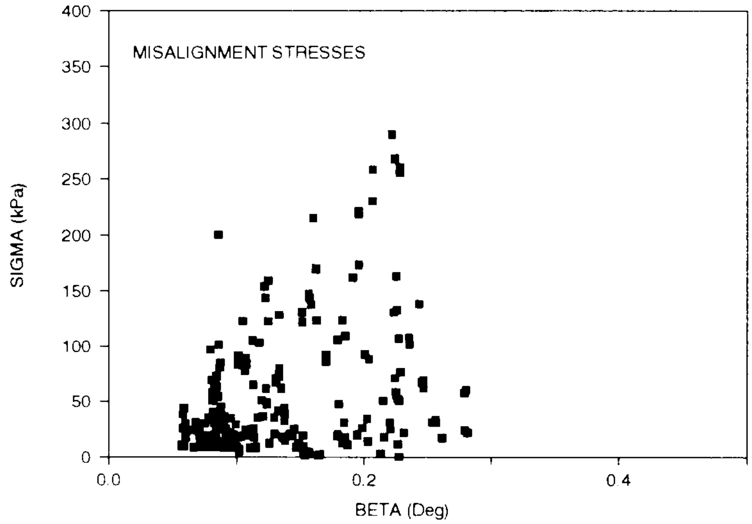


Figure 6.39 Relationship between angular misalignment and induced total radial stress in scheme 4.

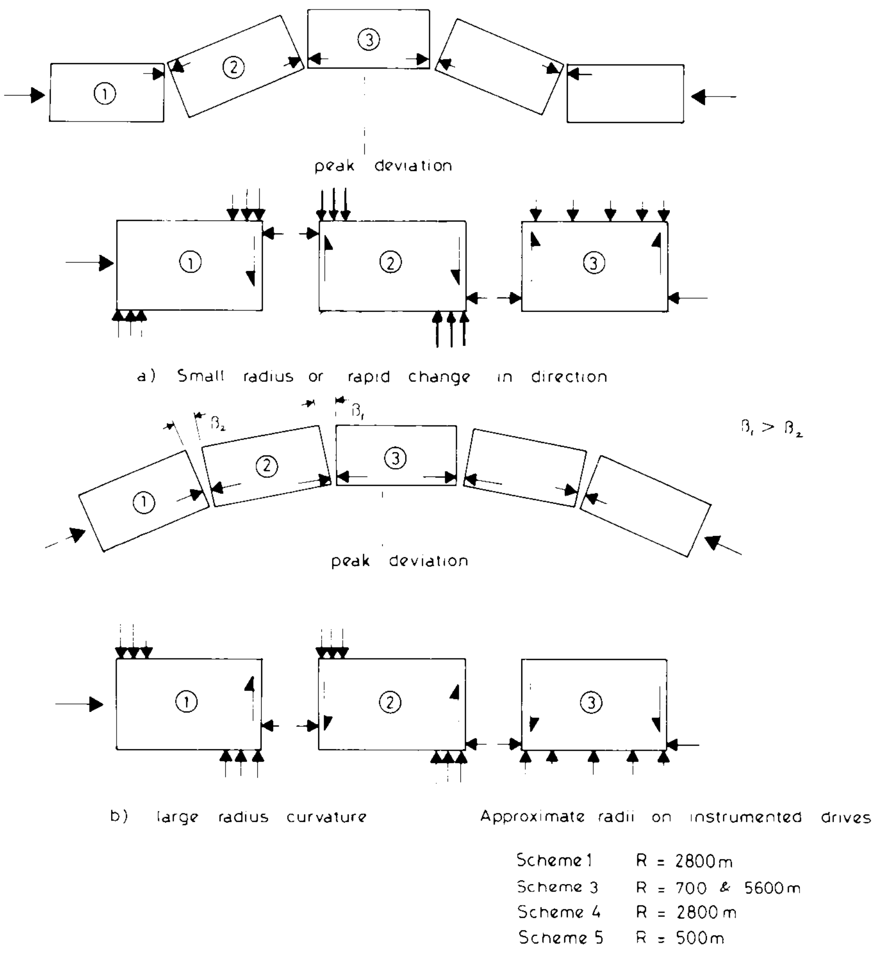


Figure 6.40 Theoretical misalignment forces.

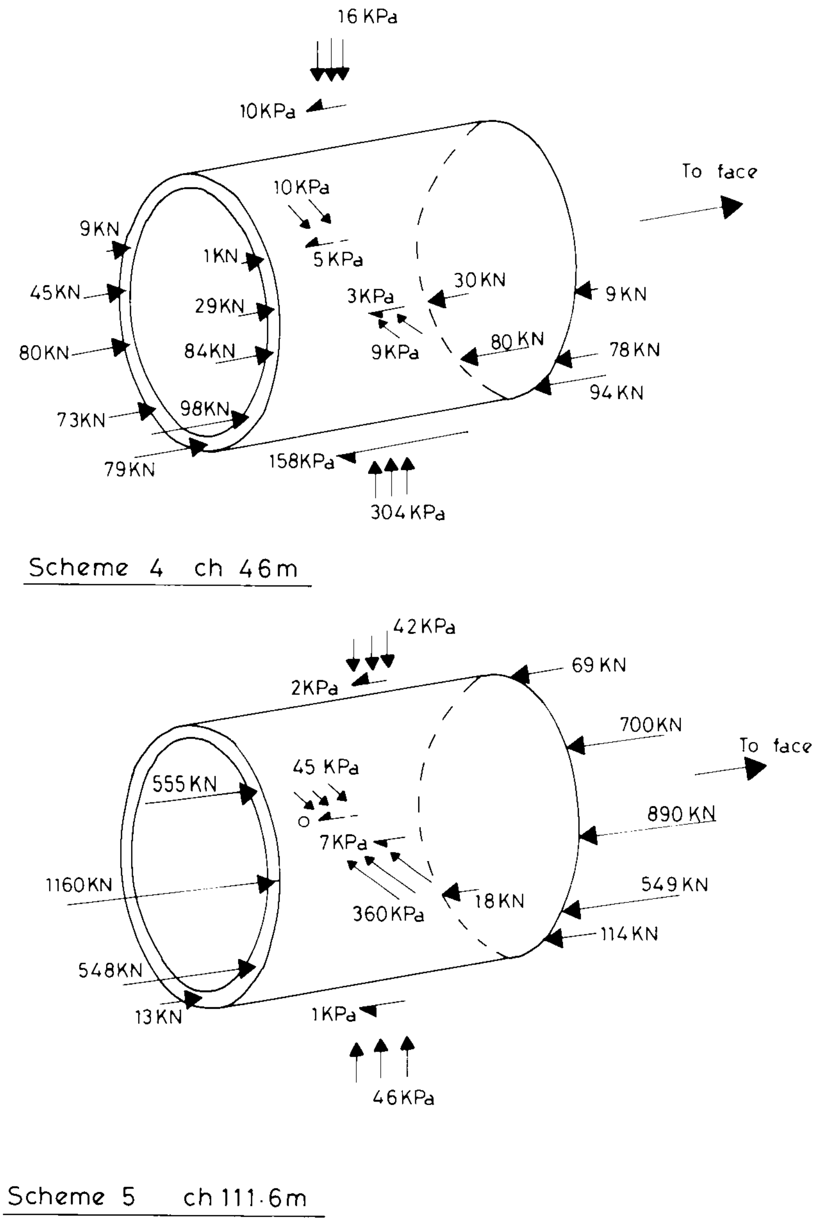


Figure 6.41 Field measured misalignment forces.

Before leaving the discussion on misalignment effects it is worth evaluating the average frictional resistance over the single pipe lengths of Figure 6.41. This is achieved by numerically integrating and subtracting the total joint loads at each end of the instrumented pipe and dividing by the pipe surface area. The result produces values of 29kPa and 11kPa respectively for schemes 4 and 5. Clearly these are much larger than the pipeline averages of 4.2kPa and 3.6kPa and further demonstrate localisation of interface stresses.

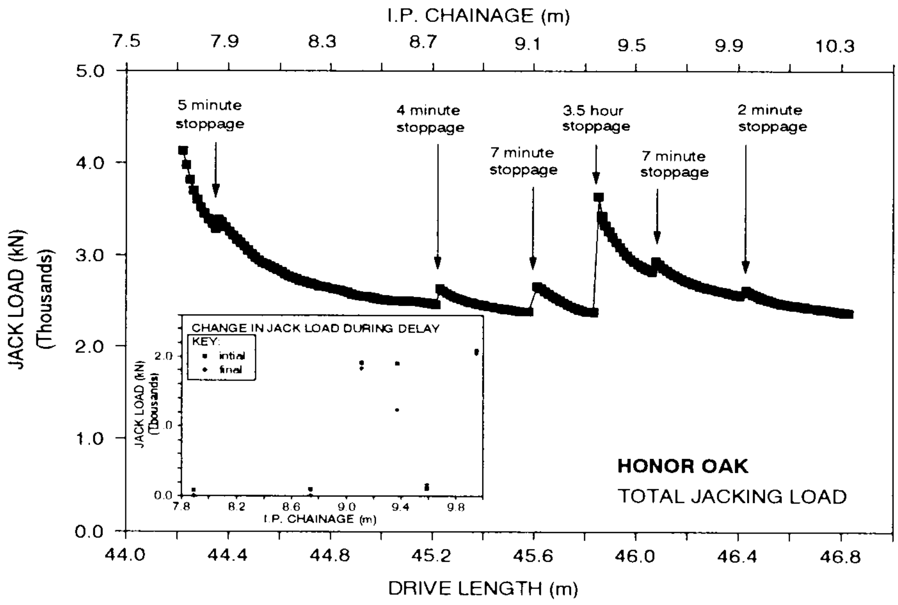
6.4.2 Time factor

Increases in pipe jacking forces in clay soils following a lapse in jacking are routinely recorded. The effect was evident in the low plasticity clay drive of scheme 1, although a more dramatic effect was observed in the high plasticity London clay of scheme 3. The restart forces over a short section of scheme 3 and the associated total radial stresses acting on the pipe at the start and end of each stoppage are presented in Figure 6.42; the rapid and repeatable increase in restart jacking forces is a pronounced feature. It has previously been suggested that the time factor allows consolidation of the ground to take place around the pipe increasing the radial pressure and hence the related frictional resistance, Auld (1982). The total radial stress plot however illustrates stress reductions during the rest periods which are not a function of changes in jack ram loading during the stoppage. Unfortunately there are no reliable pore pressure data to establish whether the reductions are the result of large pore pressure dissipation.

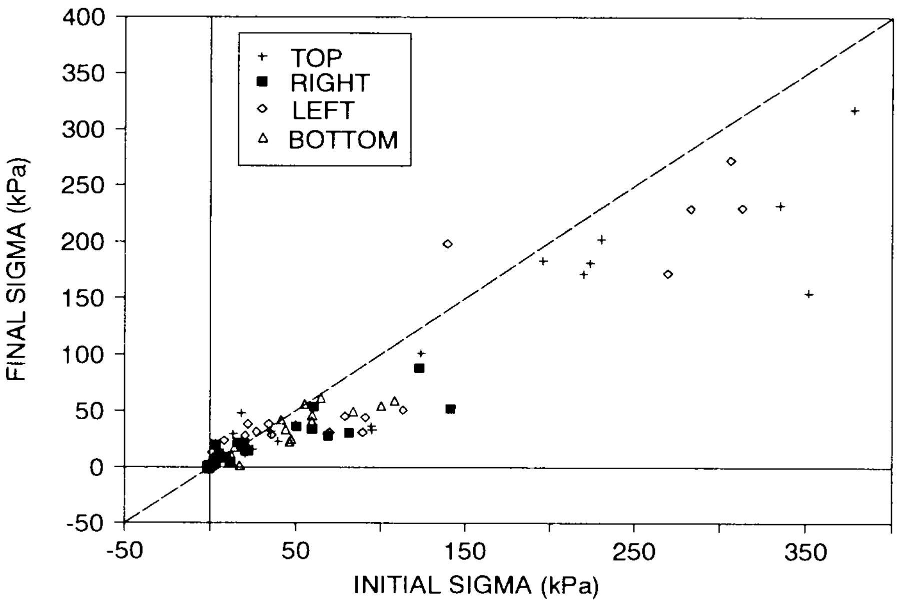
This mechanism has been investigated using the data from scheme 1. Plots of changes in total and effective radial stress during stoppages are presented in Figure 6.43. It can be seen that the total radial stresses reduce but that the effective stresses increase. It would appear that the process of shearing the ground generates positive pore pressures which then dissipate during the stoppage. The resulting increase in effective stress produces larger jacking forces which subsequently reduce during re-jacking because of the re-generation of positive pore pressures. The rapid and repeatable response of the mechanism appears to be a function of the limited depth of the interface zone. If one way drainage is assumed and the interface zone is say 5mm then for a drive through London clay the time t_c for completion of transient drainage caused by a load increase is given by:

$$t_c \;=\; \frac{d^2}{c_v} \;=\; 2\text{x}10^{-6}\ \text{year} \;=\; 15\text{sec} \qquad \text{when } d=0.005\text{m and } c_v=50\text{m}^2/\text{year}$$

Further investigation including laboratory interface tests is required before the processes involved can be fully identified and the extent of the interface zone established.

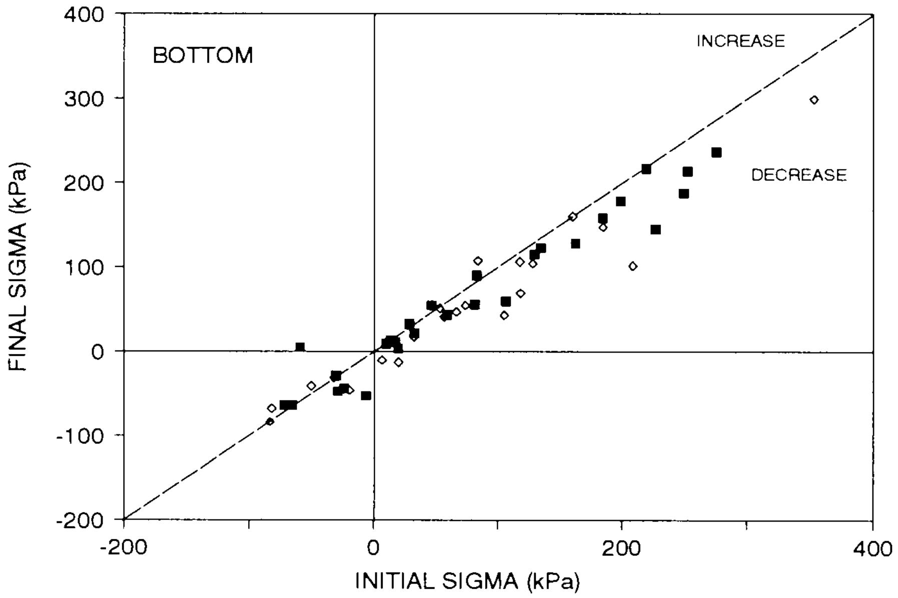


a) Jacking load

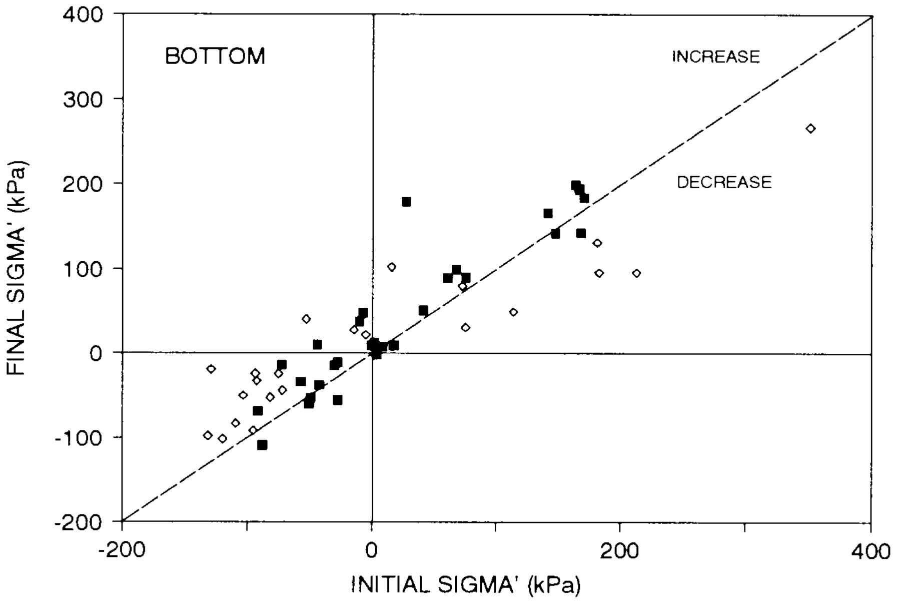


b) Total radial interface stresses

Figure 6.42 Time dependent changes during stoppages on scheme 3.



a) total radial interface stresses



b) effective radial interface stresses

Figure 6.43 Time dependent changes during stoppages on scheme 1.

The occurrence of large restart forces after short pauses have been observed during laboratory controlled ring shear interface tests between steel and London clay, Tika (1989). The tests were conducted under constant applied normal stresses and subject to fast rates of shearing of 110mm/min and 1100mm/min. The responses are included as Figure 6.44 and highlight a peak resistance which drops rapidly to a minimum value with fast displacement; the magnitude of frictional resistance increasing with increasing rate of shear. Tika suggests that the fast rate of shearing may change the basic shearing mechanism of the soil from pure sliding to sliding with some turbulence of the platy clay particles. Typical jacking rates fall between these two values, Table 6.5, and cover a large range between 100mm/min and 410mm/min. Rate effects are therefore important and need to be considered when determining the frictional resistance of cohesive soils. By contrast, high velocity ring shear tests on sand, Hungr et al (1984) indicate that rate effects are insignificant. It will be noted that although the global response of the pipe jack records seem to match laboratory trends, the shearing mechanism may be further complicated by both radial and shear stresses varying during pushes.

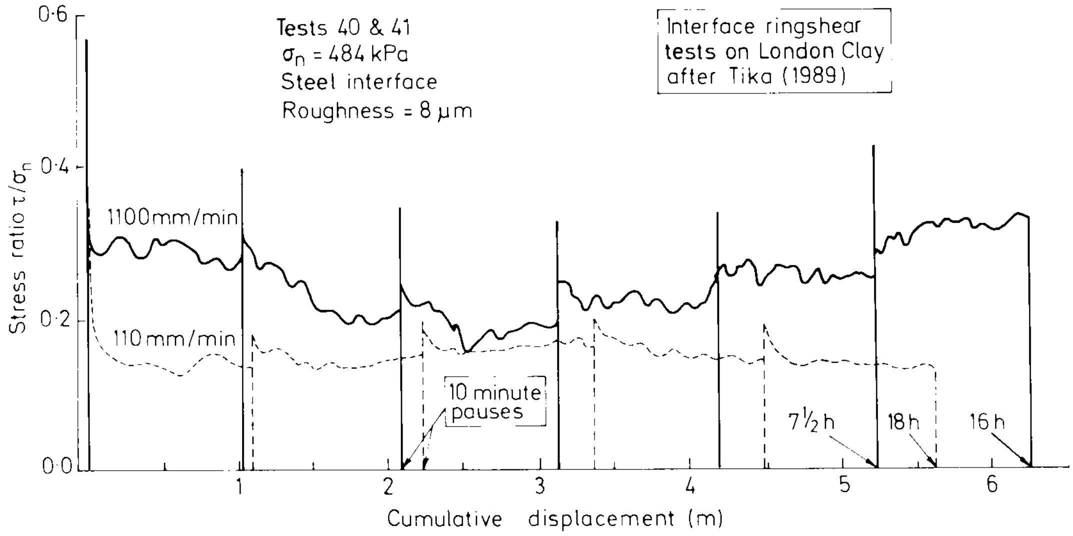


Figure 6.44 Interface ring shear tests on London clay.

	Rate of advance (mm/min)	
Scheme	2 rams	4 rams
1	270	-
2	160	-
3	335	185
4	410	200
5	-	100-150

- arrangement not applicable

Table 6.5 Average pipe velocity during pushes.

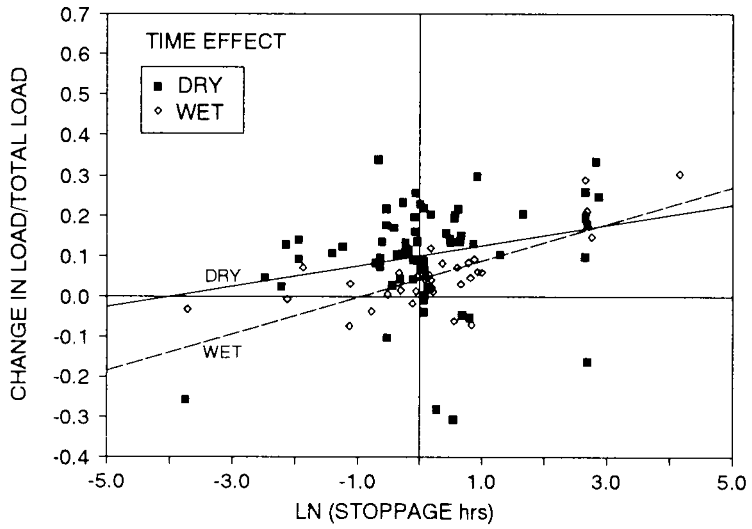
The increase in jack load after stoppages on schemes 1 and 3 have been normalised with respect to the total jacking load at the end of the previous push and plotted against the natural logarithm of the stoppage duration, Figure 6.45. A reasonably linear relationship is obtained in both schemes with the scatter probably a function of the changing face resistance. The rate of increase in jacking load can be expressed as follows:

Scheme 1 (dry) $\delta P/P=0.113+0.004\ln(t)$

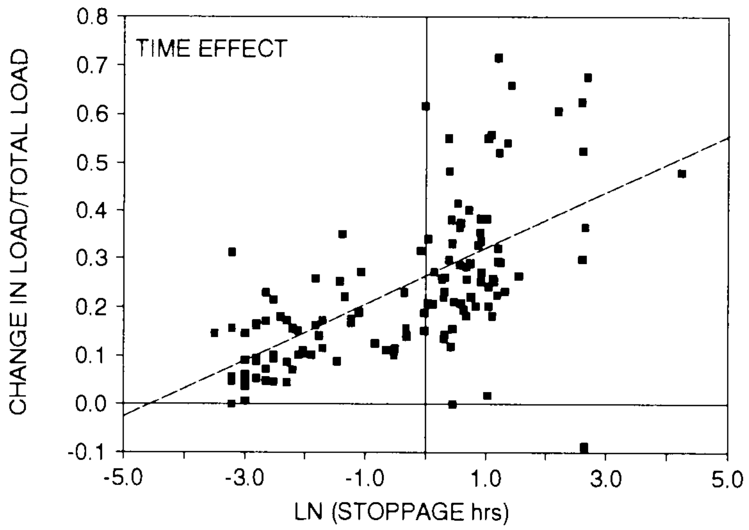
Scheme 1 (wet) $\delta P/P=0.042+0.045\ln(t)$

Scheme 3 $\delta P/P=0.259+0.058\ln(t)$ where t = time(hrs)

The time effect on scheme 1 is not particularly marked although the increased moisture content during the latter stages has significantly increased the factor. The relationship for scheme 3 demonstrates that a major part of the increase in load occurs within minutes of stopping. It is suggested that similar plots produced from jacking records from a range of pipe jack schemes would provide a useful tool for contractors to adequately plan work stoppages.



b) Scheme 1



a) Scheme 3

Figure 6.45 Time factors for schemes 1 and 3.

6.4.3 Lubrication

Use of lubricants was avoided in the first three drives, introduced during the later stages of scheme 4 and used throughout the machine drive of scheme 5. The effectiveness of lubrication is dependent upon many factors, including minor changes in ground conditions and correct selection of equipment and procedures for the injection system. Figure 6.36 provides some indication of the local variability of its effectiveness. The data are taken from scheme 4 over a section of tunnel through silty sand which varied in silt content from 70% down to 15%. At the time of monitoring it was unlikely that the full annulus around the pipe was filled with lubricant. Line B and C correspond to areas with a high silt content while line A corresponds to the coarser grained material. The apparent coefficient of skin friction given by line C is one third of that given by line A (which is similar to the un-lubricated value). Closer scrutiny of the site log of activities reveals that the difference in B and C is probably because injection had taken place overnight in the area of the instrumented pipe prior to the pushes along line C whereas B was recorded later in the shift when lubrication was being concentrated in a different section of the drive. These observations suggest that an insufficient quantity of lubricant was introduced into the drive causing localisation of its effectiveness. Inspection of the ground surrounding the instrumented pipe upon removal of the contact stress cells confirmed that the annulus was not completely filled although a layer of soil-lubricant mixture, typically 10mm thick, had formed adjacent to the pipe over the bottom half of the pipe.

In contrast the data from scheme 5 suggests that the lubricant around the pipe annulus was pressurised, supporting the ground and causing pipeline buoyancy and low frictional stresses. Plots of interface shear stress against total and effective radial stress for the top and right axis positions are included as Figure 6.46. The zero friction angles and average shear stresses close to zero are consistent with shearing within the bentonite gel layer which has a shear strength of approximately 0.05kPa.

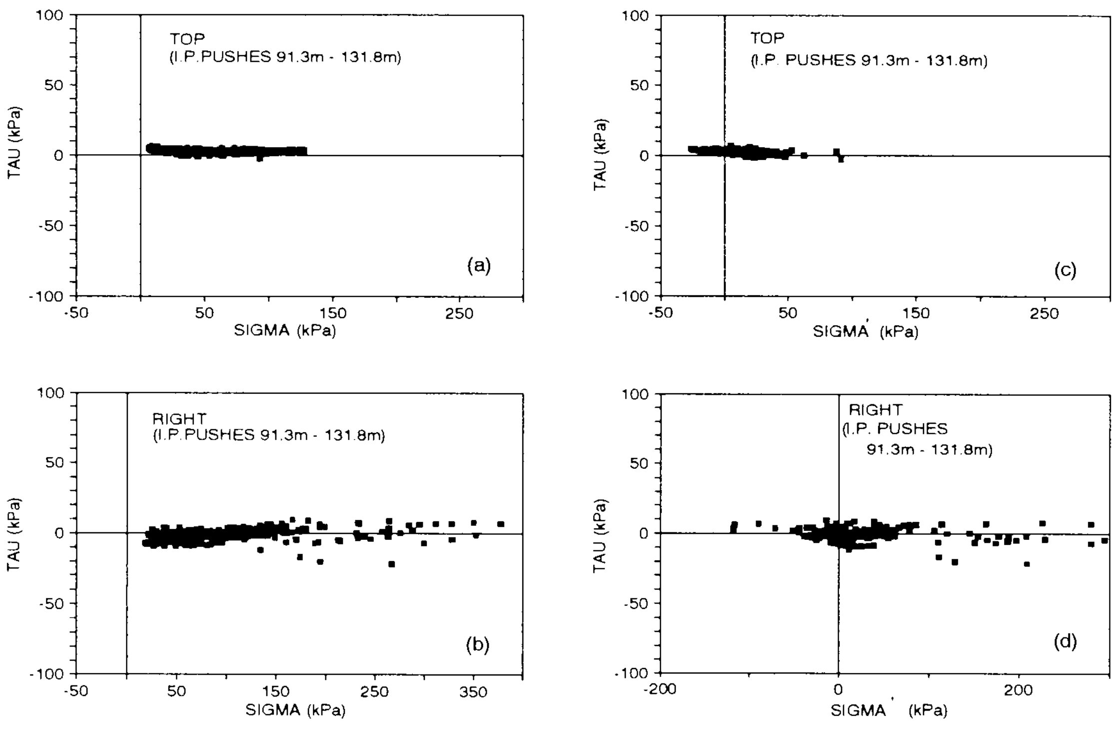


Figure 6.46 Shear stress/radial stress relationships during scheme 5; total stress responses (a-b) and effective stress responses (c-d).

CHAPTER 7:

PIPE STRESSES

7.1 Introduction

One of the principal aims of the research is to improve understanding of the loading carried by pipes during the jacking phase, when the pipes are most heavily loaded and liable to failure. Results from chapter 5 have demonstrated the severe effects of the localisation of end loading on pipe joints during the jacking process. This chapter will establish typical load paths through pipes on real sites; investigate the overall behaviour of the pipe barrel under various field conditions using the tube extensometer readings; and carry out elastic analyses for the worst loaded pipe positions in scheme 5 to substantiate the strain observations.

7.2 Pipe load paths

As the pipe string "wriggles" through the ground, the angular orientation of maximum joint compression, α, will move around the joint as shown for example by the traces from individual pressure cells in Figure 5.17. If the centres of pressure are at the same angular position at both ends of a pipe, the load will be transmitted essentially along one edge of the pipe; if they are out of phase by 180° , the pipe will be loaded across a diagonal. In general the site measurements of joint stresses correlate well with joint compressions and therefore the load paths will be established from the differences in the positions of maximum compression. The results from schemes 1,3,4 and 5 are presented in Figures 7.1 to 7.4.

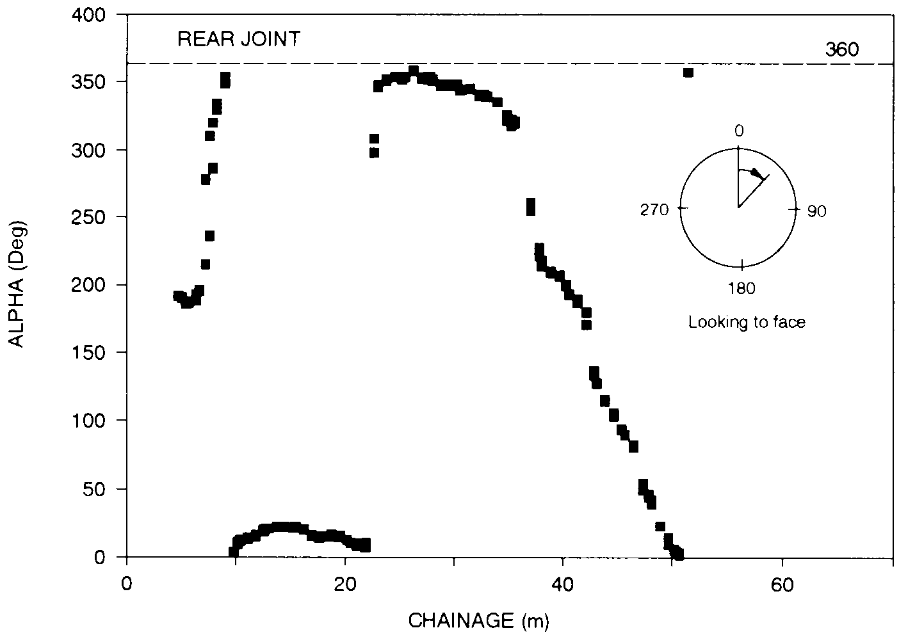


Figure 7.1 Variation in the point of maximum compression derived from the joint movement indicators across the rear joint of scheme 1.

For scheme 1 only limited instrumentation was incorporated with no joint movement indicators positioned at the front joint. The centre of compression in the rear joint is constantly on the move; during the drive it moved around the circumference 1.5 times which appears to be a function of its close proximity to the shield. This variability was also illustated in the angular deviations, β ,and could set up significant pipe strains in drives with large face resistances. Further back in the pipe strings of schemes 3 and 4, Figures 7.2 and 7.3 the variation in α is smaller. Excellent repeatability is obtained when the front and rear joints pass the same chainage. By comparing the traces with a phase difference of one pipe length it can be seen that the majority of readings lie within ±45° of loading along one edge. A value of 6° was observed in scheme 3 at the position of maximum angular deviation, β, with the maximum of 110° corresponding to a small value of β. These results are not surprising if the approximate large radius analogy of section 6.4.1 is considered.

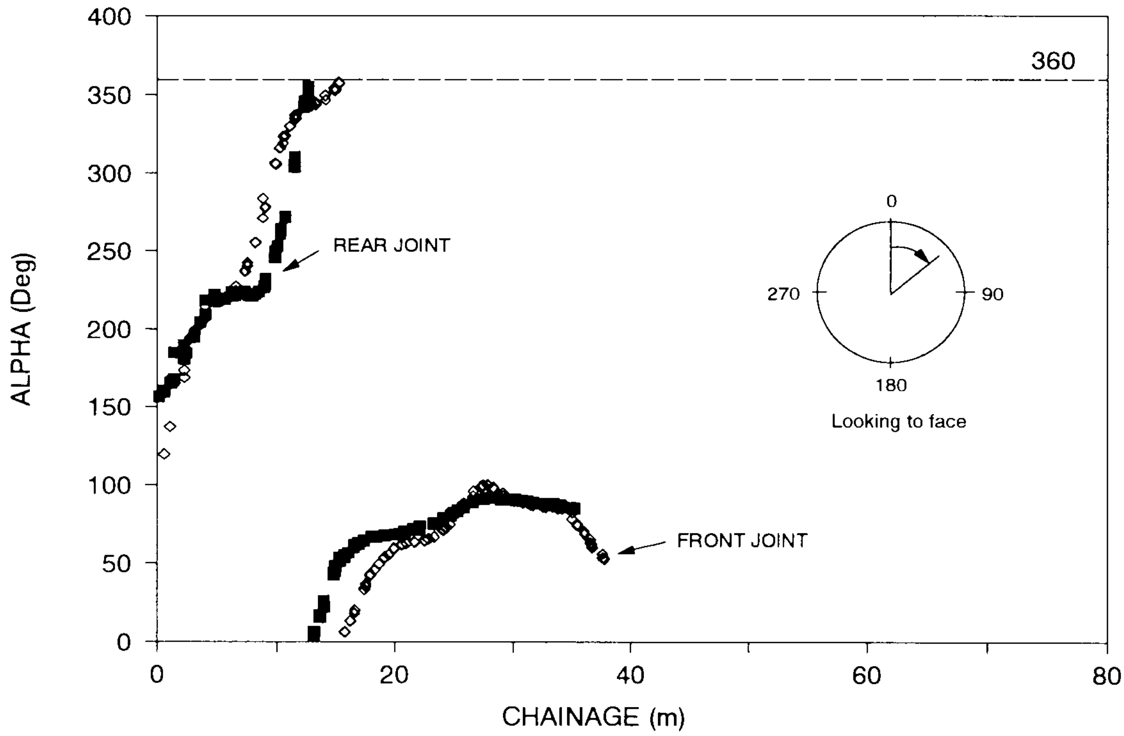


Figure 7.2a Comparison of the points of maximum compression in the front and rear joints at similar chainages throughout scheme 3.

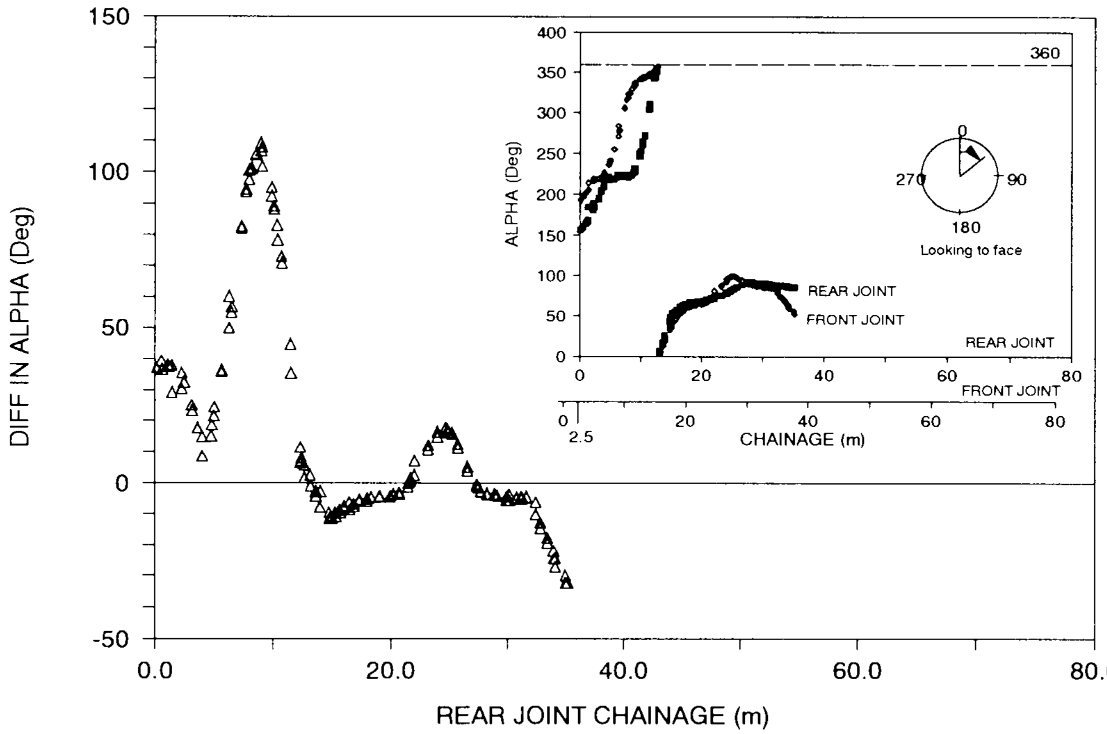


Figure 7.2b Angular difference between the front and rear points of maximum compression; scheme 3.

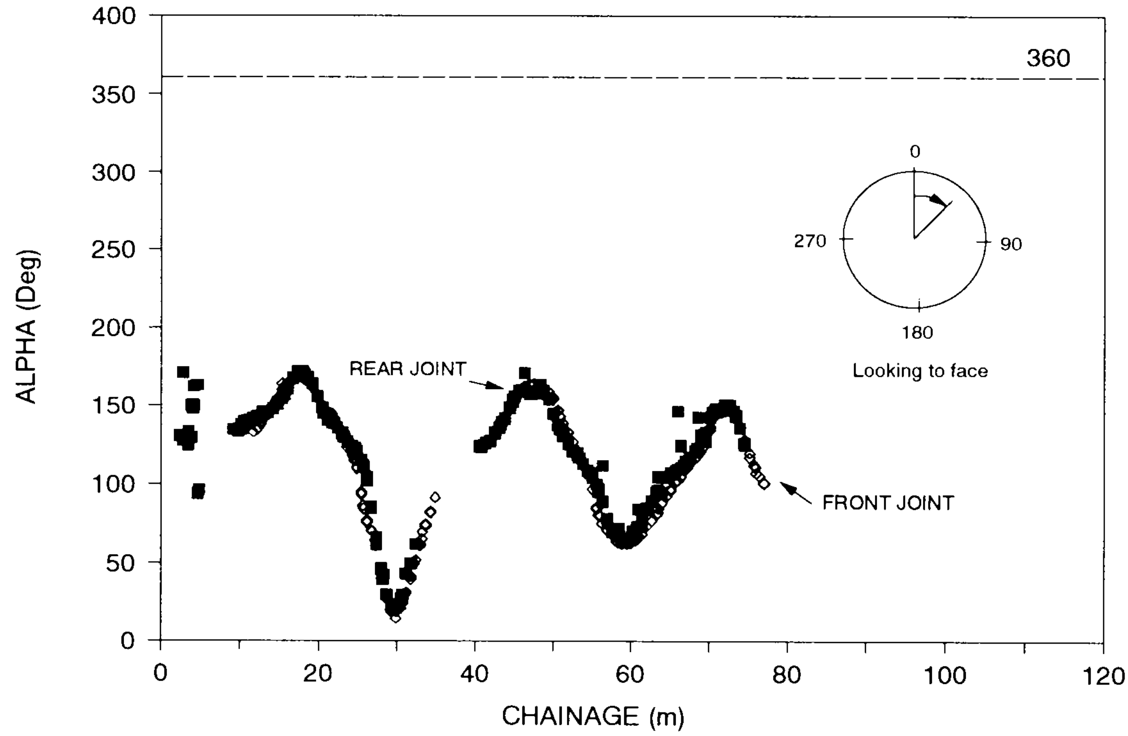


Figure 7.3a Comparison of the points of maximum compression in the front and rear joints at similar chainages throughout scheme 4.

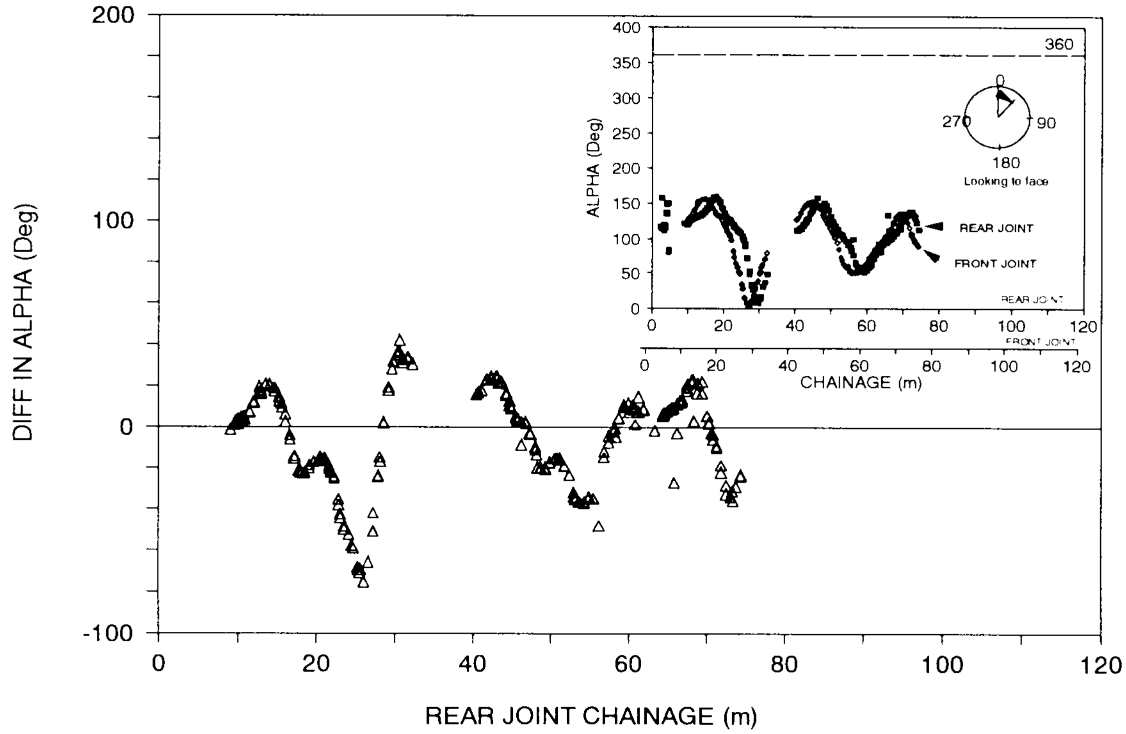


Figure 7.3b Angular difference between front and rear points of maximum compression; scheme 4.

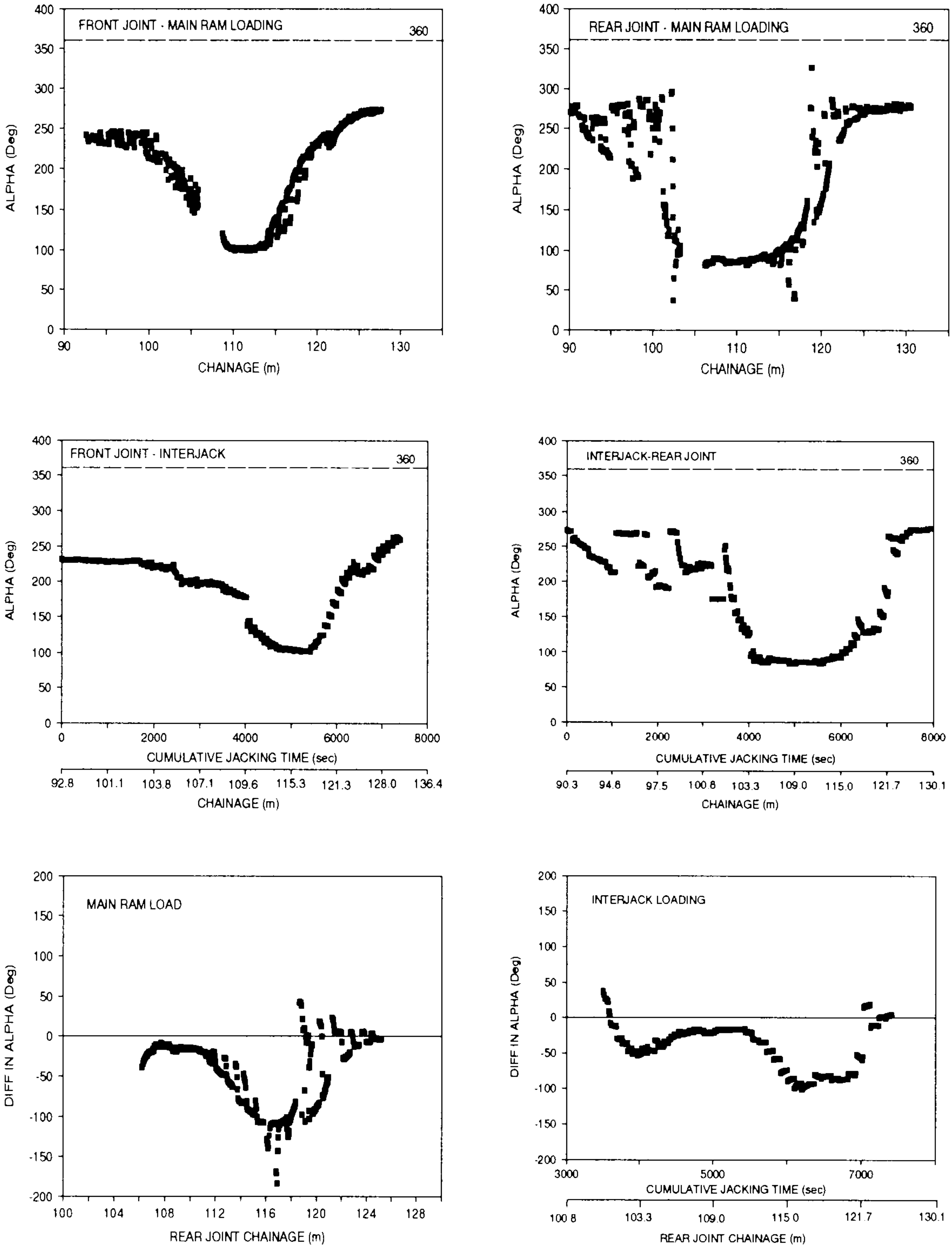


Figure 7.4 Variation in position of maximum compression in the front and rear joints and the angular difference between them, scheme 5.

In the case of scheme 5 the comparison is made over the tunnel section containing the reverse bends, Figure 7.4. Similar responses are illustrated during the main ram and interjack loading. Differences in α of zero to 10° were observed at the positions of maximum tunnel misalignments. The value increases to 120° in the section between the extremities but this corresponds to small angular deviations which do not produce extreme pressure distributions. On the basis of the observed behaviour it appears that the extreme diagonal loading case is unlikely to occur with normal site control procedures. The single edge loading model is a more appropriate worst scenario with the majority of pipes subject to various intermediate load paths.

7.3 Pipe barrel strains

Pipe barrel strain profiles have been derived from a series of tube extensometers equi-spaced around the pipe circumference. Results are presented from schemes 3, 4 and 5 in Figures 7.5 to 7.8. There is excellent agreement between large compressive strains and the positions of maximum compression in the joints. During scheme 3 the pipe was subject to overall compression with peak compressive strains of 190 microstrain. The sensitivity and rapid response of the instrument type is clearly illustrated by the series of peaks which correspond to the increase in total jacking load associated with stoppages. The strain level in scheme 4 was very low with strains over the major part of the drive below 60 microstrain.

Strains associated with the pipe damage of scheme 5 are presented in Figures 7.7 and 7.8. Values exceed 500 microstrain at the chainages which correspond to observed localised crushing of pipe joints. The strains induced during the main ram pushes are slightly larger than those from the interjack pushes. This is a function of the instrumented pipe being positioned close behind the interjack and the contractor's preference for activating the interjack and then closing it up and continuing to push the whole pipe string from the jacking pit. Good agreeement is obtained between the pair of tube extensometers on each side of the pipe which is a consequence of the horizontal alignment dictating joint load transfer. The response of the bottom probe is suspect after chainage 120m as a result of its total immersion in bentonite slurry.

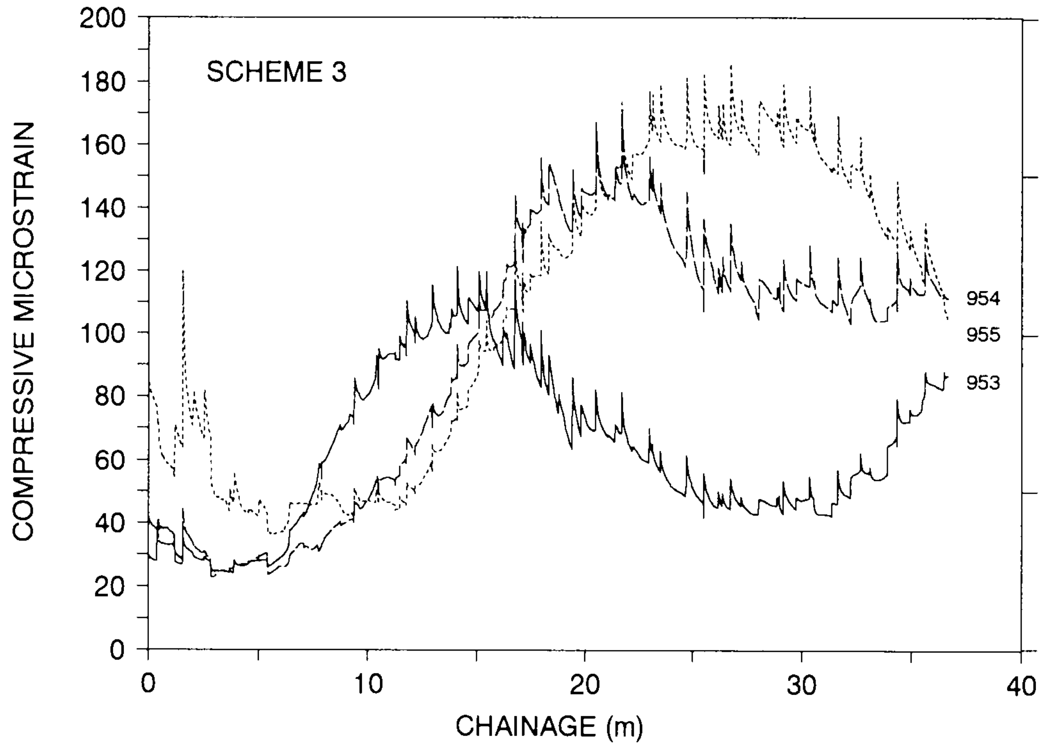
7.4 Pipe barrel stresses

During the course of the research a series of concrete specimens were cast at C.V. Buchan's pipe works and tested under laboratory conditions to determine 28 day strength and elastic moduli values. All specimens were stored in water at 20°C ± 1°C from the time they were cast and tested in acccordance with the relevent British Standards. The results are presented in Table 7.1.

Type of test	Mean	Characteristic (mean - 1.64σ)
Cube strength (MPa) (150mm side)	66.0	58.0
Cylinder strength (mPa) (150mm dia. x 300mm long)	62.4	-
Cylinder splitting strength (MPa)	3.8	-
E cylinder (GPa)	42.8	-

Table 7.1 Concrete strength and elastic modulus properties.

The strains have been converted to stresses using the simple elastic relationship $\sigma_c = E_c.\varepsilon_c$. The maximum compressive stresses in the pipe barrels of schemes 3, 4 and 5 are typically 7N/mm^2, 3.5N/mm^2 and 20N/mm^2 which are approximately one third of the maximum compressive stresses given by the joint pressure cells in the respective schemes. Since stresses are typically below $^1/3\ f_{cu}$ the assumption of elastic behaviour is applicable for determining pipe stresses.



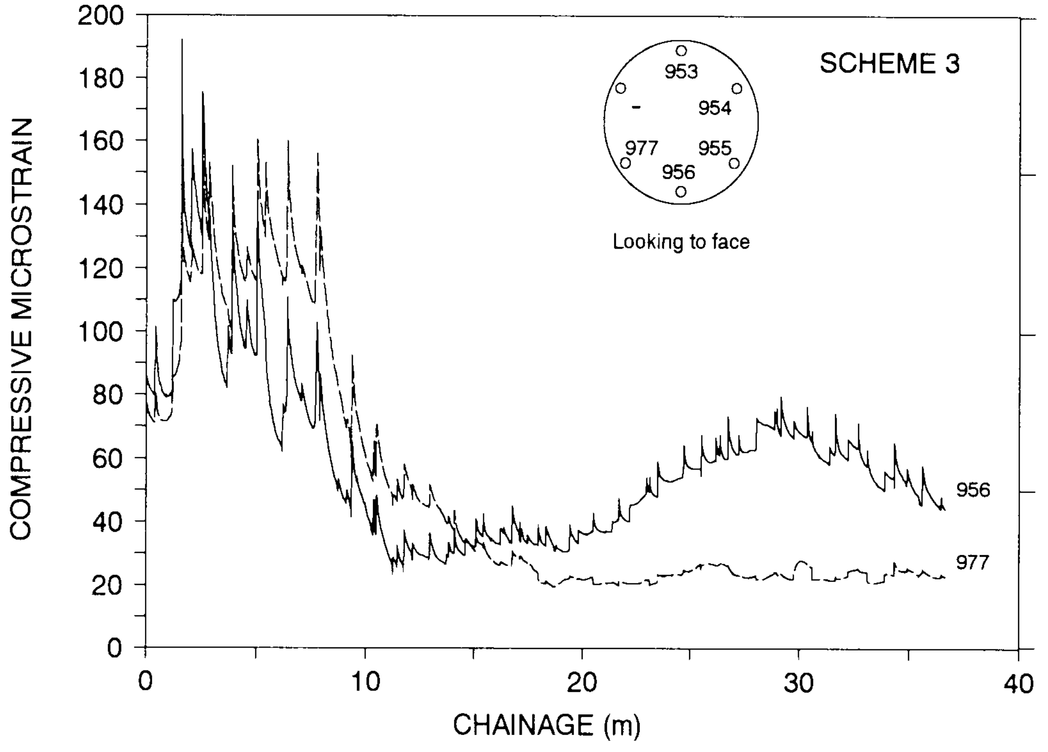


Figure 7.5 Average longitudinal pipe strains during jacking on Scheme 3.

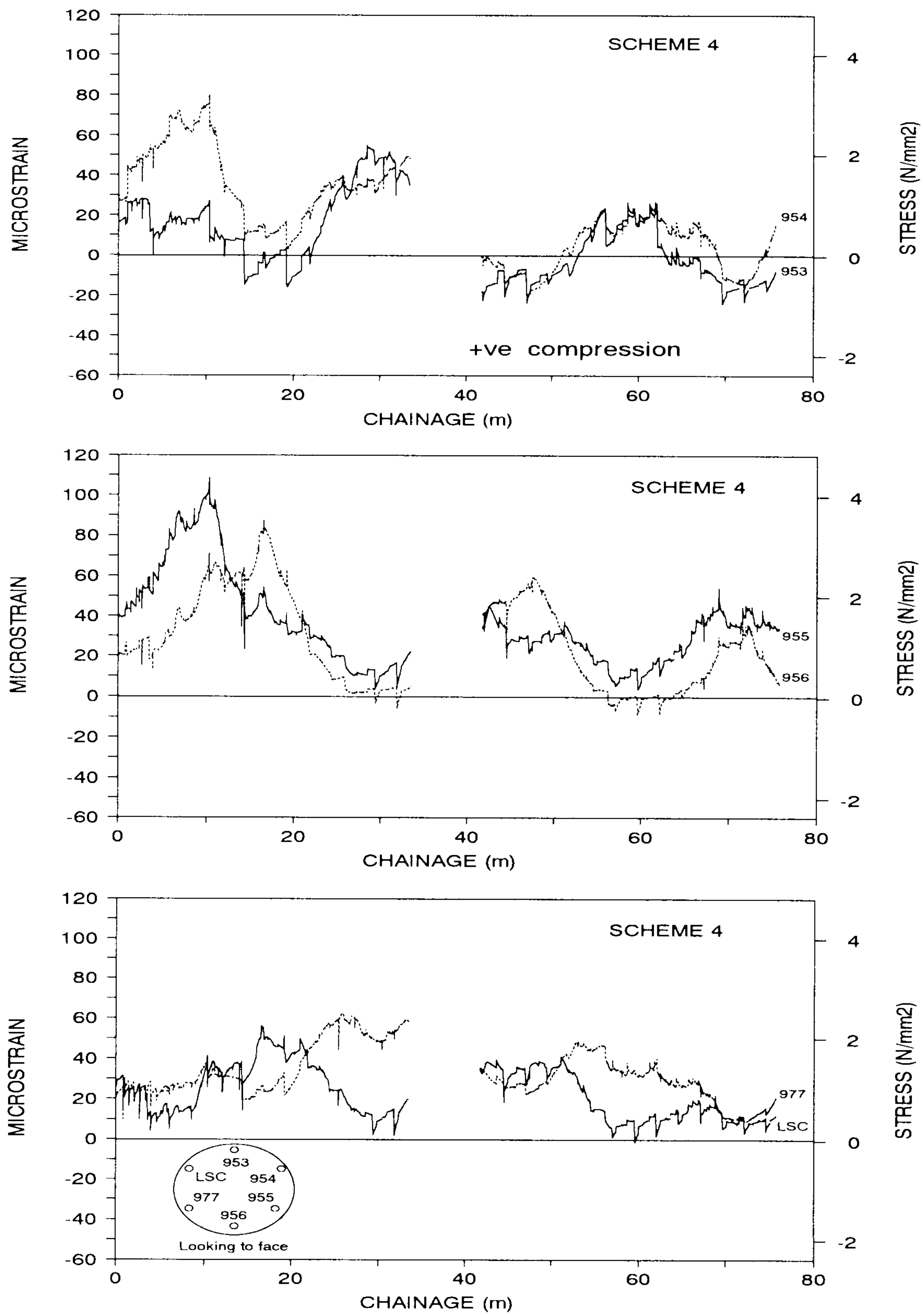


Figure 7.6 Average longitudinal pipe strains during jacking on Scheme 4.

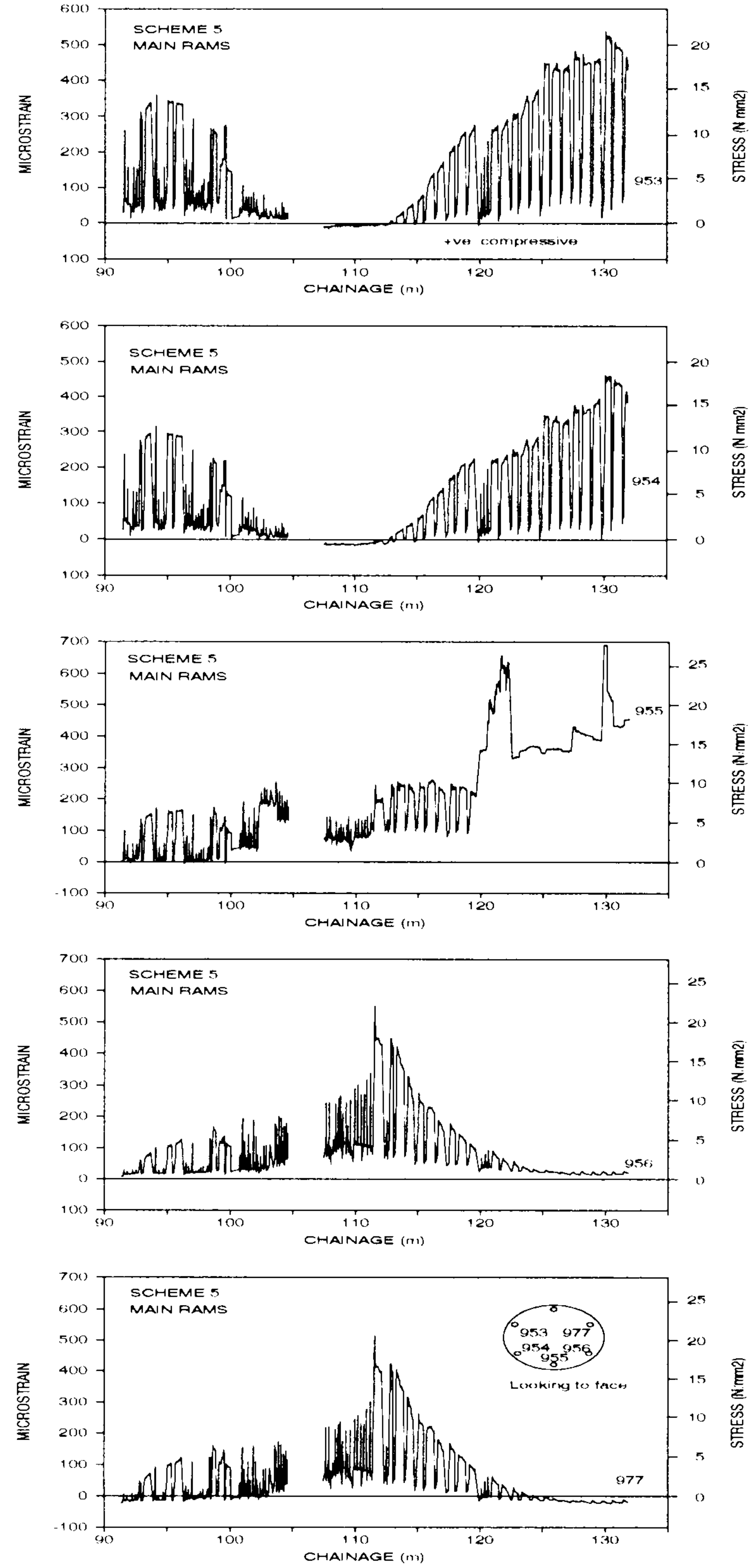


Figure 7.7 Average longitudinal pipe strains between ch.90-135m during main ram jacking on scheme 5.

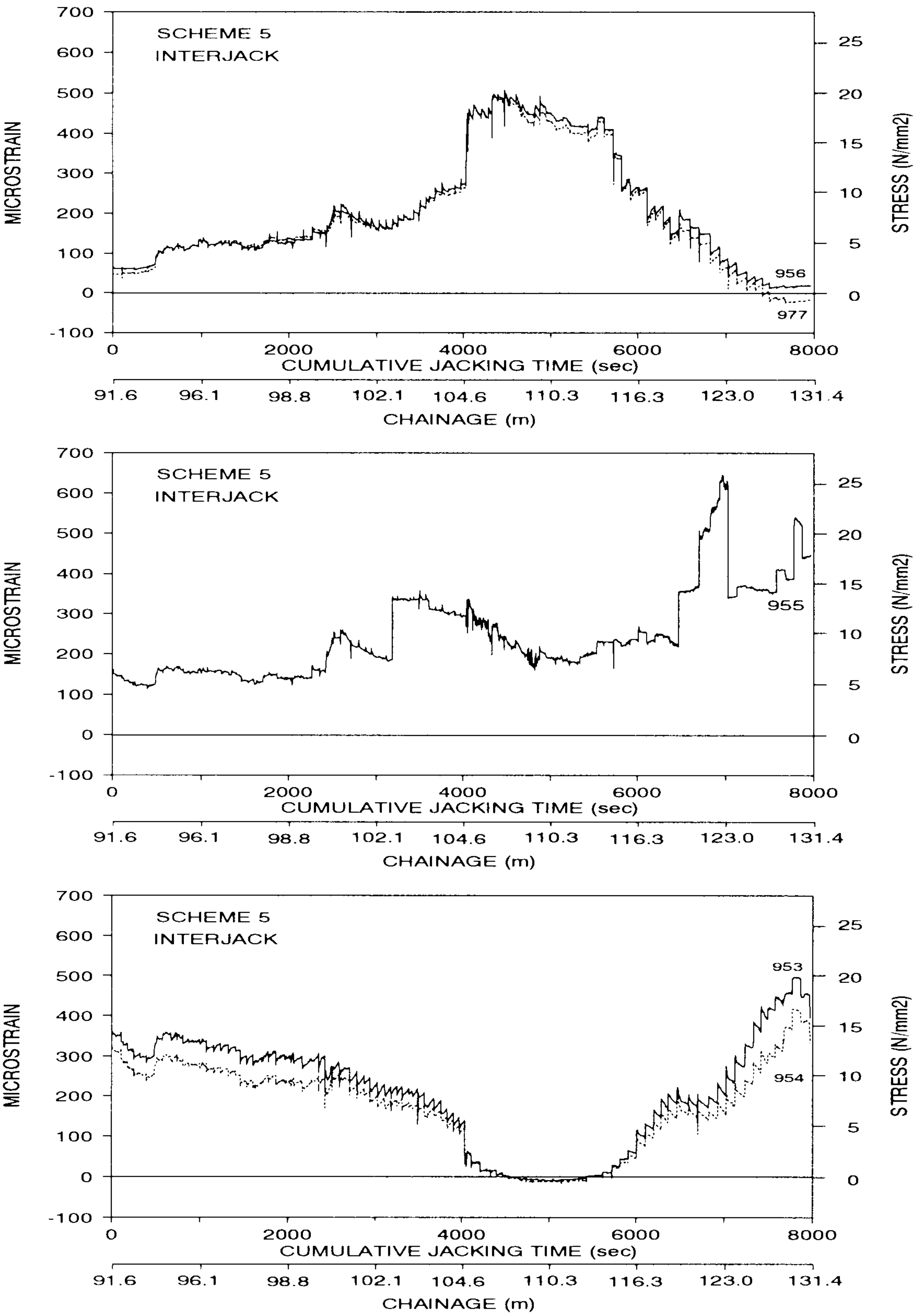


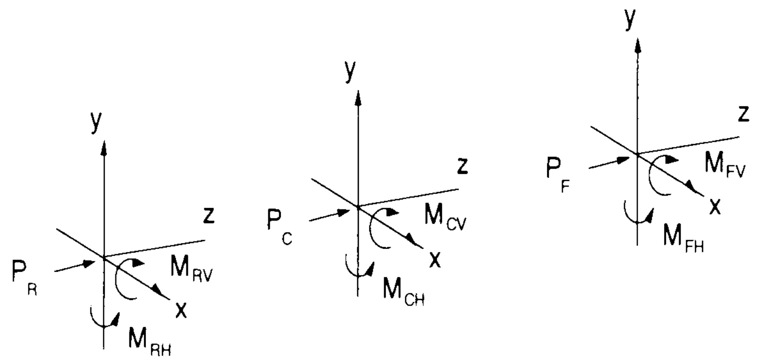
Figure 7.8 Average longitudinal pipe strains between ch.90-135m during activation of the interjack on scheme 5.

7.5 Elastic analysis of pipes

An assessment of the pipe strains based on measured interface stresses and deflected joint loading will be carried out for scheme 5. Two positions will be considered which correspond to the maximum right and left horizontal deviations. The pipe loading regimes and instrument locations are illustrated in Figure 7.9.

7.5.1 Example 1: Scheme 5, chainage 111.6m

Sign convention



Rear joint

P_R = 4525 kN

M_{RH} = 1159x0.65+(857+854)x0.6+(555+548)x0.46+(278+274)x0.25 = 2425 kNm

M_{RV} = (278-274)x0.6+(555-548)x0.46+(857-854)x0.25 = 6kNm

Front joint

P_F = -4552 kN

M_{FH} = -890x0.65-(795+719)x0.6-(700+549)x0.46-(385+331)x0.25 = -2240 kNm

M_{FV} = (114-69)x0.65+(331-385)x0.6+(549-700)x0.46+(719-795)x0.25 = -92 kNm

Interface loading

Assume that the lubrication pressure is 45 kPa and subtract from the maximum ie. 360kPa - 45kPa = 315 kPa. Assume further that contact is over the full pipe length and over a contact strip width of 0.4m based on the Roark and Young formula.

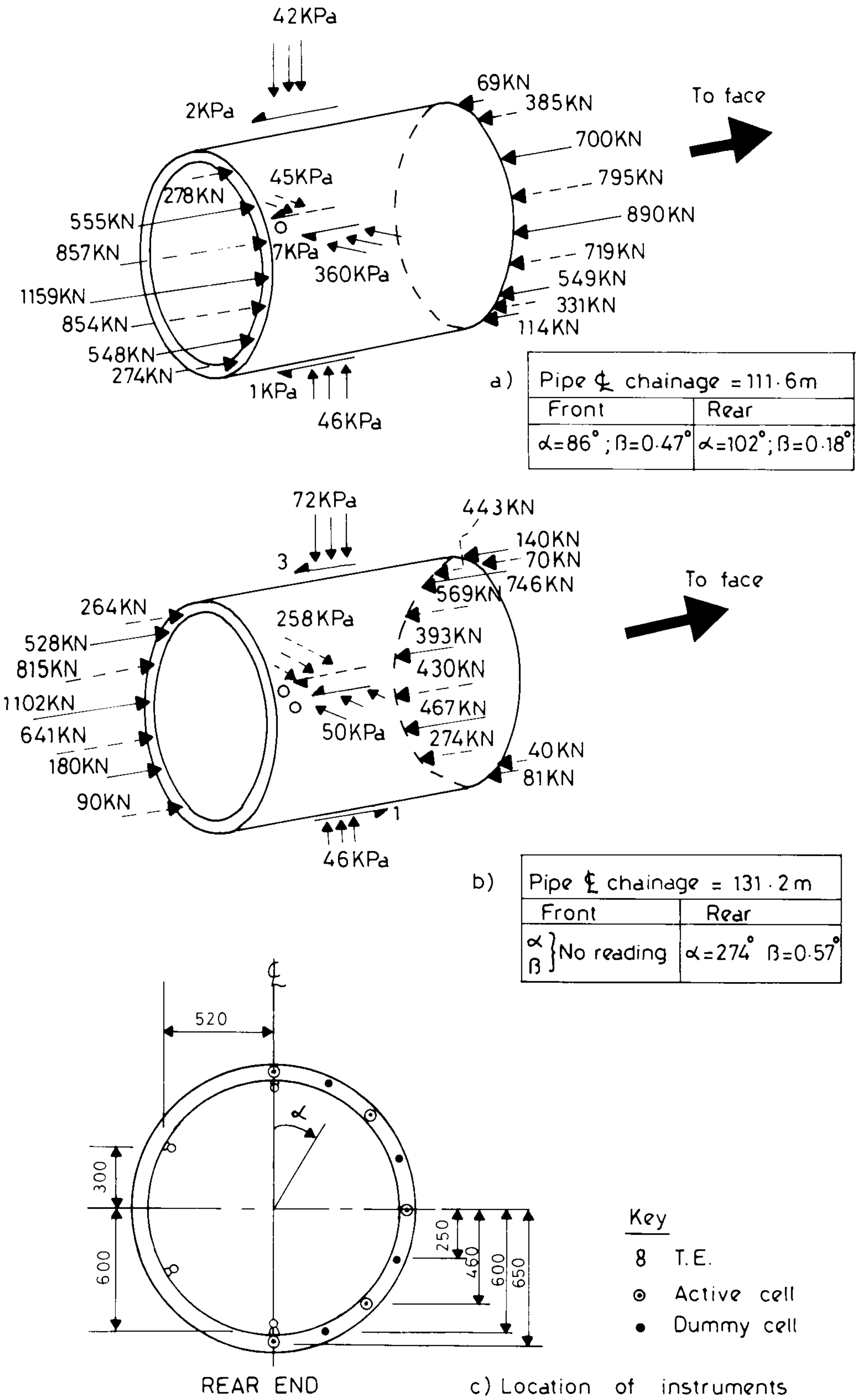
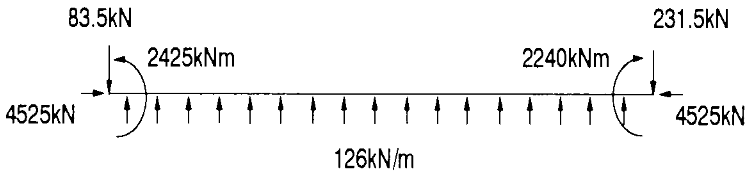
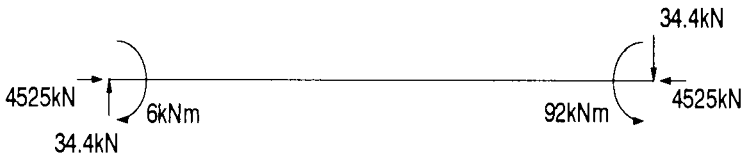


Figure 7.9 Pipe stress resultants at the positions of pipe damage in scheme 5.

Separating the total stress resultants on the pipe into horizontal and vertical planes:



HORIZONTAL PLANE



VERTICAL PLANE

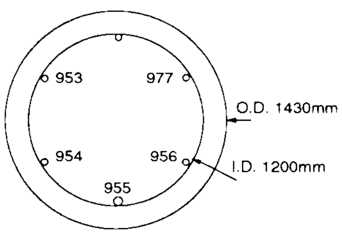
Stress resultants at the centre of the pipe

$M_{CH} = 83.5x1.25+2425-126x2.5^2/8 = 2431$ kNm

$M_{CV} = 34.4x1.25+6 = 49$kNm

Assume $P_C = 4525$ kN

Treating the pipe as a stocky column subject to axial load and biaxial bending



$A = 475 \times 10^3\ mm^2$

$I = 1.035 \times 10^{11}\ mm^4$

$$\sigma = \frac{P_C}{A} \pm \frac{M_{CH}x}{I} \pm \frac{M_{CV}y}{I}$$

For TE 953 $\sigma = 9.5 - 12.2 + 0.1 = -2.6$ N/mm^2 $\Leftrightarrow$ -60$\mu\varepsilon$ site value -6$\mu\varepsilon$

TE 954 $\sigma = 9.5 - 12.2 - 0.1 = -2.8$ N/mm^2 $\Leftrightarrow$ -65$\mu\varepsilon$ site value -7$\mu\varepsilon$

TE 955 $\sigma = 9.5 - 0.2 = 9.3$ N/mm^2 $\Leftrightarrow$ 217$\mu\varepsilon$ site value 245$\mu\varepsilon$

TE 956 $\sigma = 9.5 + 12.2 - 0.1 = 21.6$ N/mm^2 $\Leftrightarrow$ 505$\mu\varepsilon$ site value 550$\mu\varepsilon$

TE 977 $\sigma = 9.5 + 12.2 + 0.1 = 21.8$ N/mm^2 $\Leftrightarrow$ 509$\mu\varepsilon$ site value 525$\mu\varepsilon$

Good agreement is obtained between the calculated and measured values. Both the predicted and measured tensile strains are smaller than the limiting tensile strain, -89με, which supports the no crack observations in the pipe barrel made on site.

7.5.2 Example 2: Scheme 5, chainage 131.2m

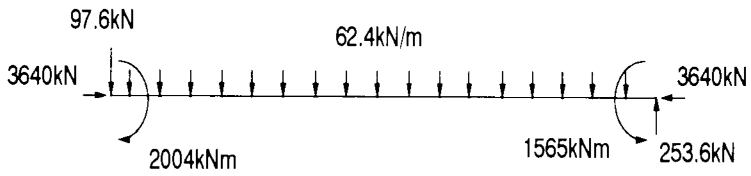
Rear joint	Front joint
P_R = 3620 kN	P_F = -3653 kN
M_{RH} = -2004 kNm	M_{FH} = 1565 kNm
M_{RV} = 308 kNm	M_{FV} = -321 kNm

Interface loading

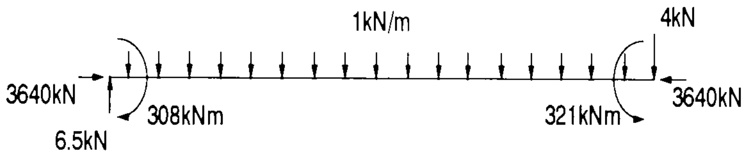
Horizontal ground stress 258kPa-50kPa = 208kPa. Elastic contact strip width = 0.3m.

Vertical ground stress 76kPa-50kPa = 26kPa. Elastic contact width = 0.04m.

Separating the total stress resultants on the pipe into horizontal and vertical planes:



HORIZONTAL PLANE



VERTICAL PLANE

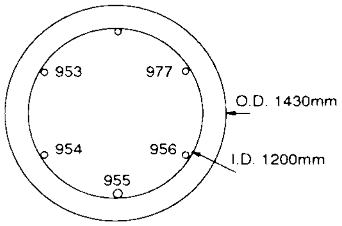
Stress resultants at the centre of the pipe

M_{CH} = -2004+97.6x1.25+62.4x2.5^2/8 = -1833 kNm

M_{CV} = 308 +6.5x1.25-1x2.5^2/8 = 315kNm

Assume P_C = 3640 kN

Treating the pipe as a stocky column subject to axial load and biaxial bending



$A = 475 \times 10^3\ mm^2$

$I = 1.035 \times 10^{11}\ mm^4$

$$\sigma = \frac{P_C}{A} \pm \frac{M_{CH}x}{I} \pm \frac{M_{CV}y}{I}$$

For TE 953 $\sigma = 7.7 + 9.2 + 0.9 = 17.8\ N/mm^2$ ⇔ $416\mu\varepsilon$ site value $492\mu\varepsilon$

TE 954 $\sigma = 7.7 + 9.2 - 0.9 = 16.0\ N/mm^2$ ⇔ $374\mu\varepsilon$ site value $408\mu\varepsilon$

TE 955 $\sigma = 7.7 - 1.8 = 5.9\ N/mm^2$ ⇔ $138\mu\varepsilon$ site value $520\mu\varepsilon$ (faulty inst.)

TE 956 $\sigma = 7.7 - 9.2 - 0.9 = -2.4\ N/mm^2$ ⇔ $-56\mu\varepsilon$ site value $19\mu\varepsilon$

TE 977 $\sigma = 7.7 - 9.2 + 0.9 = -0.6\ N/mm^2$ ⇔ $-14\mu\varepsilon$ site value $-17\mu\varepsilon$

Once again there is good agreement between the calculated and measured values which demonstrates the suitability of simple elastic assumptions for analytical purposes and the high quality of the field data. It is noted that the agreement is less good when strains are small and tensile. This is likely to be a function of the readings being close to the limit of sensitivity of the instrument which was found to be around $12\mu\varepsilon$ under laboratory conditions. Further work is needed to optimise the structural design of pipes, but it is clear that better packing materials could allow more of the potential strength of pipes to be achieved when being jacked with small misalignment angles.

CHAPTER 8:

CONCLUSIONS AND RECOMMENDATIONS

8.1 Instrumentation

One of the main objectives of the research was to develop suitable instrumentation and appropriate procedures for monitoring actual pipe jacks, without undue disruption to normal site operations. Instruments had to be selected or designed to operate in the aggressive tunnel environment, have minimal effect on the measured property and be sufficiently accurate and simple to calibrate. Where possible, advantage was taken of the reduced development costs of using commercially available instruments. The pipe joint pressure cells, jack load cells and Celesco displacement unit fall into this category and performed well. The remaining instruments were specifically designed and manufactured for the pipe jacking research and performed within specification, with the exception of the ground convergence indicator which had a poor field performance record. All of the equipment was designed for easy incorporation and retrieval from the permanent works and subsequent re-use. The only connections between the instrumentation in the tunnel and the surface based computer which controlled the data acquisition were a power cable and a signal cable which were disconnected and re-connected at the same time as the contractor's lighting cable. All instrument readings were recorded on a time basis for correlation with a detailed log of site activities.

8.2 Site work

During the planning stage it was expected that a sufficient number of potential sites would become available from which those approximating to previously determined optimum conditions could be selected. In practice, very few sites became available, and then often at very short notice, making incorporation of the research work into the contract more difficult than expected. Similarly, all site work was at the mercy of delays and problems outside the control of the research team. Finishing the work within the available timescale clearly illustrates the excellent cooperation provided by the various clients and contractors on each of the schemes. The main variable turned out to be the ground type with drives through stiff glacial clay, weathered mudstone, stiff plastic (London) clay, dense fine silty sand and loose sand and gravel, although it is regrettable that none involved an excavation in soft clay. Lubrication was used during the later stages of scheme 4 and throughout scheme 5. One expected variable, the type of pipe joint, appears to have disappeared. Either as a result of field experience or from the findings of the laboratory tests of Ripley (1989), the superiority of butt to in-wall joints whenever significant jacking forces are expected appears to have been accepted and steel-banded pipes were specified for all schemes.

8.3 Pipe joint behaviour

The transfer of load through pipe joints is dominated by effects of angular misalignments between successive pipes. These misalignments occur due to variations from exact line and level; once they have occurred they appear to change little either due to application of jacking load or the passage of subsequent pipes. Thus once a critical misalignment is established, all pipe joints passing that chainage will be affected by it. Control of line and level is often poorest at the start of a drive; this is also the location at which maximum pipe loads will occur later in the drive, giving rise to severe pipe loading conditions.

Typical misalignment angles on nominally straight drives, within specified line and level tolerances, were found to be mainly between 0.1° and 0.3°, but could reach 0.5°. Significant stress concentrations were observed in the joints (even when a packer of compressible material such as fibreboard was used) and these correlate well with the joint misalignments. Angular deviations of 0.1° confined loading to approximately half the end area of the pipe, and 0.3° to about a quarter, the exact amount depending on pipe size, packer properties and the total load passing through the joint. Bearing stresses in excess of $58N/mm^2$

occurred on one site with a joint deflection angle of 0.47° and large jacking forces. Accurate control of angular deviations during construction is therefore essential for the successful completion of highly loaded pipe strings. The establishment of actual angular deviations achieved in practice allows reasonable specifications in terms of allowable angular misalignments to be considered, instead of or in addition to current limits on line and level. It would appear that the Draft Swiss code of practice, Craig(1983), has got it more or less correct with a 0.24° limit on angular deflection, when high axial loads are expected. It should be borne in mind, that additional angles can occur due to a lack of pipe end squareness, and in theory these could add together in the worst case. The field observations and an end squareness audit carried out at three of the UK pipe works has established that this effect could be ignored. The field data have also led to a simple method for making decisions about corrective action based on minimising angular deflections, Milligan (1992); at present this is being retained as confidential information by the industrial partners in the research.

In order to avoid damage to joints due to overstressing when misalignment has taken place it is important to estimate the stress concentrations that occur with different joint deflection angles. As a first approximation the Concrete Pipe Association of Australia (1983) recommend that stress concentrations of about 3 times the uniform joint stress should be expected. This is also the basis of the UK pipe manufacturer's practice of specifying a limiting evenly distributed end stress value. The site data however suggest that localisation of transfer stresses can be much more severe with the rule of thumb value closer to a factor of 6.

Exact correlation of joint angles and joint stresses is complicated by the non-linear and stress-history related behaviour of the packing material. However careful back-analysis shows that the linear stress approach of the Australian CPA can adequately match the measured stresses, and could be used to define allowable jacking loads on pipes on the basis of pipe size, packer properties, concrete strength and angular misalignment. Adopting such a model as the basis for a design approach requires safe limits to be defined for the principal parameters, namely packer stiffness and thickness after permanent compaction, and allowable joint bearing stress. The field data show that the stress-strain behaviour of the packing material tends towards that measured in unload-reload cycles in laboratory tests and the corresponding high stiffness gives more severe stress concentrations and should therefore be used in design. Consideration of the joint geometry and the localisation of force over small areas suggests that local bearing stresses between 80% of the characteristic concrete cube strength and the lower limit of the joint face strength test of BS 5911:Part 120 can be tolerated.

The packing material plays an important role in distributing stress concentrations caused by misalignment. It is therefore imperative that guidance on preferred material types, properties and geometry are included in BS 5911:Part 120. Standardisation of its use could be achieved by considering the packer as part of the pipe and fixing it at the pipe works.

8.4 Pipe barrel behaviour

As the pipe string "wriggles " through the ground, the angular orientation of the position of maximum compression moves around the joint. Greater movement is experienced in the pipes immediately behind the shield which is consistent with expectations. If the centres of pressure are at the same angular position at both ends of a pipe, the load is transmitted essentially along one edge of the pipe; if they are out of phase by 180 degrees, the pipe will be loaded across a diagonal. The site measurements have shown that pipeline misalignment generally takes the approximate form of large radius curves with the resulting load paths within ±45° of the single edge loading case. The extreme diagonal loading condition would only occur under exceptionally poor control of the shield. The overall behaviour of a pipe under various field conditions shows that typical maximum stresses are about one third of the peak joint stress. In absolute terms the values are within the usually assumed linear elastic range for concrete. Even under the extreme conditions of loading in scheme 5, in which local compressive failure in a number of joints was experienced, the pipe barrel compressive stresses peaked at $20N/mm^2$ and tensile stresses did not exceed the tensile strength of the concrete to produce circumferential cracks.

Further work is needed to optimise the structural design of pipes, but it is clear that improved packing materials and / or better end reinforcement details could allow more of the potential strength of pipes to be achieved. Current reinforcement arrangements are generally based on empirical rules, with the main consideration being the provision of sufficient hooped reinforcement to withstand long term ground loads usually based on trench loading conditions. The field data on the other hand have shown that installation loads are considerable and the ground loading regime during installation is significantly different to assumed long term conditions. Fortunately the evolved pipe wall thicknesses are sufficiently large to ensure that longitudinal reinforcement is only needed for shrinkage and handling requirements. Economies in pipe wall thickness will however require proper consideration being given to installation loads. The fieldwork has provided a large bank of high quality data which can be used for this purpose.

8.5 Pipe-soil interaction

Data have been collected on the interaction between pipes and ground; sliding resistance at the pipe-soil interface in different ground conditions; and construction related factors such as time and rate effects, pipeline misalignment, and the effectiveness of lubricant injection, all of which affect the total jacking resistance. It is clear from the interface stress measurements that the very approximate average resistances to jacking through different ground conditions traditionally adopted in designing pipe jacks represent a gross simplification of an extremely complicated case of soil-structure interaction.

The interaction mechanisms are highly dependent upon the short term stability of the excavation. In general, pipe jack shields are usually of slightly larger diameter than the outside of the pipes. This "overbreak" considerably reduces the contact between pipes and ground and hence the resistance to jacking. For stiff soils at low cover depths the overbreak remains open and the pipes slide along the base of an open bore; readings are then only obtained from the instruments in the bottom of the pipe. This happened through most of schemes 1 and 2.

Scheme 3 was at a greater depth and in highly plastic and overconsolidated London clay. Here some very high contact stresses were registered, sufficient to cause damage to most of the stress cells quite early in the drive. It appears that the pipes were "squeezed" by high horizontal stresses, exacerbated by horizontal deviations of the pipeline. For the period for which the cells were operational, the interface resistance correlated very well with published data from instrumented driven piles and laboratory interface ring shear tests. In scheme 4, some collapse of the silty sand on to the top of the pipe occurred, while stresses on the side walls varied from zero to quite large values dependent upon the direction and magnitude of pipe misalignments. The misalignments are therefore highly significant in their effects on total jacking force as well as on local joint stresses. Peak values at mid length of a pipe typically occur over short 1m to 2m lengths and correspond to positions of maximum deviations from line and level. Theoretically, larger stresses can be expected at the pipe ends but no attempt was made to measure them during this project.

Detailed examination of individual pushes has highlighted a partially drained frictional material response in both cohesive and non-cohesive ground. The data suggest that under typical radial stress levels, slippage may be occurring at the interface between the pipe and soil and not within the soil itself. The response appears to be related to the magnitude of the radial stress, surface characteristics of the pipe and the composition and moisture content of the soil. It is further suggested that the soil at the interface is probably in an unsaturated state (even in originally saturated ground). The delay between the time of the original excavation and the instrumented pipe arriving allows the surface to dry out with additional drainage taking place during the shearing process as a result of the high permeability of the concrete. There is limited evidence that the material at the interface approaches a critical state. Use of peak friction angles from drained shear box tests should provide conservative estimates of the appropriate coefficients of friction. The main area of uncertainty remains the prediction of imposed radial loading during jacking. For stable bores, however, the jacking load can be estimated from simple pipe self weight sliding resistance, using an appropriate large-displacement (residual) or peak interface friction angle.

The undrained shear model of Haslem (1986) requires failure to take place within the soil, and seems to be appropriate for the temporary inundation phase of scheme 1, and also at radial stress levels in excess of 450kPa in scheme 3. The laboratory interface tests on London clay carried out by Tika (1989) support the observation in scheme 3, illustrating rupture occurring 0.1mm within the body of the soil under a radial stress intensity of 484kPa.

When stoppages occur in cohesive ground a greater force is needed to restart the process than would have been required to maintain continuity. The time-dependent effects are very marked on scheme 3 - total jacking forces increased if the pipe line was stationary for even a few minutes, and the typical periods of a few hours between major pipe pushes allowed increases of over 50%. Although the exact mechanisms involved require further investigation, detailed examination of the less pronounced effect in scheme 1 shows that positive pore pressures generated during the jacking process dissipate during the stoppage leading to an increase in the effective radial stress. The rapid and repeatable response of the mechanism appears to be a function of the very limited depth of the interface zone. The extremely small drainage paths, equivalent to 0.1mm, allowing transient outflow and inflow of water within a matter of seconds, even in low permeability clays.

Use of lubricants was avoided in the first three drives, introduced during the later stages of scheme 4 and used throughout the machine drive of scheme 5. The effectiveness of lubrication was seen to be dependent upon many factors, including changes in ground conditions and correct selection of equipment and procedures for the injection system. Scheme 4 used both bentonite and a polymer material but with insufficient pumping capacity. The overbreak was never completely filled with lubricant, resulting in large local variations in its effectiveness, and typical interface friction angles varying by a factor of 3. By contrast scheme 5 was through loose sand and gravel mostly below the water table; had the ground closed on to the pipe the frictional resistance would have been too great for the lengths of drive required to have been feasible. This drive was therefore fully lubricated with a bentonite slurry injected into the annulus of the overbreak. The instrument readings showed that the pressurised slurry formed a stabilised zone of soil around the pipe preventing large effective contact stresses from developing; and having done this, formed a layer of bentonite gel between pipe and soil with a very low resistance to shear. In addition the tunnel surveys showed that the pipeline was bodily lifted within the excavated bore by the introduction of the slurry. It has been argued theoretically that the pipes should be buoyant in the slurry, but this has been disputed and these site measurements are perhaps the first to demonstrate it occurring in practice.

8.6 Further work

The success of Phase 2 of the research has led directly to two further contracts, which will start at the end of 1992, supported by new grants from SERC and continued funding from the PJA and water companies. Sufficient data have now been collected to allow analytical modelling of pipes under load which can be used to extend the field and experimental findings. Detailed studies of the geometry, reinforcement , sealing arrangement and optimum packing material in the joint detail is the most urgent task. Development of a computer model could be extended to full three dimensional analysis of pipes under a variety of load systems leading to rational design approaches and reinforcement layouts.

The geotechnical aspects of the problem require further site and laboratory based work due to the complex nature of the behaviour. In particular variations in pressure along the length of a pipe and consistent pore pressure data are required, preferably in stiff glacial clay and London clay to duplicate stage 2 observations, together with a soft normally consolidated clay and two sandy gravelly materials. Each site should include extensive soil testing and ground movement, piezometric and insitu stress measurements close to the pipeline. In parallel there is a requirement for more detailed laboratory interpretation of the interface stress data, possibly treating a single push as a large-displacement interface shear test under a variety of stress paths, usually involving increasing normal as well as shear stresses. This work would allow failure mechanisms at the interface to be identified, time effects to be modelled, and small changes in properties such as moisture content to be investigated in a variety of real soils.

The amount of resistance encountered at the face depends upon ground conditions and the measures required to support the face. It was shown during this work that it is related to the edge cutting resistance of the jacking shield and can account for a significant proportion of the total jacking load. Clearly any method for predicting jacking resistance will benefit from quantitative information on the total load transferred between the shield/machine and the lead pipe. It is suggested that future site work should consider incorporating pressure cells in the lead joint, and maintaining a rigorous log of shield cutting practice or TBM face pressures.

The resultant increases in jacking capacity which may result from improved joint design will clearly focus attention on the most cost effective way of providing the thrust forces. The temptation will be to look for extra capacity from the thrust wall in the jacking pit and thus minimise the use of interjacks. A radical look at thrust wall design, based on limiting soil strains under cyclic loading conditions, is therefore worthy of consideration.

The success of the site monitoring has been very dependent upon the correct design of instruments and procedures. A number of modifications should be considered for future site work.

a) The contact stress cell and pore pressure instruments should be combined so that potential phase differences in readings due to different positions on the interface are minimised. This has the subsidiary benefit of reducing the number of openings in the pipe wall.

b) The number of instruments necessary for the intensive interface monitoring phase cannot reasonably be incorporated into a pipe which is part of the permanent works. Either sites will have to be selected which enable the instrumented pipe to be parked and broken out at intermediate manholes as in scheme 5, or the instrumentation will need to be incorporated into a special interjack skin locally stiffened with removable stiffening plates. Both frictional similitude on the outer face and the axial and flexural stiffnesses of the can need to be matched to that of a standard pipe.

c) Precise values of pipe velocity and the duration of pipe movement would be better recorded at the location of the instrumented pipe. This could be achieved by designing a wheeled tachometer positioned at the base of the pipe. The problems which beset the ground convergence indicator would have to be overcome.

d) All inwall instruments should be provided with connectors close to the device to facilitate cable removal prior to overcoring, if this remains the preferred method of extraction.

8.7 Concluding remark

The programme has been an excellent example of cooperation in research between industry, academia and government. Good managerial control, a lot of hard work and enthusiastic support from all the parties involved have been the key ingredients which have enabled the project to be completed within budget and subject to only minor delays.

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