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Packing Material in Jacked Pipe Joints

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Laboratory research has been conducted at Oxford University, during the past three years, investigating concrete pipe behaviour during installation by the jacking method. The work has investigated many aspects of the pipes' ability to be jacked and this paper highlights one aspect of that work, namely the use of packing materials in pipe joints. The laboratory exercise has been carried out using scale model pipes. It has investigated joint profiles and differences in their behaviour, and the advantages and disadvantages of using compressible packing materials in the joint. The effect of misalignment between consecutive pipes and how this influences their ability to transmit jacking load is presented. Different joint packing materials have been tested to establish their ability to distribute stress concentrations between misaligned pipes and pipes whose ends are not perfectly square to each other. A summary of allowable jacking loads on model pipes for various deflections is included.

1 INTRODUCTION

The research programme into the performance and behaviour of jacked pipes began at Oxford three years ago. The need for research has been highlighted in reports and presentations, Craig (1983) and Kirkland (1982). As a result a substantial research programme was established which would begin with laboratory investigations. Several topics were listed as areas where more knowledge was required to fully understand the behaviour and limitations of pipejacking. The Pipe Jacking Association (PJA), Concrete Pipe Association (CPA) and Construction Industries Research and Information Association (CIRIA) have been instrumental in establishing a research programme with full support from the civil engineering industry. The research is additionally supported by the Science and Engineering Research Council (SERC).

Recent reports in Underground magazine, September 1987, September 1988, of clay pipe failures have highlighted further the urgent need for a full understanding of pipe behaviour in this rapidly expanding market and for experienced personnel to be used to supervise installation.

Microconcrete pipes are used to study the following points in order to improve our understanding of the behaviour of pipes during and after installation:-

- 1 The distribution of concrete strains during pipejacking and how these change when soil loading is transferred to the pipe as it is installed in its final position.
- 2 Parameters relating the allowable jacking load to the soil type and angular misalignments between consecutive pipes.

- 3 The distribution of stress concentrations between misaligned pipes, the use of packing material to reduce this concentration and to find a suitable joint profile to minimise stresses and retain pipeline watertightness and integrity.

- 4 The possibility of using a suitable jacking strength test to supplement the existing British Standard tests.

- 5 The effect of soil stress level on the behaviour of the pipe during jacking, in particular its changes in shape and orientation with different applied jacking loads.

- 6 The most suitable and economic measurements to be made to monitor performance of a full-scale pipejacking operation.

- 7 The various failure modes occurring in jacked pipes and how pipes can be designed, installed and monitored to predict and prevent such failures.

2 JOINT PACKING MATERIALS

Packing materials are recommended for use in jacking pipe joints but no details of their nature or exact positioning in the joint were available when this research commenced. The packing material is used as an aid to distribute areas of high stress concentrations over larger pipe end area and thereby reduce the maximum stress level. In the past packing materials have been used intermittently by some pipejacking contractors and totally avoided by others. When packing materials have been used they have been associated with attempts to stop spalling of the concrete at the pipe joint.

The aim of this series of tests was to assess the advantages and disadvantages associated with the use of packing materials. The tests would lead to recommendations on the use of materials, the nature of the material to be used, how thick it should be and where it should be placed. The materials were assessed under cyclic load conditions and tested in various conditions: as purchased; saturated; saturated and room dried.

The ideal material is considered to be one that compresses greatly when loaded and recovers its thickness when unloaded, and a material that does not induce tensile strains in the concrete that are any greater than those associated with the axial load applied. The packing material should have a Poissons ratio in compression near to zero. It is emphasised that these are ideal requirements and properties and the aim of the test programme was to find the packing material most able to meet the requirements.

Loading of packing material should be between concrete surfaces to simulate the friction between packing material and pipe end that would be experienced in practice.

Part of the function of the joint packing material is believed to be prevention of stress concentrations created by high points on the concrete surface. This is especially relevant to the trowel finished end of a vertically cast pipe.

3 TEST PROGRAMME

A series of cyclic loading tests were carried out testing various prototype packing materials used in jacked pipe joints. The tests were conducted using 100mm sided concrete cubes as loading media with packing material sandwiched between them. The concrete cubes were instrumented with electrical resistance strain gauge rosettes and the changing thickness of the packing material was monitored with linear variable differential transformers (LVDT's). Load was applied using a hydraulic jack and the arrangement of the test equipment and transducer positions are shown in Figure 1.

Three electrical resistance strain gauge rosettes were used to monitor magnitudes and directions of induced strains. Four LVDT's monitored the gap between the concrete cubes and hence the compression and recovery of packing material thickness. The four LVDT's also enabled assessment of any eccentricity in the loads applied. Initially a large number of different materials were tested and further testing was carried out on selected materials as a result of these initial tests.

Tests were carried out on a large number of material types and all the materials were loaded cyclically using the mechanism drawn in Figure 2. This mechanism controls the hydraulic jack by moving the control handle using a linkage and cam driven by a motor. The profile of the cam and motor speed provided a cyclic duration of 150 seconds during which the packing was loaded for 10 seconds.

A programme was written which enabled data to be recorded during 40 second time intervals which allowed recording of data before, during and after loading. Data was recorded during every load application for the first ten applications of load and thereafter recordings were made every tenth cycle. The first and last cyclic load applications were carried out manually and allowed data to be recorded as load was being increased and decreased. This data recording process would give a clear indication of how the packing material's properties changed during the application of approximately 360 load/unload cycles.

Following initial testing further cyclic loading was carried out on the following materials at the stated stress levels and material conditions:-

Stress	Material	Condition	Initial Thickness
50N/mm ²	Dense fibreboard	As supplied	18mm, 12mm, 6mm
	Chipboard		18mm
	Exterior grade plywood		18mm, 6mm
15N/mm ²	Dense fibreboard	As supplied	18mm, 12mm, 6mm
	Chipboard		18mm, 12mm
	Exterior grade plywood		18mm, 6mm
	German chipboard		22mm
15N/mm ²	Dense fibreboard	Saturated	18mm, 12mm, 6mm
	Chipboard		18mm, 12mm
	Exterior grade plywood		18mm, 12mm, 6mm
	German chipboard		22mm
	Softboard		10mm
15N/mm ²	Dense fibreboard	Saturated then room dried	12mm, 6mm
	Chipboard		18mm, 12mm
	Exterior grade plywood		18mm, 12mm, 6mm
	German chipboard		22mm
	Softboard		10mm

Effects of stress concentrations applied to concrete have been researched previously by Williams (1979), who's report reviews current analytical procedures and presents results from a test programme to assess the concrete's ability to sustain high stresses over limited load areas. Various types of material were used and all tests were loaded to failure. Figure 3 presents results showing how the concrete bearing stress is increased as the load area is decreased and is reproduced from a report by Williams (1979). Bearing stress can be up to four times the concrete compressive strength before failure occurs. Included in the report are details of the effects of loading onto a trowelled surface which can reduce the concrete bearing stress to half the value compared with loading on flat surfaces. The results from these tests are of some use to the current research but take no account of distributed stresses; all the bearing stresses applied were concentrated and uniform.

4 RESULTS

Examination of the initial cyclic loading test results revealed several points of interest. It was clear that packing materials were being permanently deformed and the material properties were totally different after one load application had been performed compared to the properties of the material before testing began. Consideration of the conditions in which the packing material would be used led to all analysis being carried out on results recorded subsequent to application of the first load.

Some of the initial materials tested were immediately ruled out of the subsequent test programme. 18mm thick planed timber compressed along the direction of its grains rather than in the direction of load application which led to displacement of one concrete cube relative to the other. The timber was chosen as knot free and was not considered to be a viable proposition for a packing material.

Blockboard disintegrated on application of load and was not tested further; Hardboard was obtained in 3mm thickness only and was not compressible; 12mm softboard compressed to 4.5mm and then resembled dense fibreboard. Good one side shuttering plywood compressed unevenly due to many gaps in the ply as it had been laminated. Further testing and analysis was carried out on dense fibreboard, plywood and chipboard.

4.1 Stress/strain relationship

Data has been processed to give details of the stress and strain conditions induced in the packing material. Stress is obtained by dividing the load applied to the packing material by the area of material under compression and load is assumed to be distributed uniformly. Strain is the change in thickness during the current load cycle divided by thickness of packing material at the start of the cycle. The graphs are presented in Figure 4 but thicker packing

materials are shown to be strained to a lesser extent than thinner materials, yet are more compressible and therefore better at distributing stress concentrations.

If the compressive properties of the packing material are the only consideration we are looking for the material with maximum compression. This is not evident from the stress/strain plot presented and the data is replotted as compression in millimetres during the current load cycle against induced stress in Figure 5. The first of these plots shows the greater compressibility of thicker packing materials and the second plot shows the difference between similar thickness packing materials, depicting 18mm dense fibreboard as the most compressible material.

4.2 Effect of cyclic loading

A graph plotting time against load is presented in Figure 6 and indicates the magnitude and duration of load applied during a typical joint packing material test. The effect of cyclic loading on a pipejack is important due to the method of pipe installation. Many of the pipes are loaded a large number of times during installation i.e. on a 100 metre long pipejack without an interjack station the average number of load applications to the pipes is 200. If an intermediate jacking station is used it is likely that pipes nearest to it will be subjected to an even larger number of load cycles and the largest magnitudes of load.

The tests on the packing materials showed how the material properties changed during the application of a large number of load cycles. A graph showing how the material is compressed and recovers its thickness to a lesser extent with each load application is presented in Figure 7. The behaviour is much more noticeable in tests on saturated materials due to drying out of the material during the test; the duration of each test was fifteen hours.

4.3 Variation of packing material thickness

During the test series different initial packing material thicknesses were tested and the difference in their behaviour under compression loads can be seen in Figure 5 where more compression takes place on thicker materials. It is not until the geometry of a jacking pipe is considered that the effects of different compressibilities of packing materials can be fully appreciated.

When considering pipes in a straight line with perfectly square ends relative to each other, the packing material compression and hence its thickness have no effect on the distribution of stress concentrations because stress is uniform. It is not until pipes are considered whose ends are not square, even and aligned that the requirements of the packing material compression need be considered. It could easily be argued that this is the circumstance on all pipejack joints and that the ideally square pipes never occur.

Materials that are more compressible when subjected to high stresses are of most benefit in assisting distribution of stress concentrations. The ideal scenario would be to find a material that, resulted in equal stresses over the pipes' end area whatever the magnitude of deflection between pipes or unevenness in pipe end profile. This is clearly impossible to achieve so a material that has a lot of compressibility in order to distribute stresses over as great an area as possible is required. To give some indication as to how deflected pipes affect the compressive requirements of the packing material the table in Figure 10 can be examined. The table details variations in pipe joint gaps related to angular deflections.

4.4 Effect of material conditions

The test results were greatly influenced by the history of atmospheric conditions the packing material had experienced. It was important to test the material in the various conditions to simulate possible treatment and conditions the material might be subjected to on a construction site. It is possible that the packing material would be left in the open air, saturated in a pipejack with water running in the invert or saturated by ground water on the external face of an in wall jointed pipe.

The differences in the material compressive properties experienced during the tests are presented in Figure 8 where it can be seen how saturated materials are much more compressible than materials that have not been treated. Packing material that has been saturated and then dried at room temperature returns to the same thickness when compressed as material that had not been saturated but has more thickness recovery when unloaded. These behaviours were found to be the case for all materials tested and clearly demonstrate that the saturation of the packing material is not detrimental to the compressive behaviour or integrity. If it were practically possible, saturation of packing material would be a distinct advantage in distributing jacking loads over larger areas.

4.5 Differences between materials

The materials disregarded at the early stage of this test series were presented in Section 2; they were generally degraded when compressed. Comments in this section will be about dense fibreboard, chipboard and plywood.

Analysis of the results has been carried out on data from the last load cycle applied and the compression of the packing material has been analysed to provide data on jacking load capacity. This data has been based on assumed maximum allowable stresses and has been calculated for packing material on a 1200mm external diameter and 900mm internal diameter

pipe. The data assumes the full end area of the pipe is used for load transfer and is presented in Figure 9 and Figure 10 by comparing stress distributions for various angular deflections between pipes and relating these to total jacking loads. It can be seen that as joint deflections increase so load is rapidly concentrated onto a small area of the end of the pipe and hence results in reductions in jacking load capacity. Distribution of stresses on the end of a pipe is the same for a given distance from the edge of the pipe irrespective of pipe diameter, but increased end area results in larger loads being acceptable.

At this stage it can be commented that dense fibreboard gives greatest capacity for transmitting jacking forces between pipes and for distributing concentrations of stress. Saturated packing material is beneficial but perhaps not practicable. In general water enhances the ability of a packing material to distribute load and the material is beneficial towards distribution of loads on pipe ends which are not square.

Packing material properties on deflected pipe joints should be related to deflections measured relative to the pipe end squareness. This gives the best indication of the magnitude of joint gap the packing material must bridge in order to transfer load. Squareness of pipe end is thus a very important factor in considering a pipes' jacking capacity. It should be noted that pipes complying to end squareness tolerances of the British Standard could have a joint gap of up to 8mm on a 900mm internal diameter pipe. This equates to a deflection of 0.5 degrees even before pipes are deflected relative to each other. The British Standard also allows for pipe squareness to be 4mm across the wall thickness as shown in Figure 11.

All materials have been analysed and those chosen to best suit the compressive requirements are dense fibreboards. This is the most suitable material chosen due to its ability to transmit axial load with a deflection of 0.2 degrees at the joint assuming all materials are subjected to the same maximum stress. The same conclusion would be reached for other deflection angles and different stress levels.

4.6 Strain gauge readings.

Mounting positions of electrical resistance strain gauge rosettes have been shown in Figure 1. Analysis has been carried out to find principal strain magnitudes and directions and concrete stresses can be assessed below the elastic limit of the concrete. Figure 12 presents data plotting measured maximum and minimum principal concrete stresses compared to stress applied by the hydraulic jack. The maximum concrete stresses were always measured in alignment with the direction of load application immediately beneath the packing material. Tensile stresses are a potential problem with the concrete pipe; the magnitude of tensile

stresses in the concrete, measured during tests on the dense fibreboard, did not exceed the tensile strength of the concrete.

5 SUMMARY OF RESULTS

A summary of packing material test results would suggest that:-

- 1 Thick packing materials are most beneficial.
- 2 Deflection angles greater than 0.20° combined with high axial loads are to be avoided; angles are to be measured with load applied.
- 3 Wet materials improve load transfer capabilities by up to three times.
- 4 Cyclic loading conditions need to be considered due to permanent packing material compression.

Work is to be continued for a further three years during which time it is planned to instrument and monitor five pipejacks in the field.

6 ACKNOWLEDGEMENTS

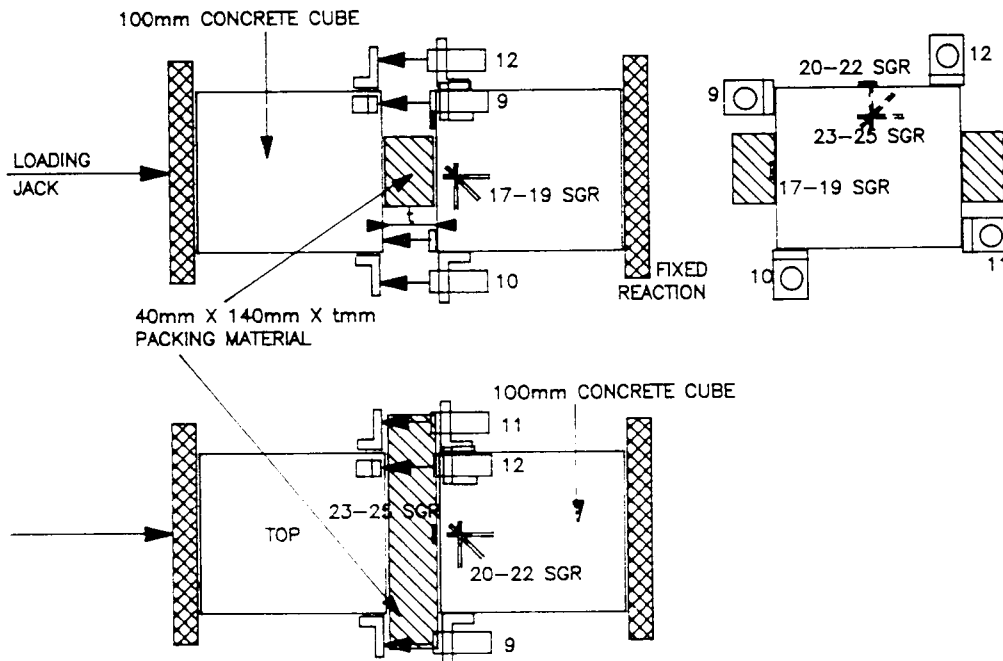
I wish to express my thanks for the support given to the project by the Pipe Jacking Association, Concrete Pipe Association, Science and Engineering Research Council and Oxford University.

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KIRKLAND, C. J., (1982), "The need for Research". Paper presented at Pipejacking Conference, Manchester. Pipe Jacking Association.

WILLIAMS, A., (1979), "The bearing capacity of concrete over a limited area". Cement and Concrete Association, Technical Report 526.



9-12 LVDT's measuring thickness
17-25 SGR Strain Gauge Rosettes

FIGURE 1 Test Equipment Arrangement and Transducer Positions.

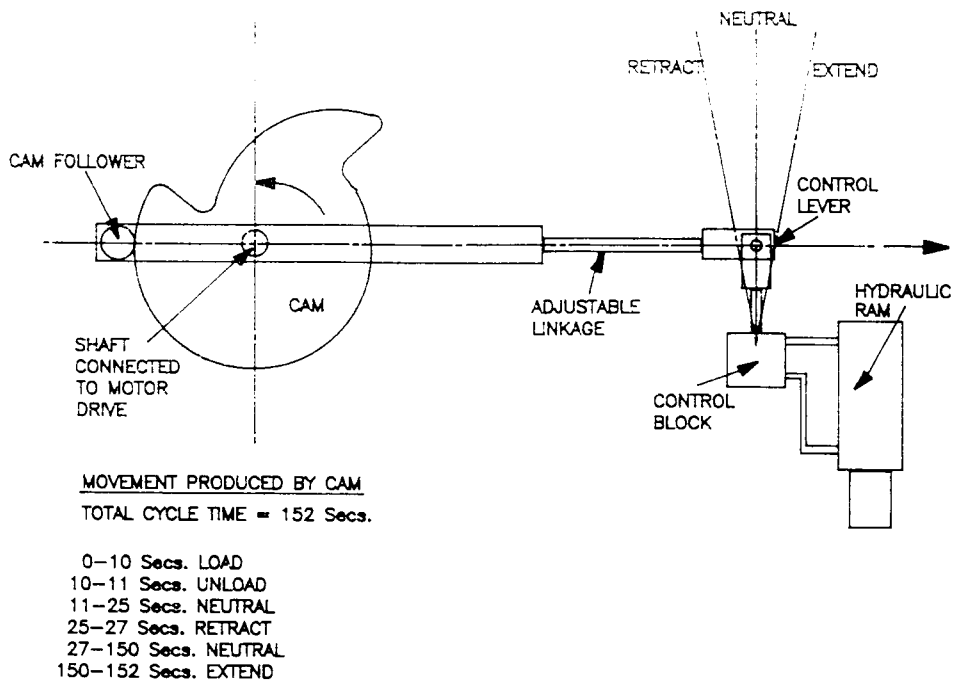


FIGURE 2 Cyclic Loading Mechanism.

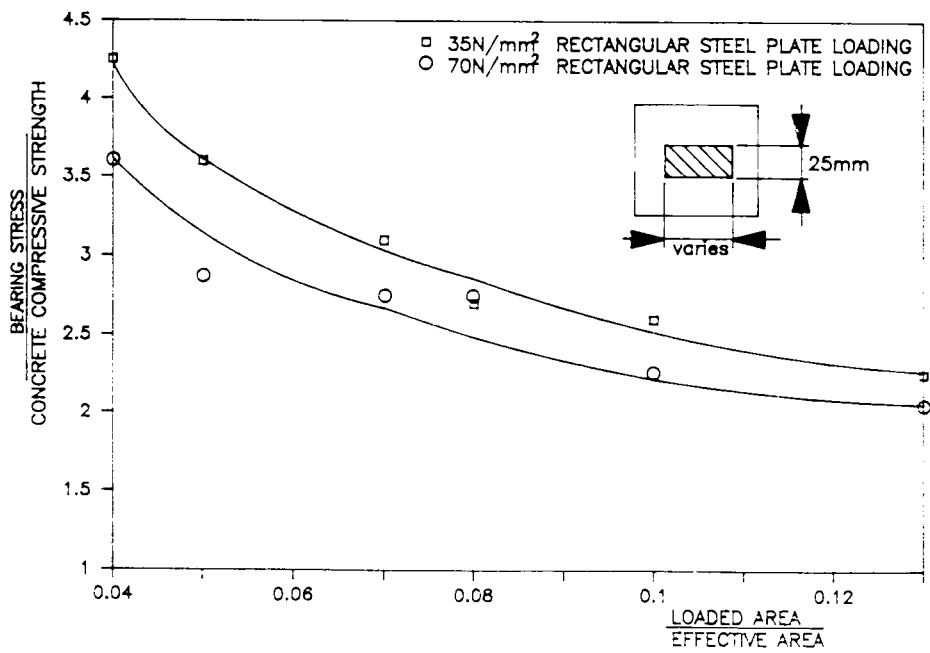


FIGURE 3 Affect of Size of Loaded Area on Concrete Bearing Stress (Williams 1979).

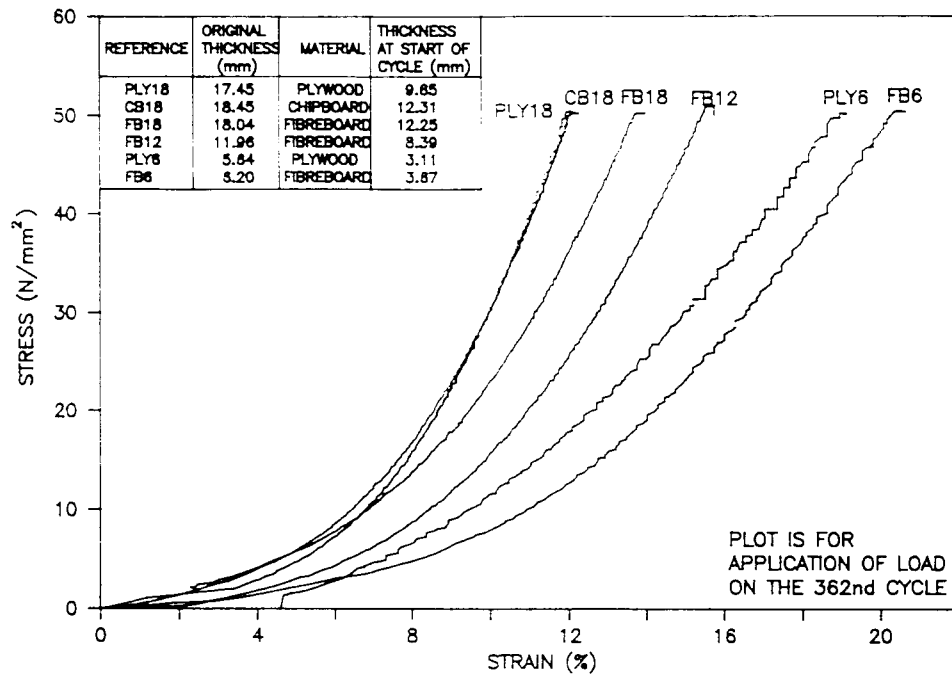


FIGURE 4 Graph of Stress V's Strain for Packing Materials.

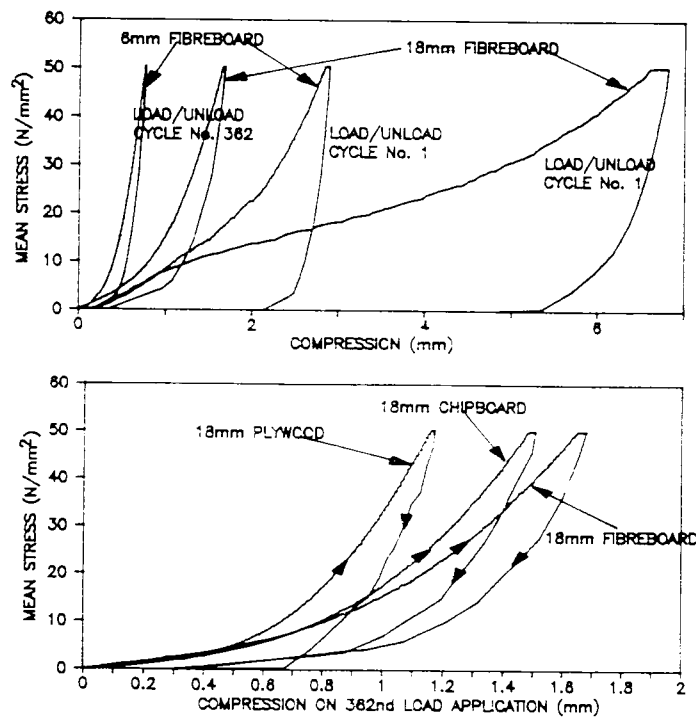


FIGURE 5 Graph of Stress V's Compression for Packing Materials.

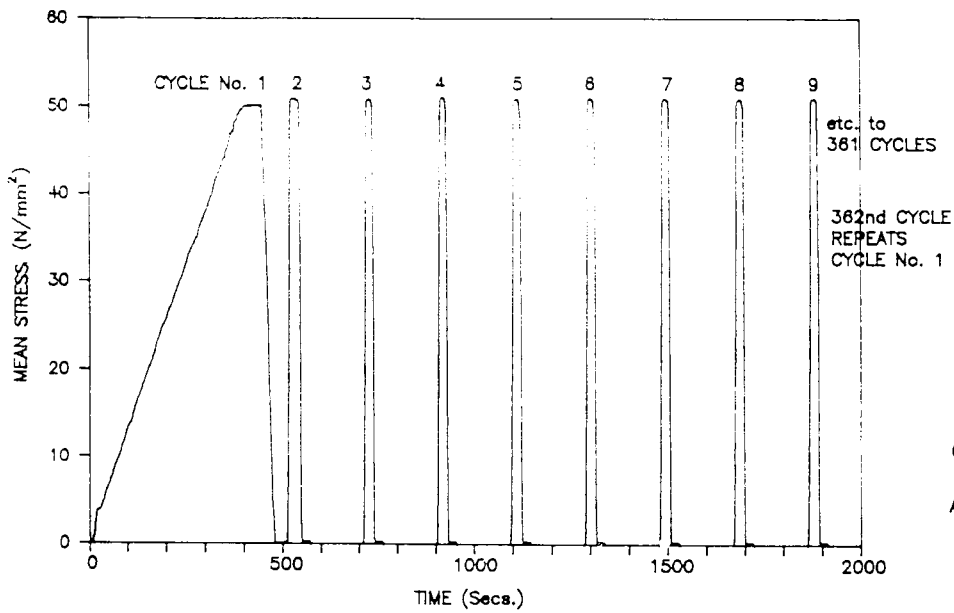


FIGURE 6
Cyclic Loading
Applications.

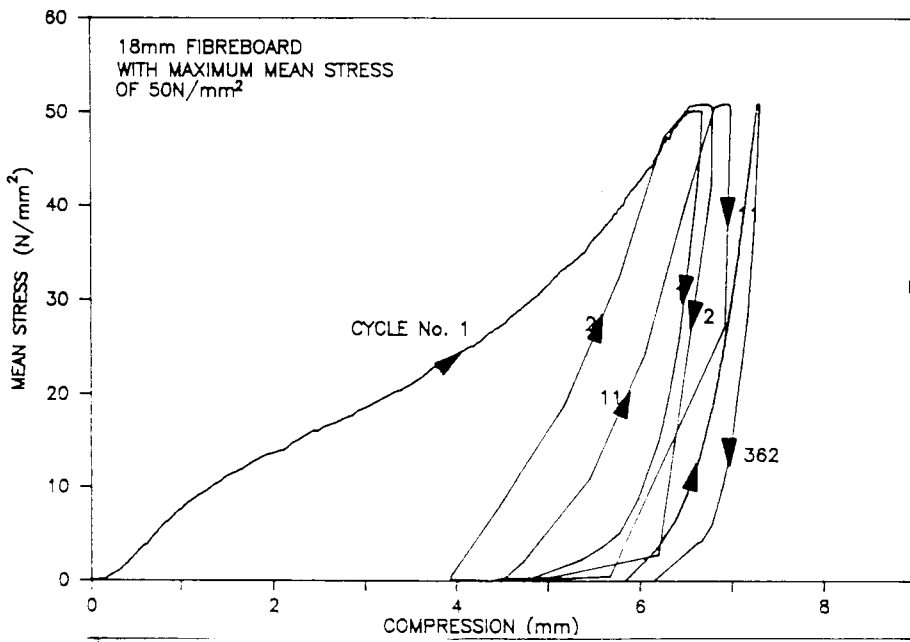


FIGURE 7
Effect of Cyclic
Loading on Compression
of Packing Materials.

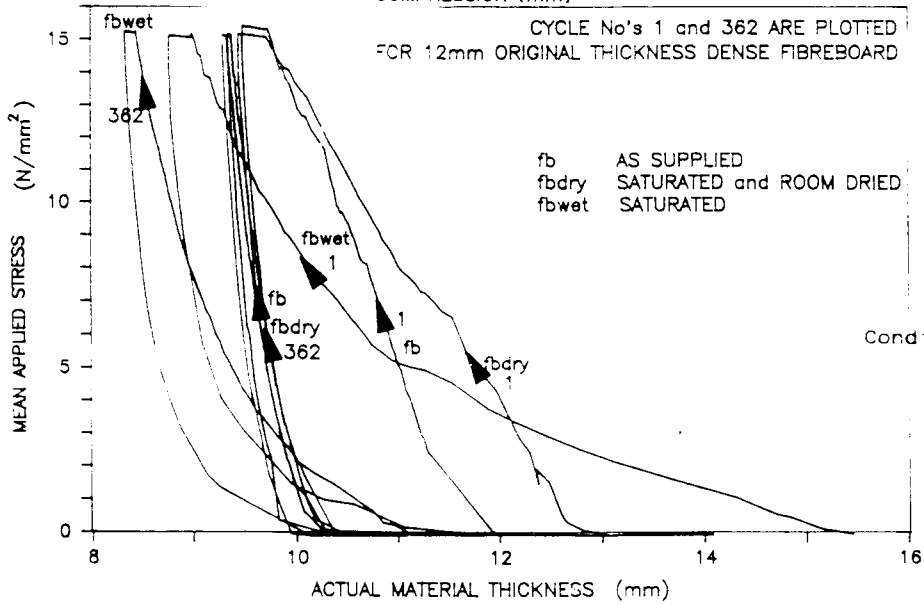


FIGURE 8
Effect of Material
Condition on Compression.

JACKING LOAD CAPACITY OF PACKING MATERIALS
ON MISALIGNED 900mm INTERNAL DIAMETER PIPES
WITH MAXIMUM ALLOWABLE STRESS OF 15 N/mm²

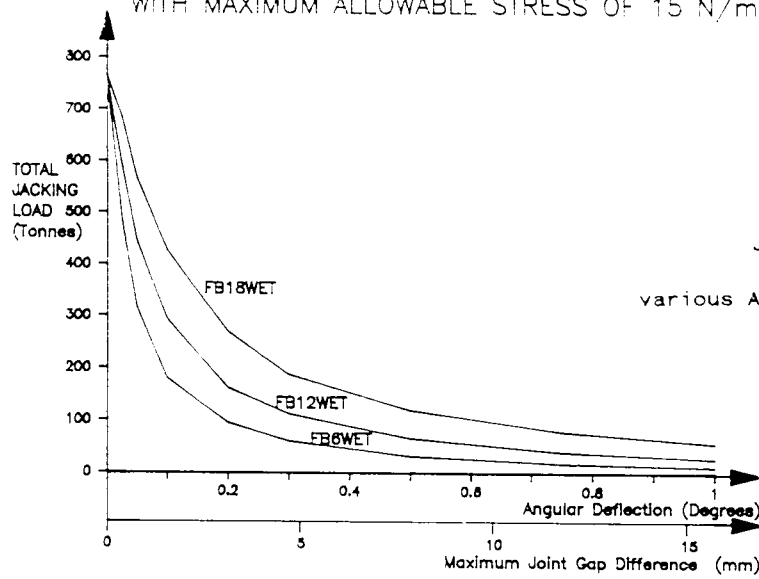
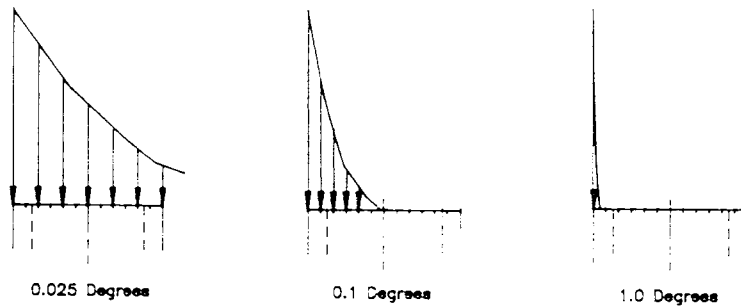


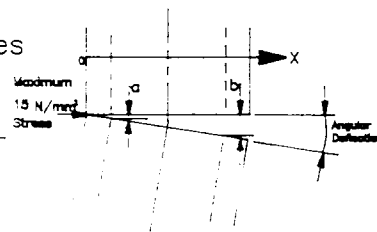
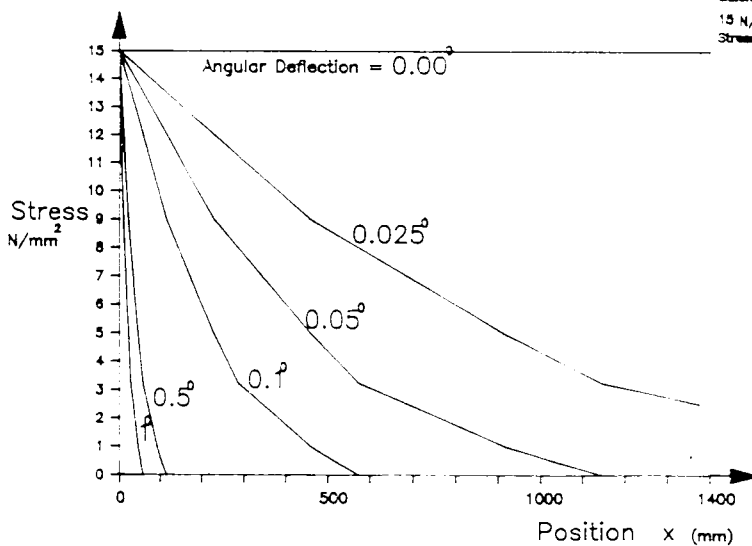
FIGURE 9

Jacking Capacity at various Angular Deflections.



Distribution of Load at stated Angular Deflections

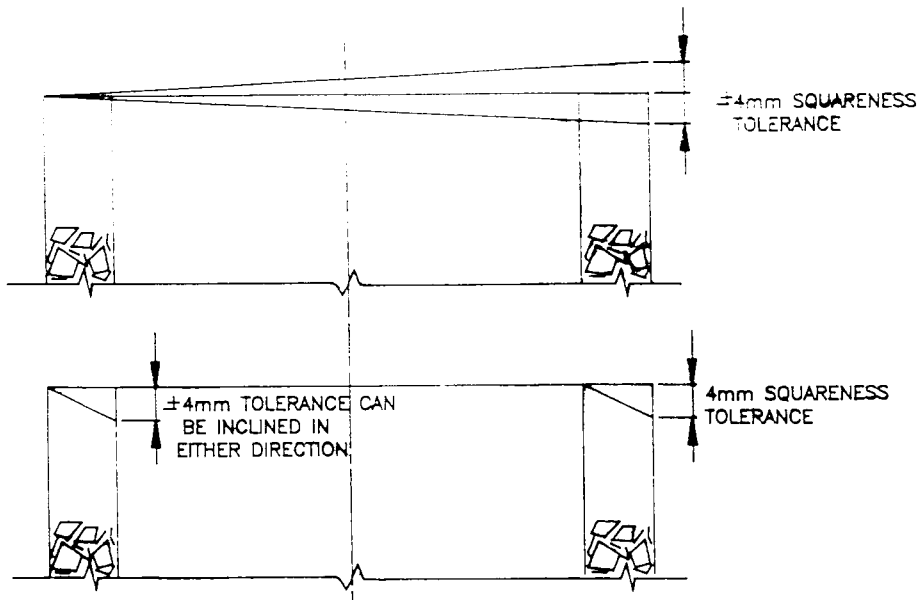
Typical Stress Distribution on Pipe Ends
for various Angular Deflections between Pipes



ANGULAR DEFLECTION (Degree)	PIPE INTERNAL DIAMETER	
	900 mm	1800 mm
0.0	0.0	0.0
0.025	0.4	0.8
0.05	0.8	1.6
0.1	1.6	3.1
0.2	3.1	6.3
0.3	4.7	9.4
0.5	7.8	15.7
0.75	11.8	23.6
1.0	15.7	31.4

FIGURE 10

Stress Distribution at various Angular Deflections.



PICTORAL REPRESENTATION OF BRITISH STANDARD SQUARENESS TOLERANCES AND HOW IT IS POSSIBLE FOR THEM TO MAGNIFY DEFLECTION ANGLES BETWEEN CONSECUTIVE PIPES

FIGURE 11 British Standard End Squareness.

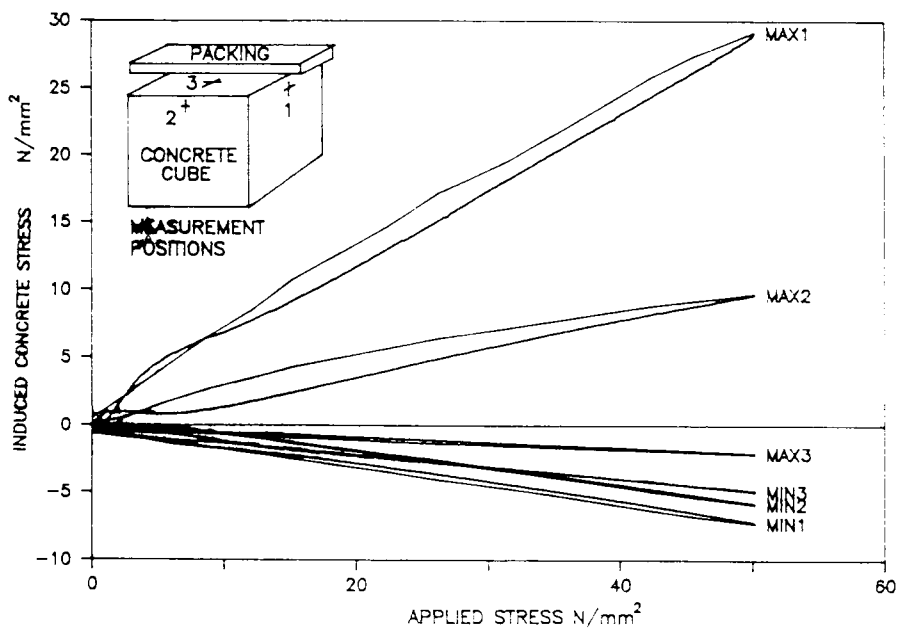


FIGURE 12 Induced Stresses in Concrete.

Field instrumentation for monitoring the performance of jacked concrete pipes

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1 INTRODUCTION

Pipe jacking is a technique for forming small diameter tunnels by pushing or jacking concrete 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 method has many advantages over other tunnelling techniques and is often used for sections of pipe under embankments, roads and railways where open cut methods are particularly uneconomic because of the need to keep traffic moving continuously. Although the technique is gaining acceptance with some specifying authorities its wider use is being held back by a lack of understanding of many factors affecting the installation and long term performance of such tunnels. This has prompted the Pipe Jacking Association (PJA) and five UK Water Service Companies to promote and support a research programme at Oxford University which aims to provide an improved understanding of the interaction between jacked pipes and the ground and the resulting pipe stresses by instrumenting and monitoring five full scale pipe jacks under normal site conditions. Such work inevitably takes place in an environment which cannot be fully controlled, but which can, with careful planning, be investigated, monitored and recorded. This paper concentrates on describing the specialist instrumentation developed for the research and provides an insight into the planning that was necessary for successful implementation.

2 OXFORD PIPE JACKING INSTRUMENTATION PROGRAMME

The Oxford pipe jacking research instrumentation consists of three instrument clusters or monitoring stations as shown in Figure 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 in 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. The majority of instruments are incorporated into the second special pipe which contains:

(a) Four Contact Stress Cells to measure the radial total stress and shear stress acting on the pipe surface.

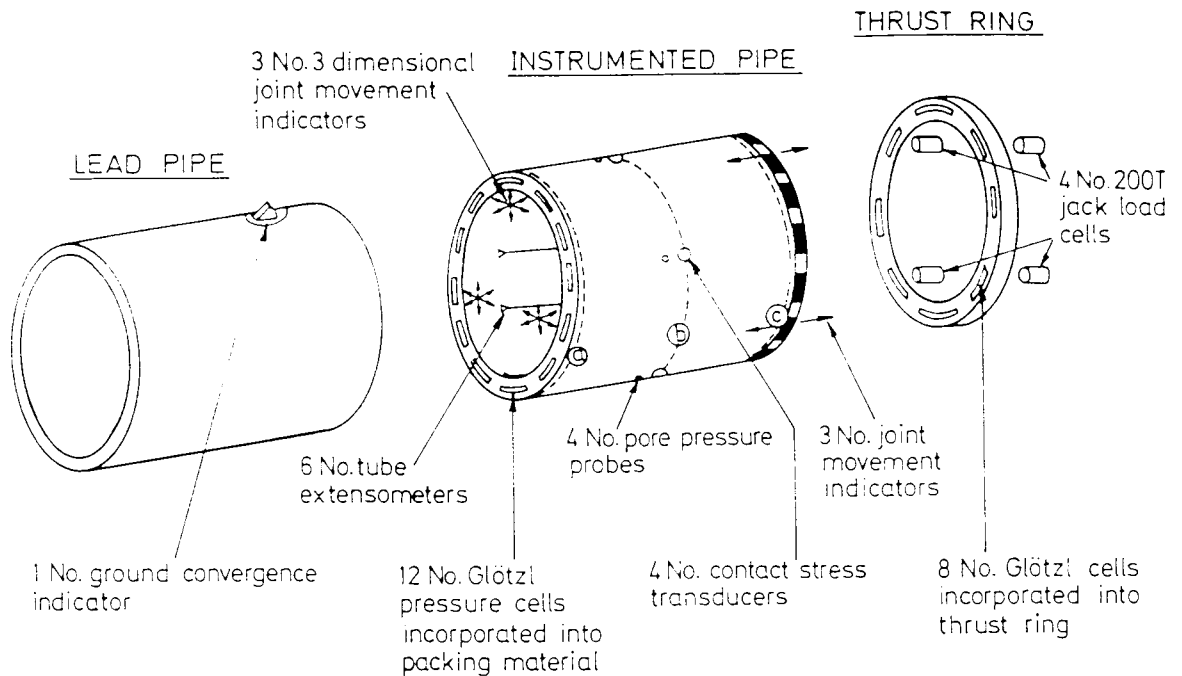


Figure 1. Schematic of Instrumentation Scheme

(b) Twelve Gloetzl pressure cells incorporated into the packing material at each end of the pipe to measure the magnitude and distribution of stresses at the pipe joints.

(c) Four Pore Pressure Probes to measure the changes in pore water pressure adjacent to the pipe.

(d) Six Pipe Joint Movement Indicators to measure the three dimensional joint gap movements and provide an alternative, approximate, indication of the stresses transmitted between pipes.

(e) Six Tube Extensometers equi-spaced around the pipe circumference to measure the overall longitudinal pipe compression.

All the above measurements are related to the total jacking force measured throughout the drive by compression load cells positioned between the jack rams and the thrust ring and the rate of pipe advance recorded using a Celesco Position (Displacement) transducer mounted above the tunnel entrance. An arrangement of Gloetzl cells incorporated into the thrust ring provides information on its effectiveness in distributing the end loads evenly throughout the jacking cycle. In addition a detailed log is kept of all the factors that affect the progress and performance of the drives. Such factors include tunnel face logging and insitu and laboratory soil testing as the jack proceeds to amplify the site investigation, regular line, level and chainage surveys, pipe dimensional checks and concrete strength and stiffness testing. Where possible, surface settlement above the pipe jack is also recorded.

3 INSTRUMENT DESIGNS

The main objective of the measurement programme is to monitor pipe performance during the installation phase which is usually three to four weeks depending upon the length of drive, ground conditions and method of excavation. Each instrument has been designed to operate in the aggressive tunnel environment, have minimal effect on the measured property, provide the required accuracy and ease of calibration and avoid significant disruption to normal site activities.

3.1 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 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 Figure 2. Successful performance of the instrument is highly dependent upon correct selection of the return spring stiffness which must be compatible with the ground conditions existing at each site.

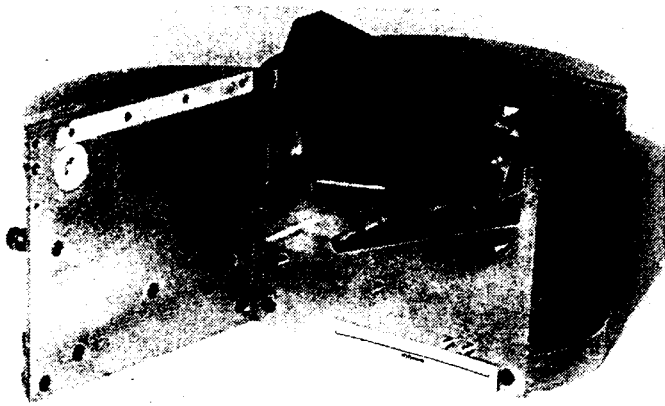


Figure 2. Ground Convergence Indicator

3.2 Contact Stress Cells

The contact stress cell measures both the radial total stress and the shear stress acting at specific locations on the pipe surface. The assembly is shown in an exploded view in Figure 3. At the heart of the instrument is a Cambridge Earth Pressure Cell, Stroud (1971), which is machined on a CNC milling machine from a single block of aluminium alloy 2014A and 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 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. The assembled contact stress cells are calibrated in a special calibration rig which applies shearing and compression forces simultaneously to the loading platen. A typical set of responses for an instrument under radial stress and shear stress calibrations is presented in Figure 4.

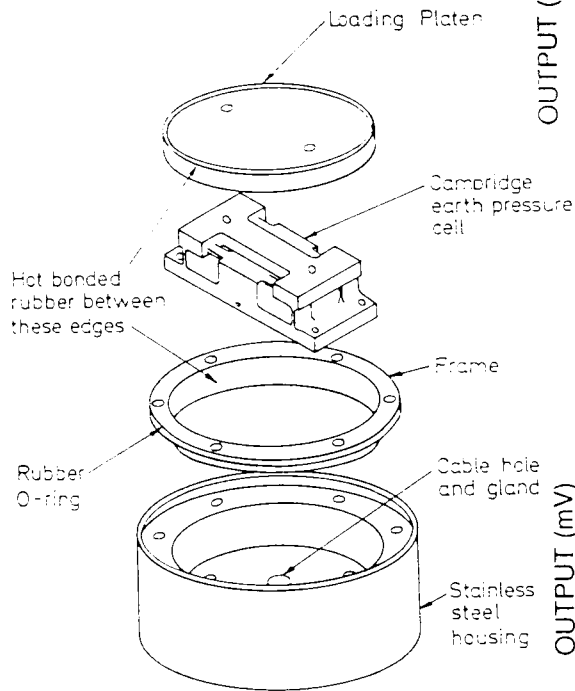


Figure 3. Exploded view of Contact Stress Cell

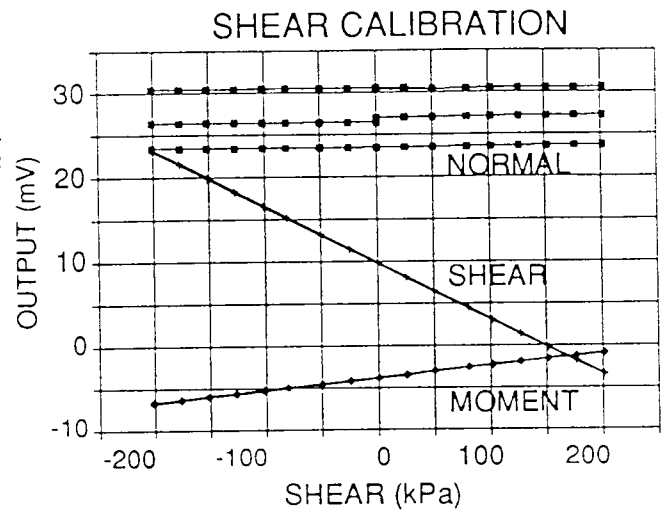
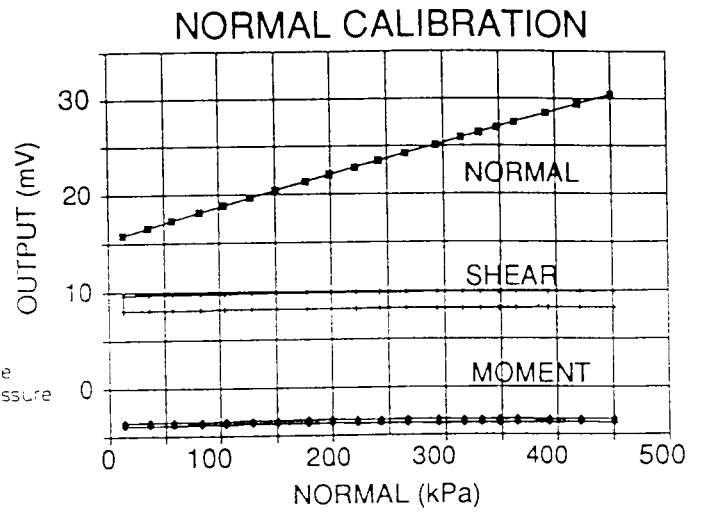


Figure 4. Contact Stress Cell calibration

The main requirements of the measurement performance can be seen to be satisfied:

- (a) Load capacities : Radial $\geq 500\text{kPa}$: Shear $\geq 200\text{kPa}$
- (b) Linear relationship between signal and load
- (c) Minimal cross-sensitivity
- (d) Fast response to changes in load

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. Shear box tests on a ground polymer modified mortar finish indicated that the material satisfied both frictional similitude and ease of incorporation into the instrument by gluing a 5mm thick mortar disk into a recess in the loading platen prior to profiling. Direct adhesion of the cells into specially cast high tolerance holes using a fast curing structural glue was used to minimise possible cell under-registration.

3.3 Gloetzl Pressure Cells

The most important consideration when selecting a suitable measurement device is that it has minimal influence on the desired measurement. This is particularly difficult to achieve in the confined and stiffness sensitive pipe joint region where pipe end stresses are being monitored by Gloetzl Pressure cells incorporated into the packer material.

The Gloetzl Pressure cell is a commercially available instrument which comprises a rectangular flat jack formed from two sheets of stainless steel, welded around the edges.

For the pipe jacking research, the space between the plates is filled with oil and a closed system is achieved by connecting the cell to a strain gauged diaphragm pressure transducer. Calibration of the hybrid 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 because the low stiffness of the specified Medium Density Fibreboard (MDF) packing material resulted in a stiffer cell-packer response (Figure 5a & 5b). A set of calibration tests using MDF packers with different cell-packer areas, applied stress levels and packer moisture contents have been carried out to establish the cell over-registration factor. Temperature changes in the pipejack joint are monitored using PRT temperature gauges mounted in lugs welded to the sides of a limited number of the rectangular plates.

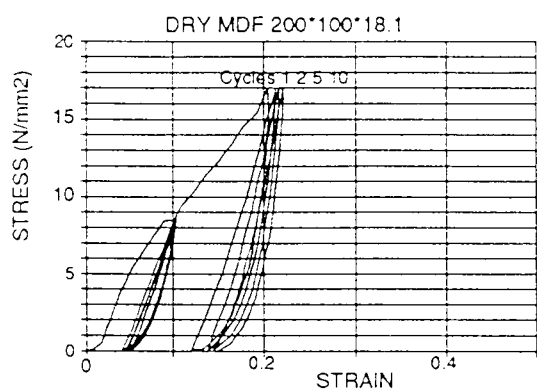


Figure 5a. Stiffness response of dry packer

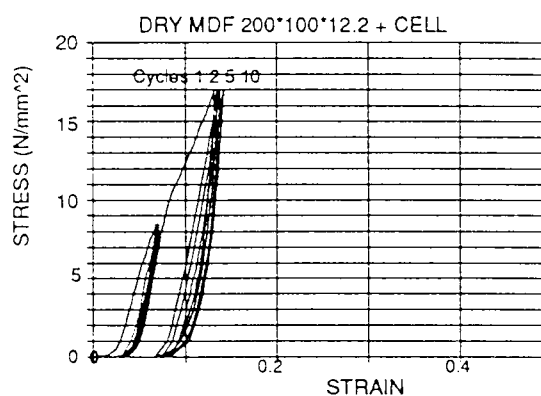


Figure 5b. Stiffness response of dry packer & cell composite

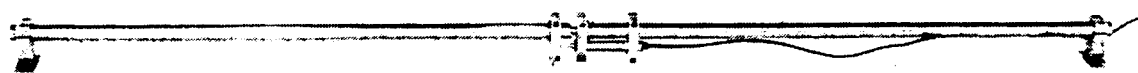


Figure 6. Tube Extensometer

3.4 Tube Extensometers

The tube extensometer enables concrete movements over a 1.6m gauge length to be recorded using an LVDT mounted at the centre of the device. The instrument as illustrated in Figure 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.

3.5 Pore Pressure Probes

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. Details of the probe are given in an exploded view in Figure 7.

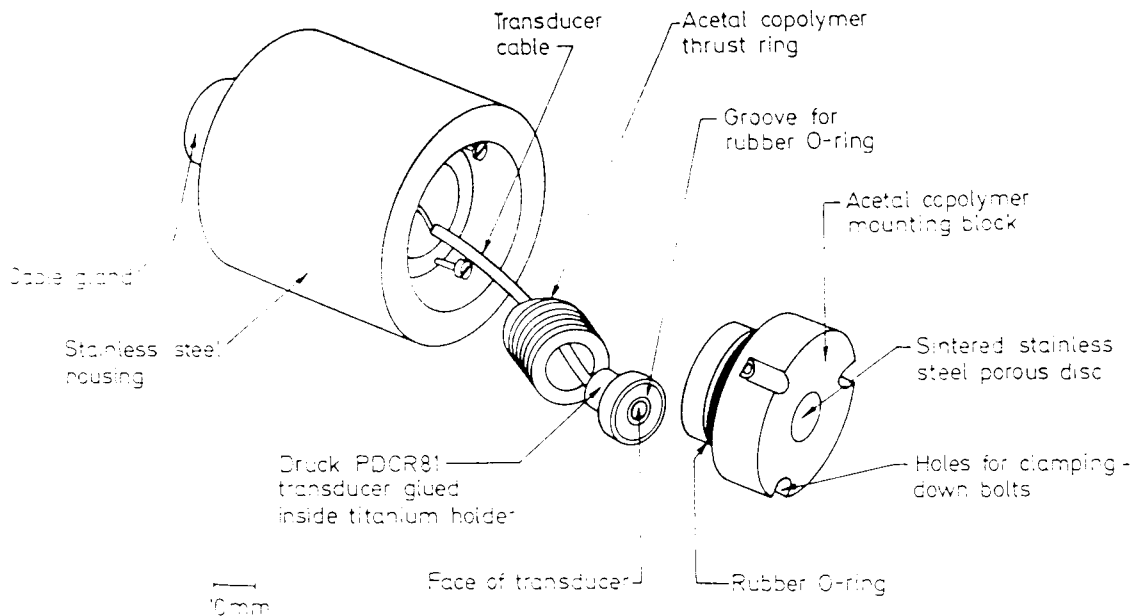


Figure 7. Pore pressure probe

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 cylindrical container which can be attached to a glycerine - air interface. Transfer of the saturated probe to the steel housing which is glued into a cast hole through the pipe wall is left until the instrumented pipe is ready for jacking to minimise possible problems with desaturation.

3.6 Pipe Joint Movement Indicators

The pipe joint movement indicator provides detailed information on the joint gap compression, angular deviation and shear movement as jacking proceeds. The instrument 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.

3.7 Jack Ram Load Cells

The jack ram load cells are a 200 tonne heavy duty compression type commercially available from Strainstall Ltd. The standard cable connection and cable type has been modified to ensure operation under submersed conditions. 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 and its method of attachment to the main jack rams are illustrated in Figure 8. A protective hood prevents accidental damage to the load cell.

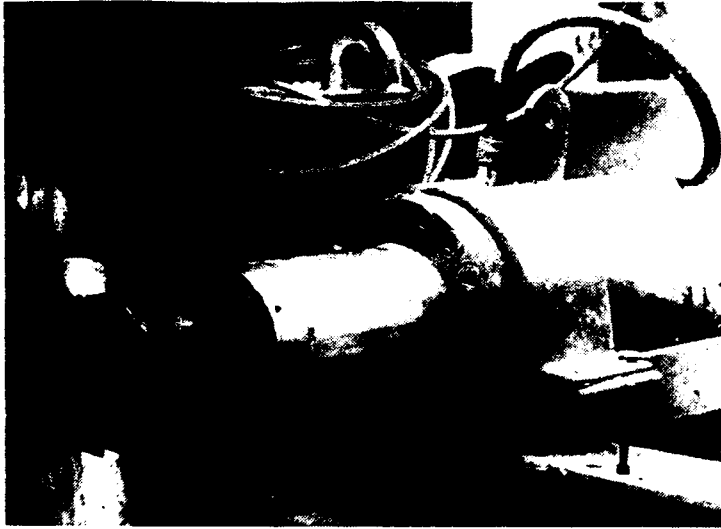


Figure 8. Jack ram load cell

3.8 Data Acquisition System

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 Personal Computer to be used as the basis of control. The system enables a family of standard analogue input modules capable of accepting information from the various instruments to be located close to the measurement station. Short lengths of analogue signal cables and communication in digital form between the control module and the host computer, situated above ground, minimises the risk of signal corruption. The system is capable of 16 bit measurement performance and is readily expanded to 120 channels which is the maximum anticipated for the fieldwork. The 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 signal and power cables which enter the pipe jack are required to be disconnected each time a pipe is introduced into the drive.

The control module and analogue input modules as supplied by Measurement Systems Ltd were not suitable for rough field conditions and could not power up the instruments. It was therefore necessary to house each module in a sealed steel box which was internally arranged to accommodate a single module and a purpose built stable 'dc' power supply capable of supplying either 5V or 10V to the 16 instrument channels supported by each box. Lemo underwater cable connectors were used to interface the instruments to the boxes. Instrument logging strategies were time based with all readings time stamped for cross correlation with site construction activities.

4 PLANNING AND EXECUTION OF THE FIELDWORK

The success of any site based research is not only dependent upon the selection of suitable instruments but also thorough planning and execution of the work during the design and construction phases 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, understanding the roles and motivations of the parties involved and of maximising the lead in time to the fieldwork. Clear communication was important at all stages with particular care being taken to ensure that proper provisions for the research were provided in the contract documents. During the site

work a member of the contractors staff was made personally responsible for the correct interpretation of the instrumentation installation and protection procedures which were designed to minimise delay to the contractor's programme. Recovery of the instruments was left until the pipe jack operations were complete.

4.1 Instrument Installation

Two methods of pipe manufacture are used in the UK, vertically cast and centrifugally spun both of which use a dry mix and substantial compaction forces. The high risk of instrument damage during the casting process and subsequent delivery to site ruled out installation at the pipe works. It was therefore necessary to cast the required holes and tufnel anchorage blocks for the in-wall and surface mounted instruments and deliver the pipes to site with sufficient time to complete the installation above ground. Installation of the instruments and data acquisition system took approximately 3 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.

4.2 Instrument Protection

Protection of the instruments and data acquisition system against mechanical damage is provided by a steel liner which fits inside the main instrumented pipe and is detailed to avoid significant stiffening of its response (figure 9). The liner has a 300mm smaller diameter than the pipeline and is fabricated in two 1.4m lengths which is slightly longer than the 2.5m instrumented pipe length. Each overhanging portion is bolted to a steel cradle fastened to the leading and trailing pipes. The liner is supported in the instrumented pipe by a timber cradle which is shaped to cover a 150 degree arc about the pipe invert and allow the instruments to be correctly positioned. Cross holes in the timber cradle provide ducts for routing cables around the pipe circumference. Articulation of the liner is provided by a central steel banded joint which allows the two halves to slide $\pm 10\text{mm}$ relative to each other. A set of ramps in the leading and trailing pipes allow muck skips to travel through the liner. Although the smaller diameter of the liner produces a constriction in the pipeline it has been found in practice that the protection it provides is well worth the slight reduction in productivity that it causes.

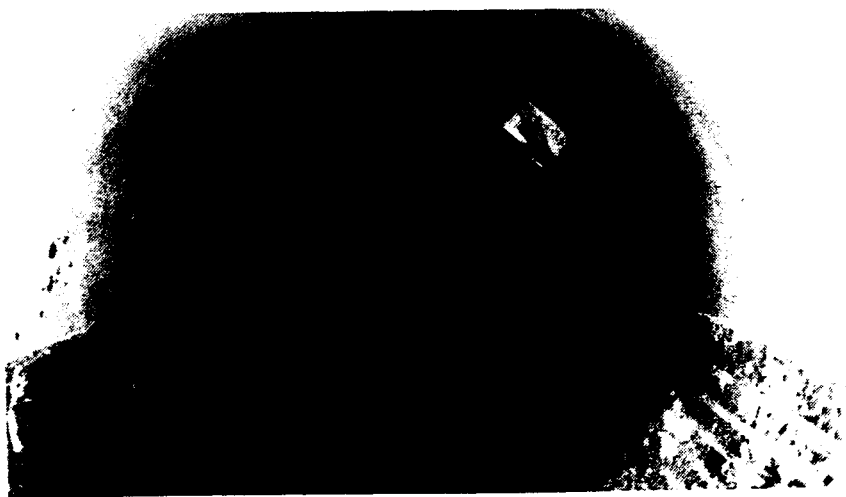


Figure 9. Protective liner

4.3 Instrument Recovery

Recovery of the instruments was carried out on completion of the drive avoiding restricted access to the pipes. A variety of techniques were used for the different instruments. Recovery of the Gloetzi cells located in the joints required special anchorage brackets to be bolted to each side of the instrumented joint and cylinder jacks inserted to jack the joint apart and allow removal of the cells and insertion of replacement packing material. The pipeline was then re-jacked by the main jack rams to close the joints. The in-wall instruments were recovered by overcoring and making good the openings left in the pipe using precast concrete plugs and epoxy mortar. The surface mounted instruments, data acquisition units and thrust pit instruments were simply unbolted from their anchorage points.

5 PERFORMANCE OF THE INSTRUMENTS AND SITE PROCEDURES

A pilot test was successfully completed during August 1990 on a 65m long drive, through stiff boulder clay, in Bolton. The Client, Bolton Metropolitan Borough Council, acting as agents for North West Water Ltd., designed the Bury Road Re-sewerage Scheme to alleviate foul flooding which was affecting premises in the Brightmet area of Bolton. Part of the scheme, the 1200mm internal diameter Milnthorpe Road Retention Tank, was brought forward as a separate phase so that the research could be carried out 'off-line'. This allowed the main instrumented pipe to be installed close behind the shield, jacked out the far end, inspected for damage and then re-inserted at the beginning of the same drive. The contract ran smoothly throughout, with no contractual problems, which was a direct result of the positive approach adopted by all involved. The

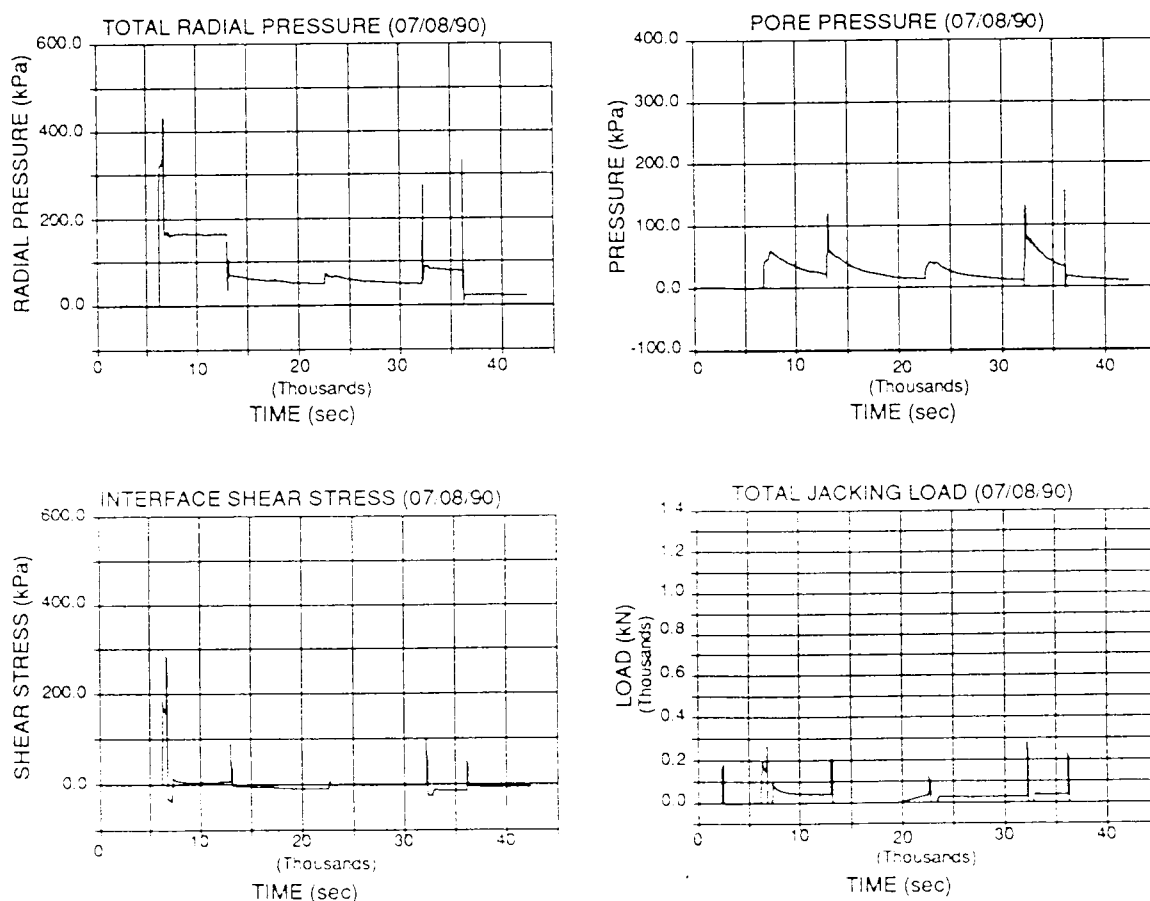


Figure 10. Contact stress cell & pore pressure readings

additional work and disruption to normal site activities was estimated to cost £11000, all of which was covered by bill items. All the instruments were recovered for recalibration and re-use after the test. Recalibration of the various instruments agreed to within 2% of the original values.

The performance of the instrumentation system was very satisfactory. There was no mechanical damage to any equipment (including power and signal cables) during the contract period. The casualty rate of the instruments was almost zero with only one contact stress cell failing in service as a result of moisture ingress past a malleable copper washer seal and a second cell illustrating long term drift due to moisture ingress past a faulty 'O' ring. While detailed results and their interpretation are not the subject of the current paper it may be of interest to present limited field data. Figure 10 illustrates the responses of the bottom pore pressure probe and contact stress cell which were obtained over a single working shift. The graphs clearly illustrate the rapid build-up in pore water pressure and total radial and shear stress during each jacking operation. Subsequent release of the jacking force produces a small amount of locked-in interface shear and a rapid reduction in total radial stress and pore water pressure followed by a slow continual dissipation until the next push.

6 CONCLUSIONS

The success of the pilot test has shown that the instrumentation and test procedures are correct and that the remaining tests can proceed with confidence. The positive approach adopted by all involved was absolutely essential to the success of the pilot test given the inevitable interference to work associated with carrying out research in the confined space of a 1200mm I.D. tunnel. Disruption was kept to a minimum and progress was only slightly delayed, principally because the instrumentation was almost 100% successful and the planning and execution of the work eliminated many areas of possible delay and conflict.

ACKNOWLEDGEMENTS

The instruments described in this paper were developed as part of the Oxford Pipe Jacking Research Project which is sponsored jointly by SERC, the PJA and five UK Water Service Companies, Northumbrian, North West, Severn Trent, Thames and Yorkshire. The contact stress cells were designed jointly with Mr Clive Dalton of Cambridge Insitu Ltd; thanks also to Dr Andrew Bond (formerly of Imperial College) for his advice on the design of the pore pressure probes and Mr Ron Morton for his help in manufacturing the hardware for the data acquisition system. The authors also acknowledge the assistance and co-operation given by Bolton Metropolitan Borough Council as agents for North West Water, Laserbore Ltd of Sheffield and Buchan Concrete Ltd which was essential to the success of the pilot test.

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**PIPE JACKING AND
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The Oxford research project seeks to improve understanding of the loading induced on pipes, and the interaction between pipes and ground, during their installation by pipe jacking. The first stage involved laboratory studies of model pipes. The current second stage, described in this paper, involves field measurements on specially instrumented pipes during actual site operations. The measurements are related to detailed line and level surveys of the pipe line and to local ground conditions. The aim is to allow better prediction and control of pipe jacking operations in the future.

The work is supported by the Science and Engineering Research Council, the Pipe Jacking Association, 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 paper sets out the history and objectives of the project, the method of management, the overall concept of the instrumentation, the selection of sites, the programme of work, problems encountered and progress to date. Much valuable information from real site conditions has already been obtained and limited preliminary analysis carried out. Space restrictions prevent a meaningful presentation of the data in this paper.

HISTORY

This research project arose from the need of the pipe jacking industry, in the form of the Pipe Jacking Association (PJA) and Concrete Pipe Association (CPA), to understand better how pipes and ground interact during pipe jacking operations. Following a survey report by Craig for CIRIA, the Construction Industry Research and Information Association (1), discussions were held between the PJA research committee and Oxford in the latter part of 1984. An application in 1985 to the Science and Engineering Research Council (SERC) was successful and provided funding in conjunction with the PJA and CPA for initial model testing in the laboratory.

This first phase was carried out by Kevin Ripley, starting in February 1986 and finishing three years later (2). 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 butt joints (steel-banded or similar) to in-wall joints for the transmission of large jacking forces (3).

In the summer of 1987 discussions on phase two were initiated, leading to a further grant from SERC in 1988 and final agreement on additional funding from the PJA and the five Water Authorities in July 1989, some four months after work had started on the project. This phase involves instrumentation of actual pipe jacks in the field and is being carried out by Paul Norris, with some assistance from a research student, Chris Eggleton, who is undertaking a subsidiary project to study the effects of lubrication. The project period is again three years, although it may be extended for a few months, with completion in the summer of 1992.

FUNDING AND MANAGEMENT

The contract cost for phase two is about £220,000, as shown in Table 1; SERC carries about 20% of the total, the PJA about 40%, and the remainder is divided equally between the five Water Companies. In addition, each site operation involves 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 will have carried these additional costs for one site.

The programme of research is overseen by a management group with two representatives from the PJA, one from Oxford, and one from each of the water companies. This group meets four times a year and receives progress reports from the research team. The funds are administered via a special account with Thames Water, apart from the SERC contribution which is used to support the research assistant directly through the University.

OBJECTIVES

The overall objectives of the work arise from the CIRIA report (1), which listed the following main areas requiring investigation:

- (a) friction loads in different ground conditions;
- (b) load/deflection characteristics of the joints with different packing materials;
- (c) the effect of cyclic loading on the pipes at intermediate jacking stations;
- (d) the effect of lubricants in reducing friction along a pipe;
- (e) the development of a site investigation test suitable for the prediction of frictional forces.

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 project is therefore to investigate the load transfer between pipes and the contact pressures between pipes and soil.

More detailed research objectives include the determination of the areas of contact between pipe and ground in both granular

and cohesive soils, the normal and shear stresses developed at these contacts, the effects of lubrication, the stresses and strains in individual pipes, the effects of angular deviations at joints, and the performance of the packing material. Some of the points were studied in the laboratory investigation, but needed confirmation at full scale, while other effects could only be achieved in the field or depended on realistic ground conditions and construction procedures.

INSTRUMENTATION AND DATA COLLECTION

Instrumentation

The instrumentation developed to meet these objectives is shown schematically in Figure 1. The full set of instruments is arranged in three groups, in the lead pipe, in the main instrumented pipe and at the jacking pit. For drives through cohesive soil, the lead pipe contains a ground convergence indicator which measures the rate at which the ground closes onto the pipe, particularly when jacking is halted overnight or at weekends. It consists essentially of a hinged arm spring-loaded against the ground and connected to a rotary potentiometer.

The main instrumented pipe is located further back in the pipe string and contains the following instruments:-

- (i) four contact stress cells, to measure both radial and shear total stresses on the surface of the pipe, with their active face flush with the pipe surface and provided with a similar surface finish to the pipe;
- (ii) four pore pressure cells adjacent to the contact stress cells, measuring the local pore pressure and hence allowing determination of the effective radial stress;
- (iii) three joint movement indicators at each end of the pipe, to measure the movements across the joint gaps in three dimensions;
- (iv) twelve Glotzl pressure cells built into the packer in the joint at either end of the pipe, to measure the magnitude and distribution of the loads transferred across the joints;
- (v) six extensometers fitted to the internal surface of the pipe and equally spaced around it to measure the compression of the pipe under load.

In the jacking pit the total jacking load is monitored continuously using two or four load cells positioned between the jack rams and the thrust ring, and the overall movement of the pipe string by a displacement transducer mounted above the tunnel entrance. It was also intended to incorporate Glotzl cells into the thrust ring to check its effectiveness in distributing the jack loads, but to date this has not been possible in practice.

A flow chart summarising the objectives of the instrumentation is shown in Figure 2.

Details of the design, construction and calibration of the instruments are discussed by Norris and Milligan (4). All instruments were designed to operate successfully in the aggressive tunnel environment, have minimal effect on the property to be measured, be sufficiently accurate and reasonably simple to calibrate, and disrupt normal site operations as little as possible.

Data collection and supplementary information

A modular data acquisition system for up to 120 channels has been developed, with control and input modules mounted inside the main instrumented pipe. The instruments are connected to these by short lengths of cable, and the analogue signals from them are converted to digital signals for transmission by a single cable to the personal computer located in a container adjacent to the jacking pit, reducing the risk of signal corruption. This signal

and the power cable to the instruments have to be broken and reconnected each time a new pipe is introduced, in the same way as the contractor's power cables.

All results are recorded on a time basis, with logging intervals controlled by the computer; at intervals the data is backed up on floppy disks to ensure that no data is lost. A site log records all relevant activities on site for later correlation with instrument readings. A daily survey of line and level of the full length of the tunnel is made, and face logs and additional soil sampling and testing allow correlation of readings with local ground conditions. Where possible, surface settlement above the tunnel is also measured.

To protect the instruments and data acquisition and control boxes from passing traffic, a steel liner is fitted inside the main instrumented pipe; this is designed and fixed so that it carries none of the jacking load itself. Inevitably this liner constricts the tunnel bore locally and causes some reduction in the contractor's output. Further details of all of the above are provided by Norris and Milligan (4).

SITE SELECTION AND PROCEDURES

Site Selection

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 perhaps naively expected 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 important parameters affecting the performance of pipe jacks; decisions were then taken as to which would be kept constant and which varied between the schemes. The result appears in Table 2; 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. Other parameters were to be kept as consistent as possible except for the final drive when a longer and deeper drive would be attempted, and some instruments then left in place to monitor long-term behaviour.

For the earlier four schemes, typical lengths and depths were chosen, 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. The packing material was specified as medium density fibreboard, on the basis of the laboratory work; use of the same material throughout was essential to allow proper calibration of the Glotzl cells in the pipe joints.

It was always intended that the first drive would be a pilot scheme, using 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.

Details of Schemes

In practice the process of selecting schemes was very different.

Details of the actual schemes instrumented are given in Table 3. 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 schemes were both taken on at short notice, with contractors already on site, and the research introduced as a variation to the contract. Scheme two was a late replacement for one that had been in an advanced stage of planning, but was then postponed; while scheme three followed scheme two much too closely for comfort, requiring an early exit from scheme two and negligible time for repair or recalibration of instruments between jobs.

In parallel with schemes two and three, arrangements were being made for scheme four on a job in Yorkshire. Contract documents incorporating the research work were in preparation, when this scheme was also postponed and would then be too late to be included in the three-year programme. Fortunately two other sites have become available, both in cohesionless material, and this will allow a full set of five schemes in a variety of ground conditions. In both cases, the research has had to be introduced as a variation to a contract that had already been let. Thus only in the case of the pilot drive 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 1200 and 1800mm, and cover depths from 1.5m to as much as 21m under an embankment in scheme three.

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 three. At the time of writing, scheme four was just starting; it was intended to start this without lubrication, but to use lubrication and interjacks as necessary depending on the rate of increase of the total jacking loads. Comparison between lubricated and unlubricated behaviour should then be possible. All four of these schemes have been hand excavated. The final scheme, in loose sand and gravel below the water table, will be driven using an Euro-Iseki Unclemole tunnel boring machine and lubricated throughout.

Site Procedures

Suitable site procedures are as important as good design of instruments and data acquisition systems if site monitoring is to be successful without hindering the work excessively. As far as possible, preparation of the instrumented pipes takes place off-line. The exact methods of fixing the instruments varies with the way in which the pipes are formed (vertically cast or centrifugally spun), but the necessary holes, inserts or brackets are built in at the pipe works. On site, the instruments are fitted while the pipe is on the surface, and only parts of the joint movement indicators (and sometimes some of the Glotzl cells) have to be glued in

place after the pipe is in the tunnel. The total time for assembly and system checking of the instrumentation is between two and three days.

Likewise a timber cradle for the protective liner for the instruments is fitted while the pipe is on the surface, though the liner itself and ramps at either end of it have to be fixed once the pipe is in the tunnel, causing a few hours delay. Once the monitoring is in progress, good communication between miners and researchers ensures that there is very little disruption to the normal working pattern. The instrumentation may even be of assistance; in particular, the jack load cells provide a precise and continuous record of total jacking loads which may help with decisions on whether or when to lubricate or use an interjack.

The full length of the tunnel is surveyed once a day, but this is arranged so that it is done substantially within normal site breaks. Similarly, sampling or additional geotechnical testing at the face is organised to be done during breaks. On scheme three, when 24-hour working became necessary, the research team also worked shifts to maintain full-time monitoring.

Removal of instruments takes place at the end of the drive or, if before then, at a weekend. The liner has to be removed, then extensometers, joint movement indicators and data collection boxes are unbolted. Instruments cast into the side walls of the pipe are recovered by overcoring, and the resulting holes made good as required. Removal of the Glotzl cells requires that the pipe joints are opened slightly, when they can be prised off the packer material and pulled out of the joint. The joints can be opened using two small jacks operating on brackets bolted to the pipe walls on either side of the instrumented joint, provided that the instrumented pipe is near one end of the tunnel or an actual or dummy interjack so that the friction on only a few pipes has 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 a tedious operation and caused some superficial damage to the cells.

DATA HANDLING

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

A 386AT personal computer is used for processing. A limitation of the 123 software used is that the ASCII files produced on site can only be 240 characters wide (16 channels) for importing into a spread sheet. Approximately 70 channels are used in the research and so the site data files have to be split into smaller manageable groups of instruments. This stage is performed using a Fortran program written at Oxford.

Although the data can be scaled and converted to engineering units at the time of capture, it is preferred to record only the voltage readings from the various instruments. The conversions are carried out in the spread sheet at Oxford, minimising the possibility of introducing errors at source on site.

Traces of each instrument response on a daily basis are produced. These form the starting point for detailed analysis of individual pushes and cross correlation between different instruments. The data handling process described above can typically take from four to six weeks to complete for each scheme. Back-up copies of each stage of processing are created and stored on floppy disk. Detailed analysis of the Bolton site is well advanced, principally because the reliability of the data had to be verified before commencing work on the remaining sites. All other detailed analysis will be performed once the site phase of the project is completed.

PROGRAMME, PROBLEMS and PROGRESS

Programme

The research contract started on 1st April 1989, the original outline programme envisaged that the pilot instrumented drive would take place early in 1990, with the four remaining drives in the year from the middle of 1990. After nine months, the more detailed programme shown in Table 4 was set out. Approximately the first year would see the completion of the design, manufacture and calibration of the instrumentation, the next year and a half would be required for the five pipe jacks, and the final six months for analysis of results and writing of a thesis.

Problems

The difficulties experienced in obtaining suitable sites have already been mentioned. Some other problems have been weather-related. The pilot scheme started in some of the hottest Lancashire weather for decades, and then suffered torrential rain which affected ground conditions to the depth of the pipe. Scheme two took place during the worst of the severe winter weather; an attempt to hold a management meeting on site produced a nil attendance apart from those already on site because travel was almost impossible. These extremes of temperature were potentially damaging to instruments exposed on the surface; also some are glued in place and correct application and curing of the glue was very difficult.

Unwelcome encounters on site were an unexpected water pipe which temporarily flooded scheme one and an uncharted mine shaft which delayed scheme two. Scheme three progressed surprisingly smoothly given that it was on the tightest schedule.

Generally speaking the instruments have to date proved extremely successful. No major modifications were found necessary after the pilot scheme, and failures have been surprisingly few. There was some trouble with ingress of fine particles past the seals of the ground convergence indicator, but constraints of programming or ground conditions have in any case prevented this from being used after the first scheme.

The most vulnerable instruments have, as expected, proved to be the contact stress cells. These contain at their heart a transducer with relatively slender strain-gauged webs; they are liable to damage by overloading and to strain gauge failure if any moisture gets in. Gauge readings have been lost on one or two occasions, and one complete cell was crushed in scheme three. On this same scheme some of the other stress and pore-pressure cells were forced back into the pipe walls. These events appear to be related to extremely high local stresses on the pipe, yielding of the glue line bonding the cells in place may well have saved them from more serious damage.

Progress

Minor contractual delays meant that the pilot drive was about three months late in starting, but this was immediately recouped

because modifications to instruments were not necessary. The lack of suitable sites then again put the programme four months behind, but some of this was regained by running scheme three hard on the heels of scheme two. Completion of the final two schemes this summer will leave the site programme about two months late, almost entirely controlled by the availability of sites. To allow adequate time for analysis and writing up of the work, the overall programme will be extended by two to three months. Financially, the project is running and should be completed within budget; to date only trivial sums have been required from the contingency sum.

CONCLUSIONS

A contract for instrumenting and monitoring real pipe jacks under actual field conditions to a degree never previously attempted is now almost complete. Despite problems in obtaining suitable sites on which to work, the project has been extremely successful and a wealth of invaluable data has been obtained. Analysis of the data should allow conclusions to be drawn which will improve understanding of the jacking process, the loading induced in jacked pipes and the relations between jacking forces and ground conditions.

ACKNOWLEDGEMENTS

This project could not have succeeded without the contributions of many organisations and individuals too numerous to mention by name. Thanks are due to all the sponsors of the project, particularly their representatives on the management committee; the Water Companies and their agent authorities for providing sites and other assistance with the work; the pipe manufacturers for providing specially prepared pipes at only nominal additional cost; the contractors on the various schemes for co-operating with the research work; and various staff, technicians and students in the Engineering Department at Oxford who have helped with the project.

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

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

Table 1. Costs and Funding of research work.

		SCHEME 1	SCHEME 2	SCHEME 3	SCHEME 4	SCHEME 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
	Position of Test Pipe	Pipe No.3	Pipe No.3	Mid	Pipe No.3	Mid
	Lubrication	No	No	No	No	No
	Packer Type	Medium Dense Fibreboard (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	

Table 2 Ideal site requirements for research.

ACTUAL SCHEMES

	1	2	3	4	5
Date	August 1990	January 1991	March 1991	July 1991	Sept 1991
Location	Bolton, Lancs	Gateshead, Tyneside	Honor Oak, SE London	Chorley, Lancs	Cheltenham
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 and Gravel
Cover (m)	1.5 - 1.7	7 - 11	11 - 21	5 - 7	4 - 7
Drive Length (m)	65	110 *	80	160	350
Position of Test Pipe	Pipe No.3	Pipe No.10	Pipe No.15	Pipe No.15	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.

Table 3. Details of actual schemes monitored.

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

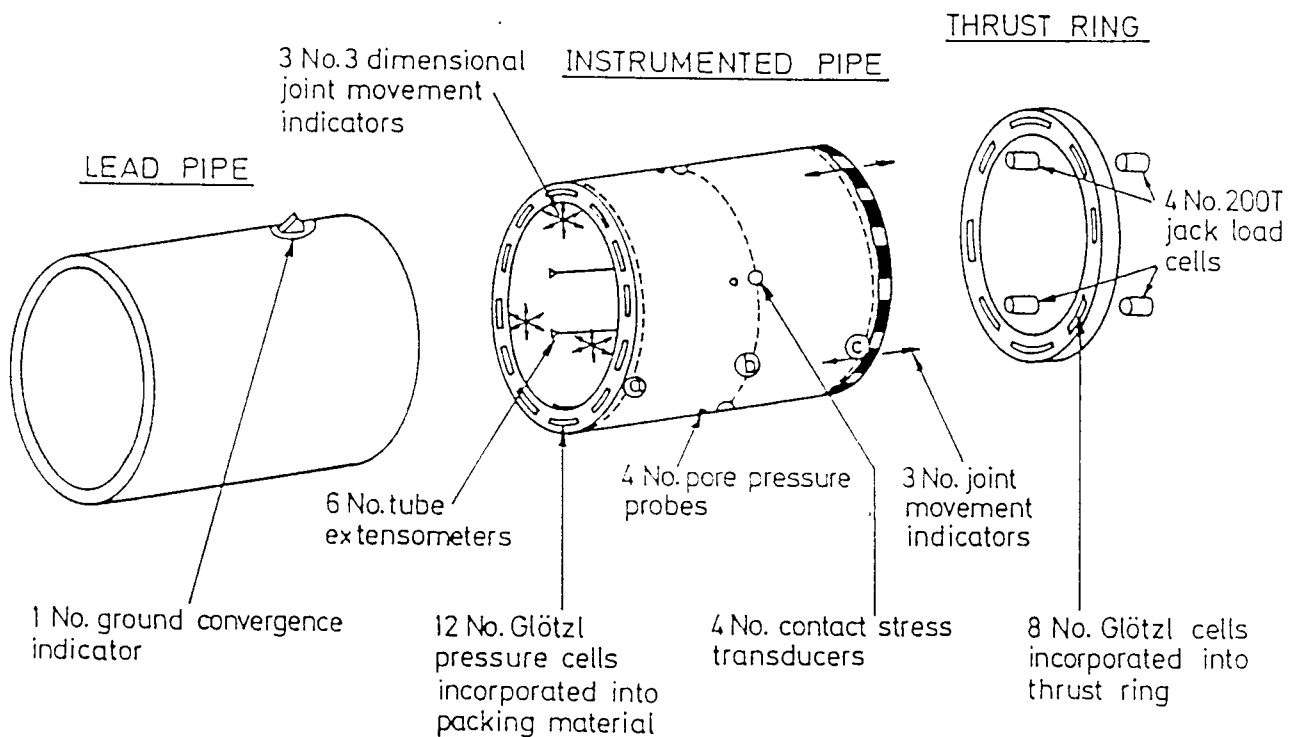


Figure 1. Schematic of instrumentation layout

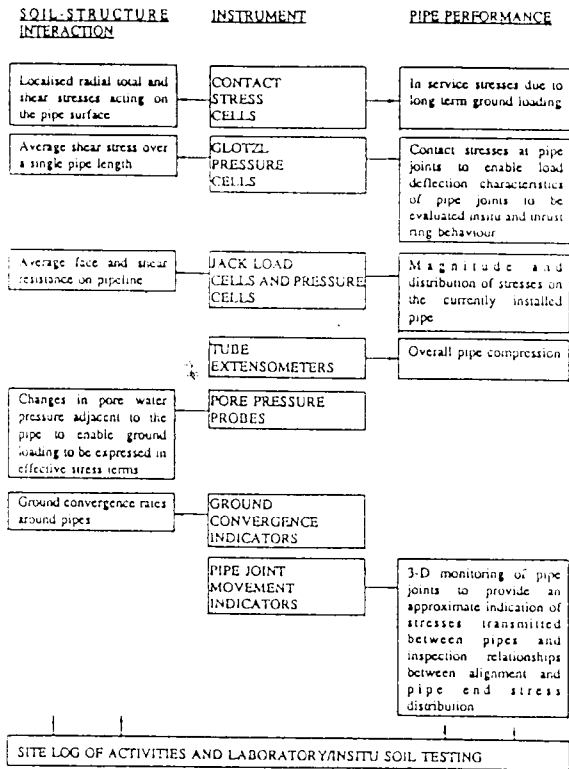


Figure 2. Flowchart indicating objectives of each instrument type.

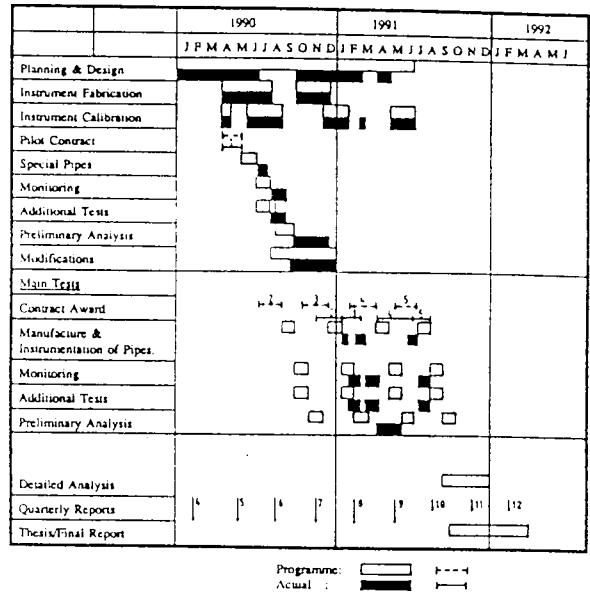



Figure 3. Pipejack research programme



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A detailed line drawing of the United States Capitol building in Washington, D.C., serving as the background for the central text.

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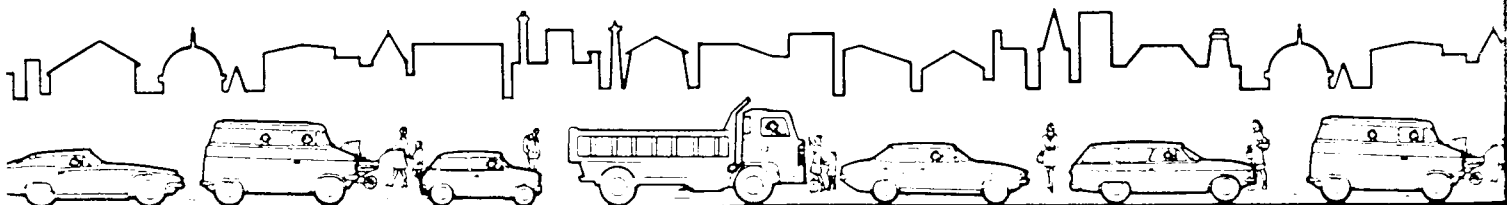
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National Association of Water Companies
National Society of Professional Engineers
National Utility Contractors Association

Water Environment Federation



PIPE END LOAD TRANSFER MECHANISMS DURING PIPE JACKING

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A series of five pipe jacks have been monitored during construction as part of an ongoing research programme. In each case a heavily instrumented pipe was incorporated into the pipe string, to allow measurement of 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.

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. Control of line and level is poorest at the start of a drive, while the tunnel surveys have shown that misalignments remain essentially unchanged during continued jacking, under a wide variety of conditions. The worst angular misalignments therefore tend to coincide with the most heavily loaded pipes. Since relative angular rather than absolute deviations control the load transfer mechanisms between pipes, which may lead to failures under high jacking loads, uncritical adherence to specifications based on absolute line and level may be counter-productive.

INTRODUCTION

This paper presents some of the results obtained from instrumentation and monitoring of a series of pipe jacks. The work forms stage 2 of an ongoing programme of research at the University of Oxford. Stage 1 involved testing of scale models of concrete jacking pipes under laboratory conditions; full details of this work are given in Ripley (1989). The current stage has involved incorporating an instrumented pipe into pipe jacked tunnels under construction on five sites, to confirm under actual field conditions the findings from the laboratory work and investigate aspects of performance which can only be studied realistically at full scale.

The planning and implementation of the research programme, including the choice of sites, problems encountered and progress made, are described by Milligan and Norris (1991); generally, the project has been very successful and yielded a vast quantity of valuable data. Details of the instrumentation and data acquisition systems used are given in Norris and Milligan (1991). However to understand properly the results presented here, it is necessary to summarise briefly the nature of the instrumentation and of the projects monitored.

The instrumentation system is shown in Figure 1. The instrumented pipe contains the following:-

- (i) contact stress cells to measure both radial and shear interface stresses between the pipe and the ground;
- (ii) pore pressure cells, close to the stress cells;
- (iii) joint movement indicators measuring movements across the joint at each end of the pipe;
- (iv) joint pressure cells incorporated into the packer in the joint at each end of the pipe;
- (v) extensometers fitted to the inner surface of the pipe to measure the compression of the pipe under load.

On the first site, a ground convergence indicator was used in the lead pipe. In the jacking pit the total jacking load was recorded by jack load cells, and the forward movement of the pipe string by a displacement transducer. Readings from the instruments were backed up by a detailed log of all site activities, a regular (usually 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.

Details of the five schemes monitored are given in Table 1. Pipe internal diameters ranged from 1200 to 1820mm, and both vertically cast and spun pipes were used. Ground conditions included a wide range from highly plastic London clay to loose sand and gravel, with cover depths from 1.5 to 21 metres. The first few schemes involved relatively short hand-excavated drives, while the final scheme used a slurry tunnel boring machine; this drive was also lubricated throughout, while scheme four used lubrication only during the later stages and the use of lubricants was avoided in the first three. 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 pressure cell readings. The instrumented pipe was located close behind the shield in the first scheme, and positioned progressively further back in the pipe string in subsequent schemes.

This paper concentrates on the measured tunnel alignments, the pressure distributions in the pipe joints, and the relations between them. Details of the instrumentation arrays at the rear joint of the special pipes for schemes 1, 3 and 5 are illustrated in Figure 2. Data has been selected from these schemes for the following reasons:-

- (i) Bolton, scheme 1 - tunnel at shallow depth in a stable bore, ground contact only at the base of the pipe, instrumented pipe near front end where face loading conditions dominate;
- (ii) Honor Oak, scheme 3 - deep tunnel in highly plastic clay, large contact pressures between pipe and ground;
- (iii) Cheltenham, scheme 5 - machine driven lubricated drive in sand and gravel below the water table, relatively large deviations from line and level and some pipe damage under jacking loads.

PIPELINE MISALIGNMENT

Excavation at the face of a pipejack 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 the pipe joints. Specifications for pipe jacking

normally require the contractor to maintain records of the position of the first pipe and the associated installation jacking forces.

Throughout the current research regular surveys (usually daily) were carried out for the full length of the tunnel. The resulting plots are presented in Figures 3, 4 and 5. For clarity only three surveys are shown for each scheme. Schemes 1 and 3 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 position A of Figure 5. Examination of the change in position of pipes reveals that misalignment of the pipelines did not significantly alter throughout construction and that 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 pipejack while personnel are familiarising themselves with the ground conditions should reduce this effect.

The importance of directional control of a pipejack is illustrated in Figure 6. Specifications often state $\pm 50\text{mm}$ as the allowable tolerances on line and level. However an example presented by Hough (1986) shows how large deflections can become. Two 2.5m long pipes can have a maximum deflection angle of 4.6° and yet still be within line and level tolerance. Deflections of this magnitude considerably exceed the 1° angular deflection of the quality assurance control tests of Table 6 of 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 axial loads are being applied but to assess the three dimensional orientation of pipes to enable full interpretation of misalignment angles. The three dimensional misalignment angle can readily be determined from line and level surveys as demonstrated in Figure 7. Plots of the angular misalignment derived from the line and level surveys for Schemes 1 and 3 are included in Figures 3 and 4. In general the plots indicate that deflection angles are less than 0.25° on the two 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 diameters, 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 previous assessment of pipe deflection is generally only applicable to unloaded or partially relaxed pipe lines 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. Results of changes in pipe alignment at the rear instrumented joint are presented for schemes 1 and 3 in Figure 8. Values at the start and end of each push are recorded. The value of beta, during a push, increases or decreases dependent upon the pipeline misalignment. Pipes do not necessarily realign under load. It is apparent that the variation in angular misalignment for scheme 1 is greater than for scheme 3. This would be expected because the instrumented joint in scheme 1 is 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 in scheme 3 shows excellent

agreement with the position of rapid change in line and level in Figure 5. 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 maximum misalignment angles were 0.45° for scheme 1 and 0.3° for scheme 3; misalignment angles were generally smaller than 0.2° in both schemes. Although the misalignment angles for the Cheltenham scheme have not been presented it is worth noting that at the location of pipe damage an angular deviation of 0.47° was recorded.

LOAD TRANSFER AT JOINTS

The field work allows correlation of measurements of joint gaps to deflection angles, load distribution at joints and compression of the packing material. Examples of the results at a selection of angular misalignments are presented in Figures 9 a-f. To explain the behaviour of the various examples it is necessary to consider the characteristics of the packing material used in the joints. Figure 10 presents the results of a series of uniaxial compression tests on dry Medium Density Fibreboard (MDF) which has been used throughout the fieldwork. 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/mm² 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.

The plots of Figures 9a, e and f indicate good agreement between the measured stress distributions and angular misalignments at the maximum values of angular deviation. This is because the previous stress history of the packer has negligible affect 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 9b, the relationship between angular measurement and stress distributions 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 11, 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.

Figure 12 compares the applied stress and joint compression from a single cell with the laboratory compression tests. The field data indicates that the unloading stiffness of the precompressed material should be considered for pipe design. It is suggested that an appropriate value is the slope of the tangent to the unloading curve of the stress cycled material at a stress level equivalent to the compressive strength of the concrete.

CONCLUSIONS

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 a location at which maximum pipe loads will occur later in the drive, giving rise to severe pipe loading conditions.

The general pattern of stress distribution in the joint matches that suggested by Milligan and Ripley (1989). Detailed comparisons are difficult to make because the compression of the packer is influenced by its stress history throughout a drive, which may be very complex. As a result, stress concentration may occur in the joints even when angular misalignments are small. The field data shows that behaviour tends to that of precompressed material, and that for pipe design an appropriate stiffness value should be used.

The field data also shows that angular misalignments of 0.25° are typical in well-constructed pipejacks, and that 0.5° is not uncommon. By such an angle the localisation of end stresses is already severe, and will usually dominate the structural design of the pipe. Further improvements in packing material and perhaps joint design will be needed if large pipe end loads are to be transmitted safely through such angles.

ACKNOWLEDGEMENTS

This research has been made possible by the generous financial support of the Science and Engineering Research Council, the Pipe Jacking Association, and five water service companies: Northumbrian, North-West, Severn Trent, Thames and Yorkshire. The assistance and support of many individuals through the project management group and in connection with each of the sites is also gratefully acknowledged.

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	1	2	3	4	5
Date	August 1990	January 1991	March 1991	July 1991	September 1991
Location	Bolton, Lancs	Gateshead, Tyneside	Honor Oak, SE London	Chorley, Lancs	Cheltenham
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)	1.5 - 1.7	7 - 11	11 - 21	5 - 7	4 - 7
Drive Length (m)	65	110	80	160	350
Position of Test Pipe	Pipe No.3	Pipe No.10	Pipe No.15	Pipe No.15	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.

Table 1 Details of the instrumented schemes.

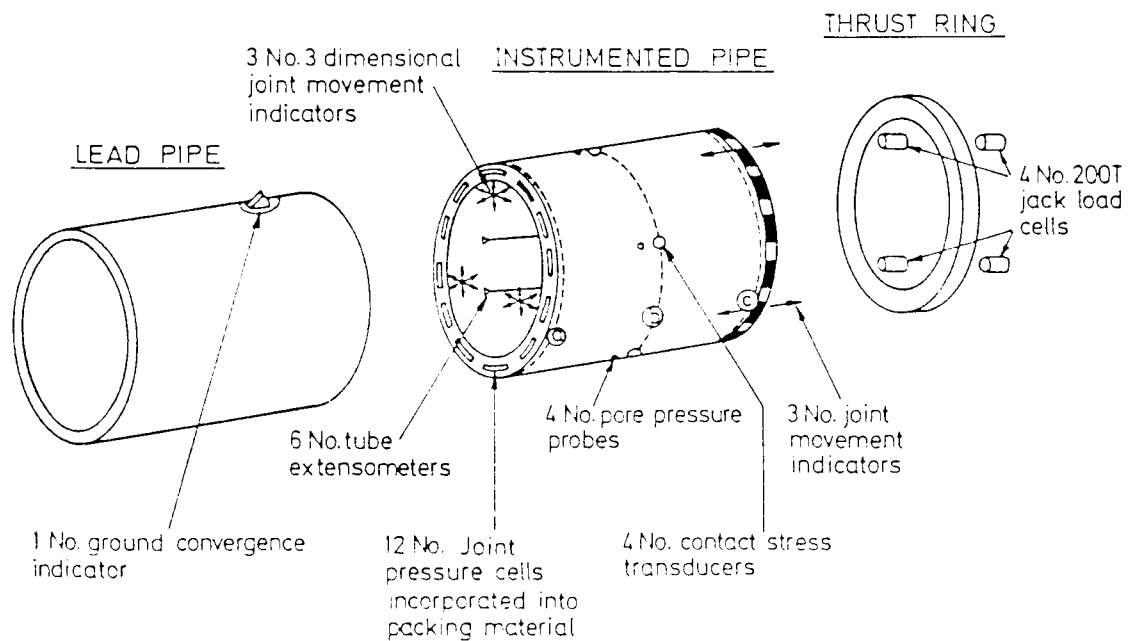
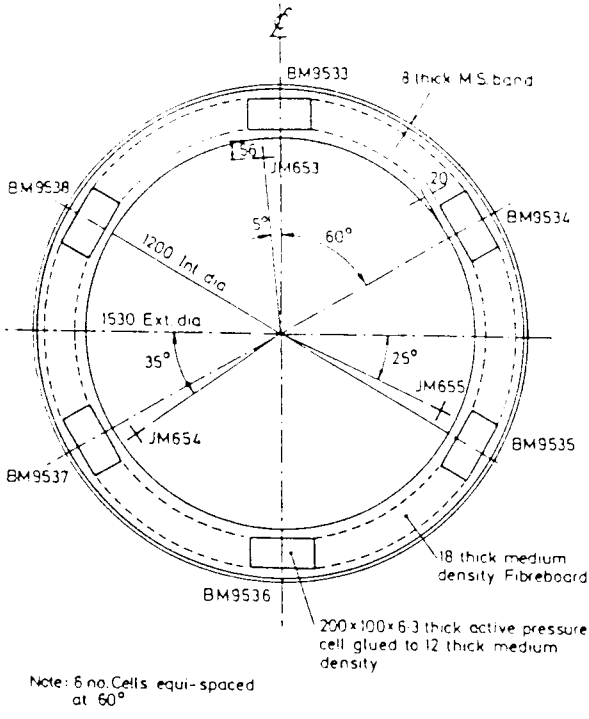
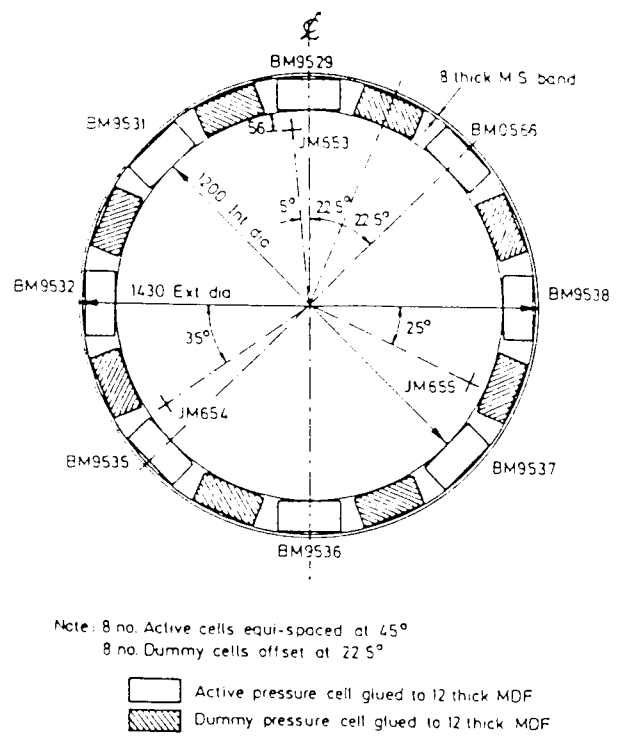


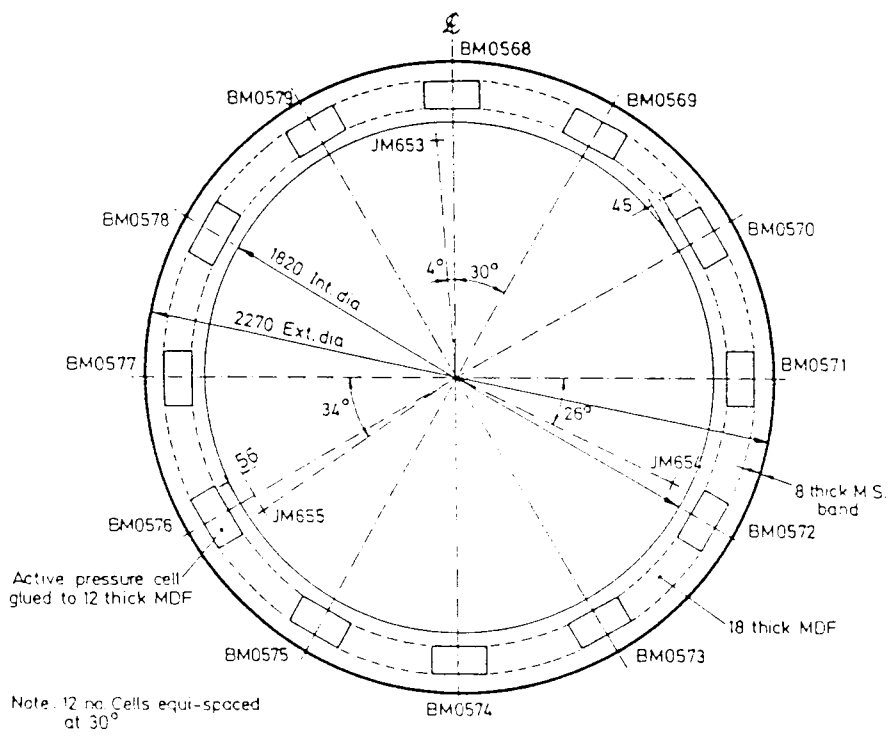
Figure 1 Schematic of instrument arrangement



BOLTON



CHELTENHAM



HONOF OAK

Figure 2. Schematic of Instrumented Rear Joints.

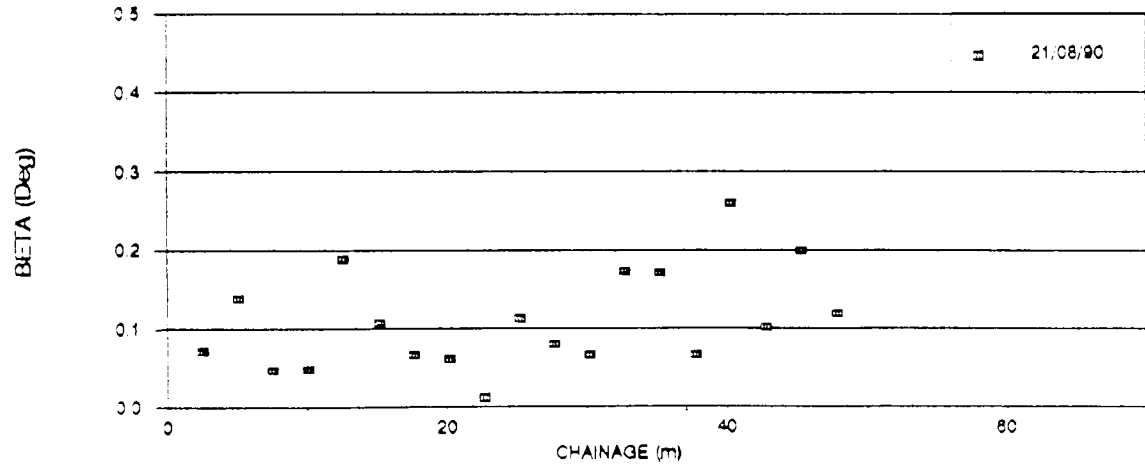
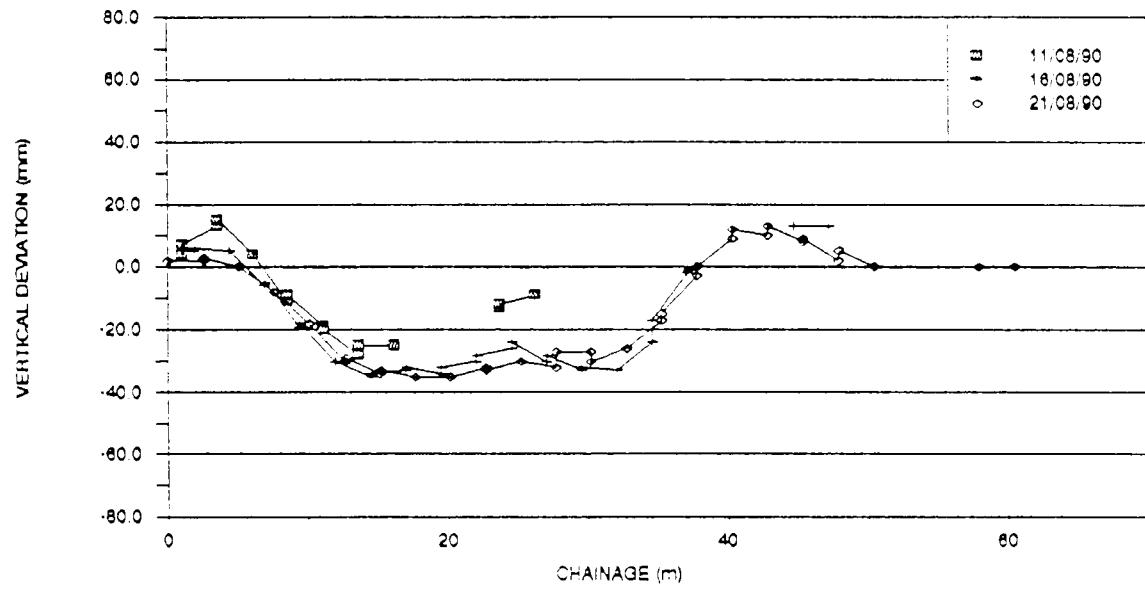
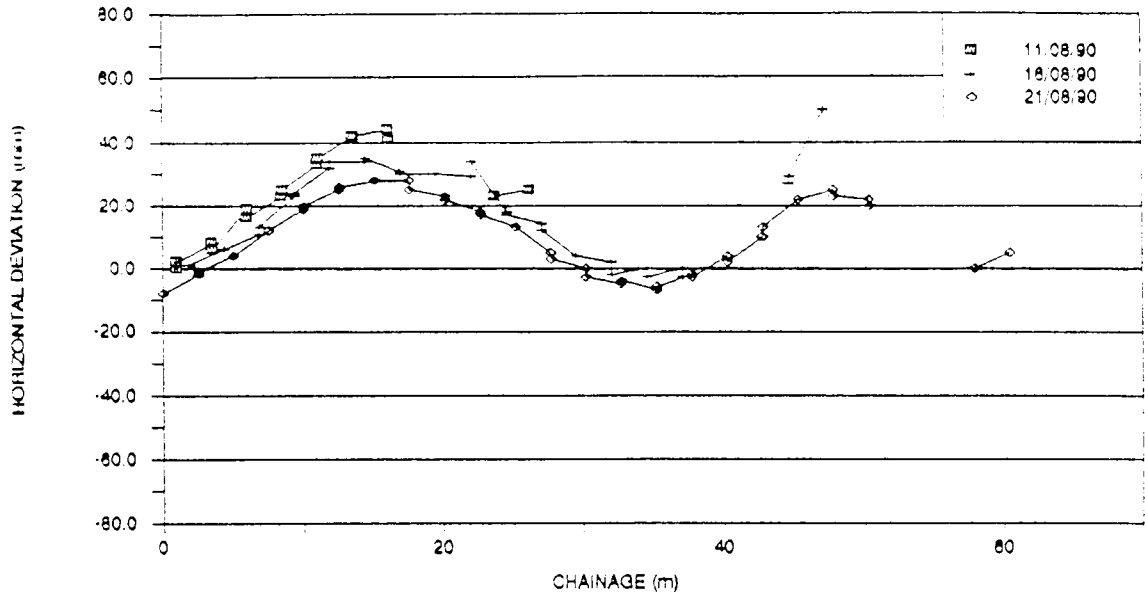


Figure 3. Bolton Tunnel Alignment Surveys.

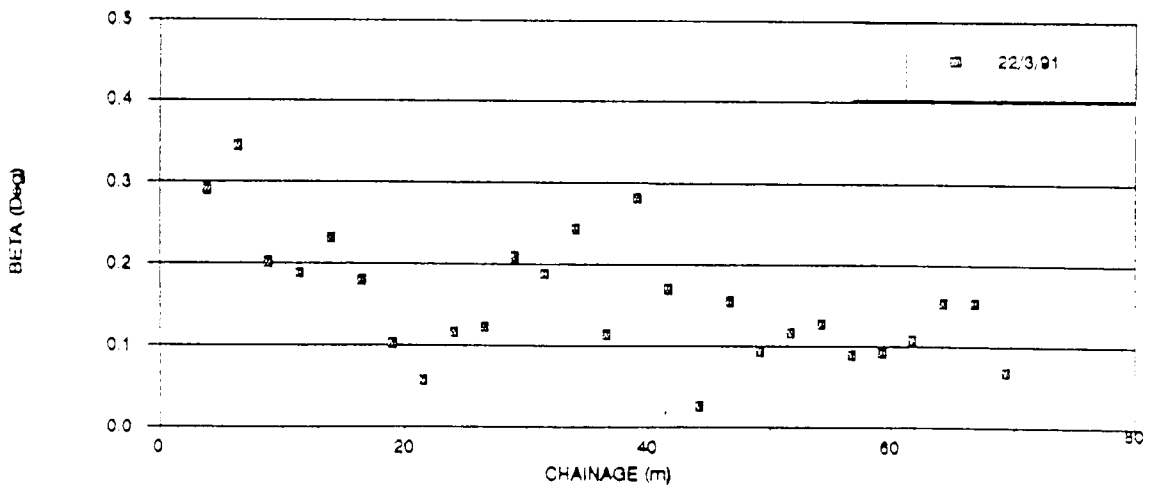
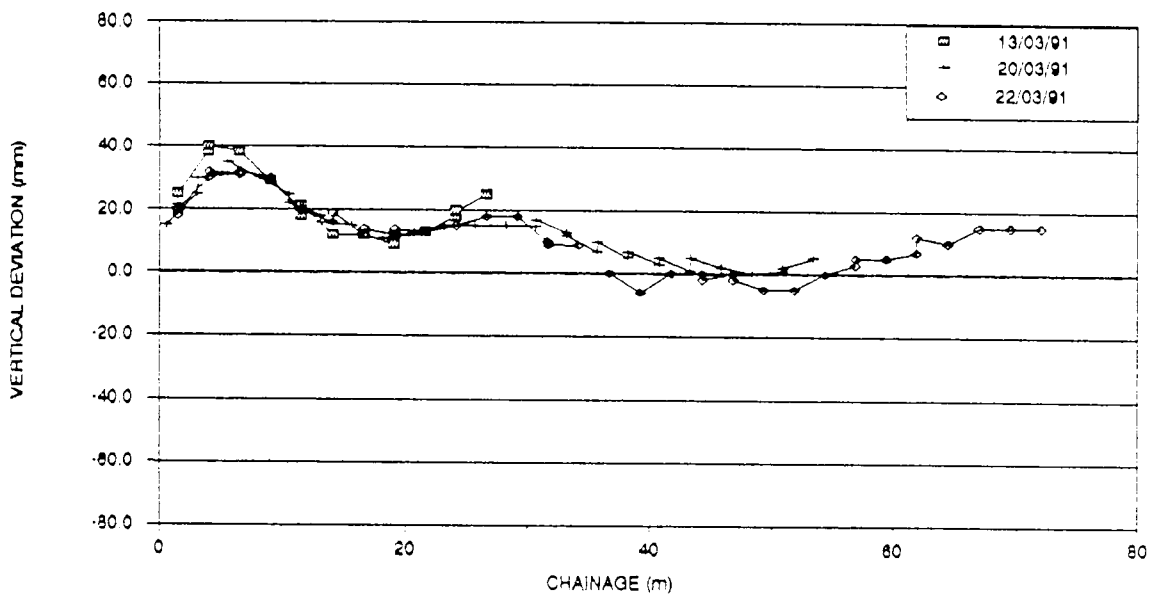
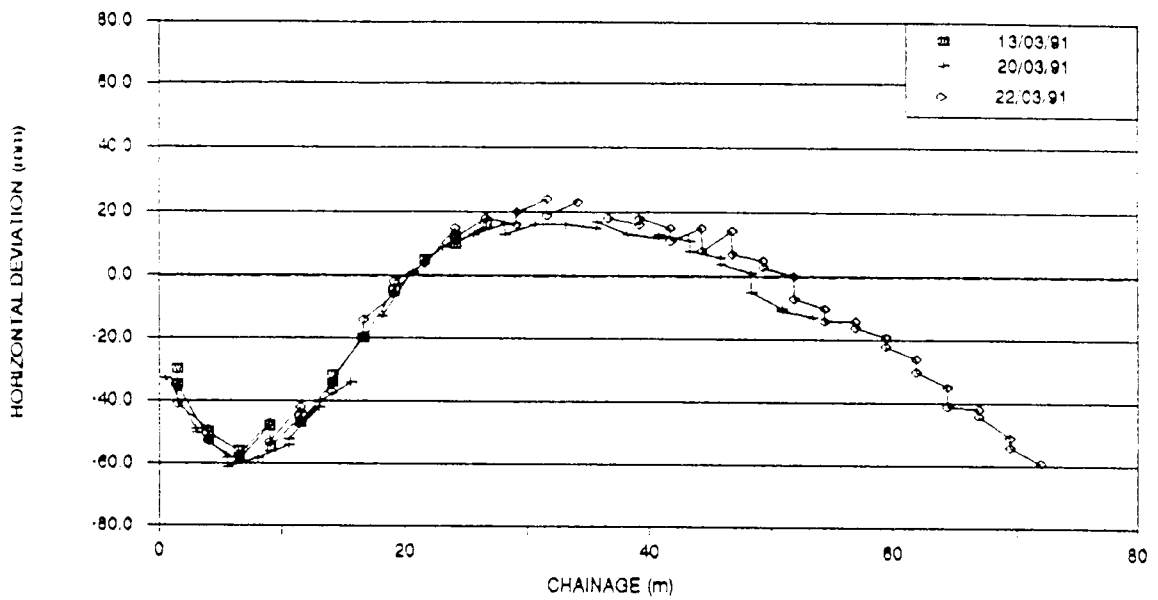


Figure 4. Honor Oak Tunnel Alignment Surveys.

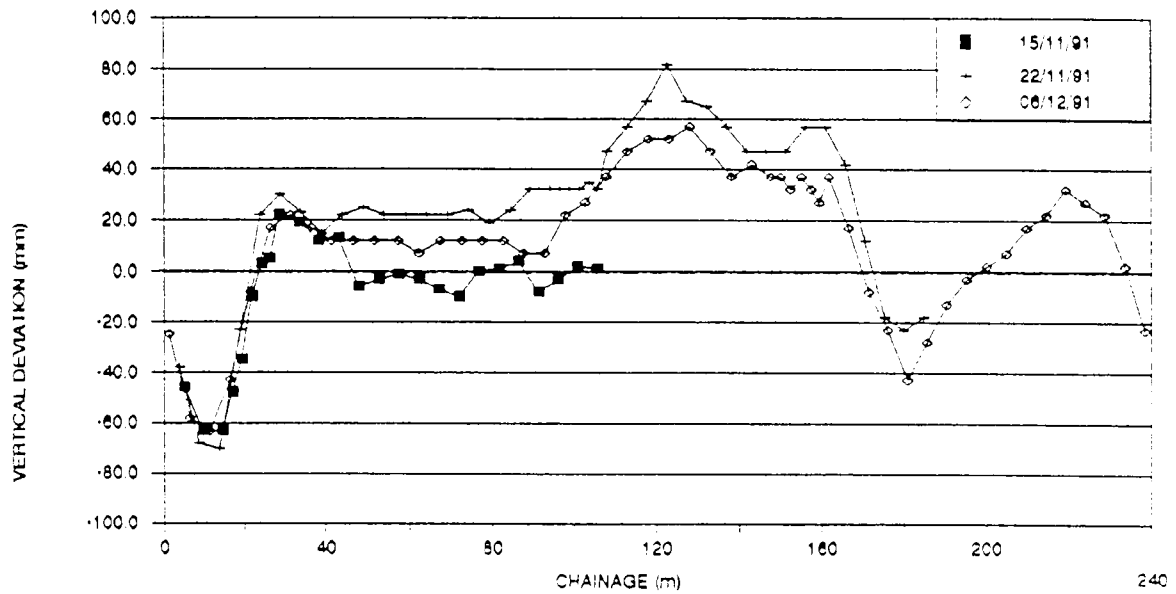
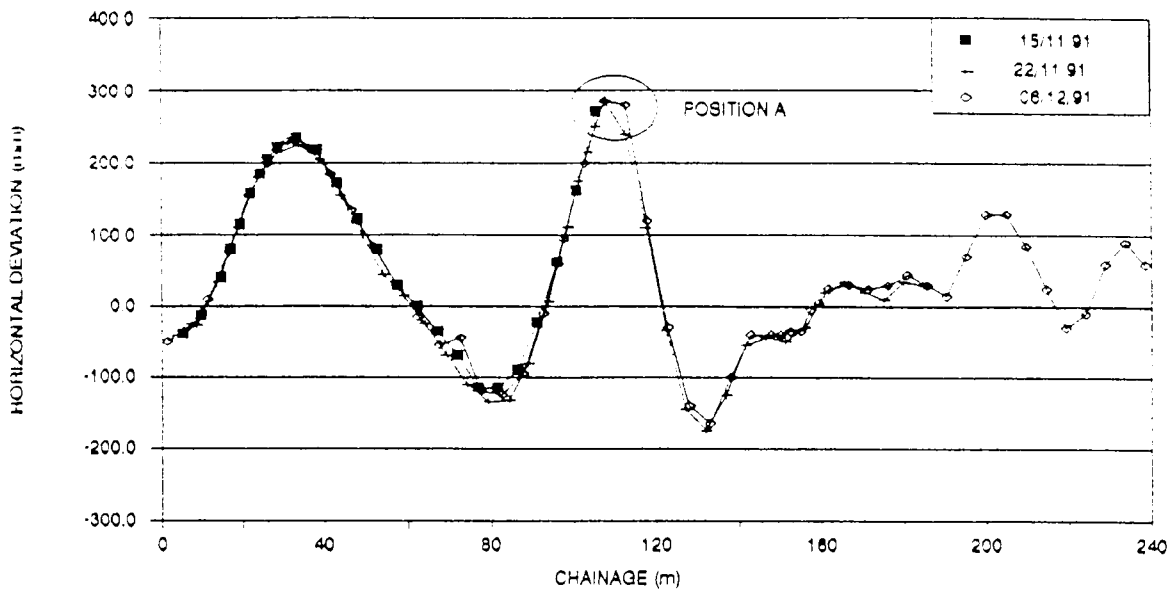
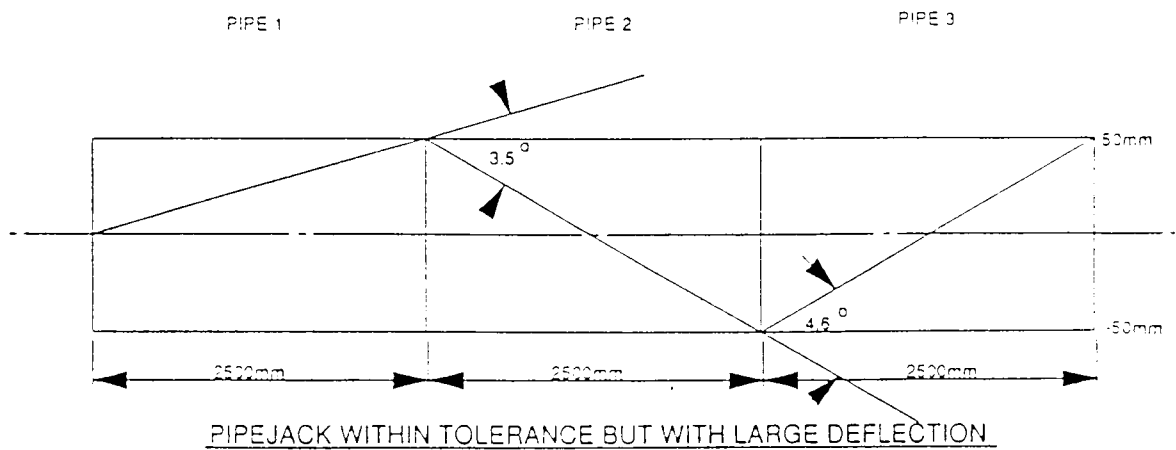
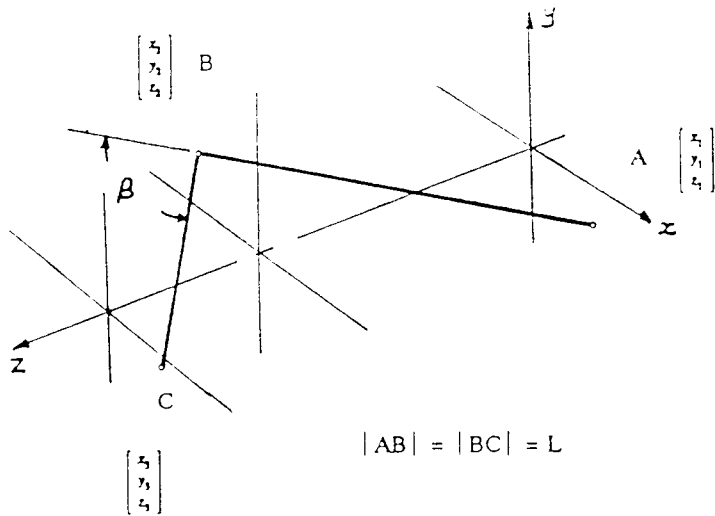


Figure 5. Cheltenham Tunnel Alignment Surveys.



PIPEJACK WITHIN TOLERANCE BUT WITH LARGE DEFLECTION

Figure 6.



Direction Cosines of AB are:

$$l = \frac{x_2 - x_1}{L}$$

$$m = \frac{y_2 - y_1}{L}$$

$$n = \frac{z_2 - z_1}{L}$$

where $(x_2 - x_1)^2 + (y_2 - y_1)^2 + (z_2 - z_1)^2 = L^2$

(ie) $l^2 + m^2 + n^2 = 1$

Likewise direction cosines of BC are:

$$l' = \frac{(x_3 - x_2)}{L}$$

$$m' = \frac{y_3 - y_2}{L}$$

$$n' = \frac{z_3 - z_2}{L}$$

By definition, misalignment angle β given by:

$$\cos \beta = ll' + mm' + nn'$$

Figure 7. Determination of misalignment angle from line and level surveys.

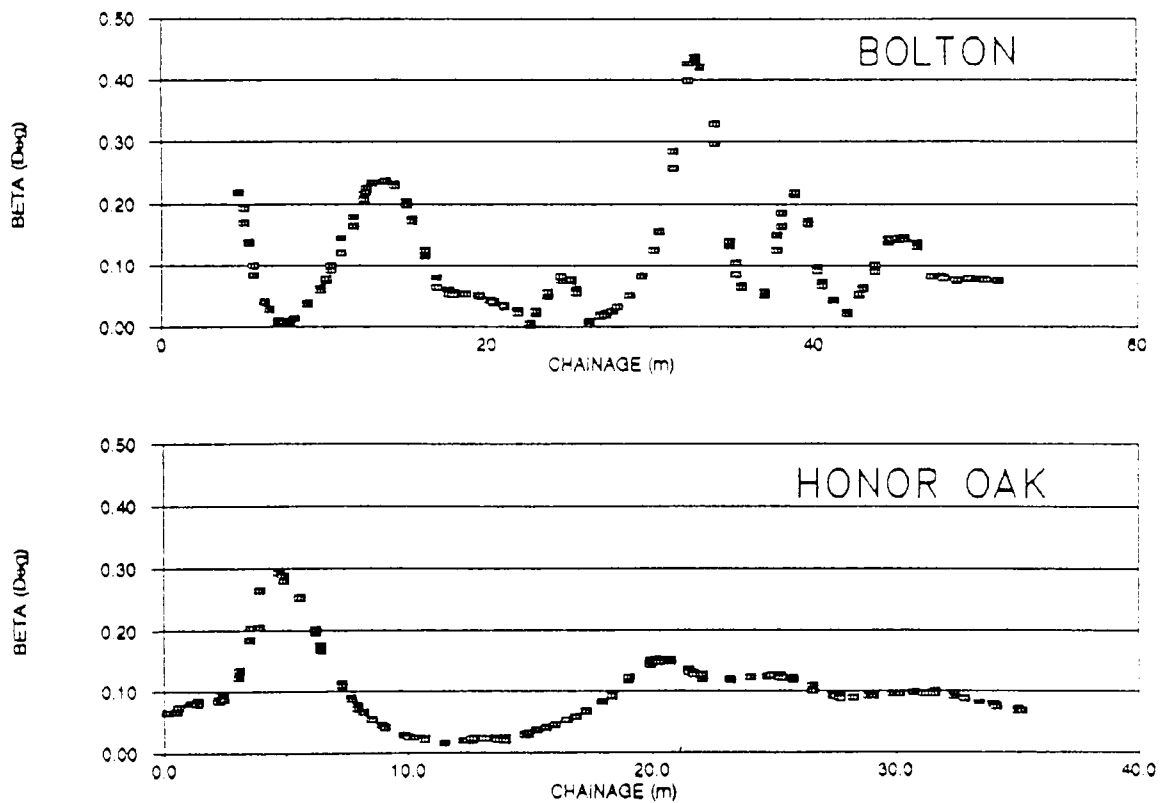


Figure 8. Misalignment angles under load.

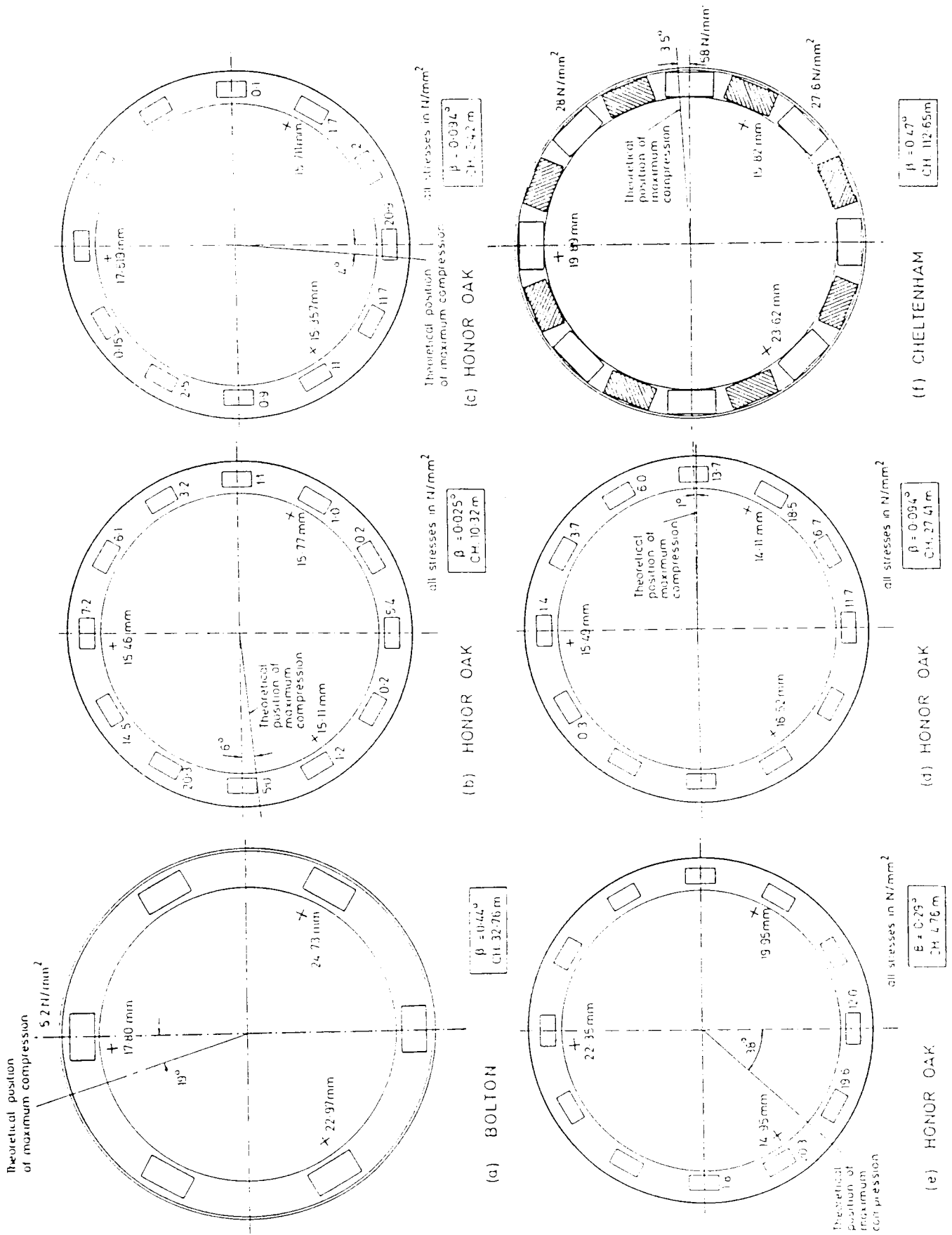


Figure 9. Relationship between measured joint angle and pressure distribution.

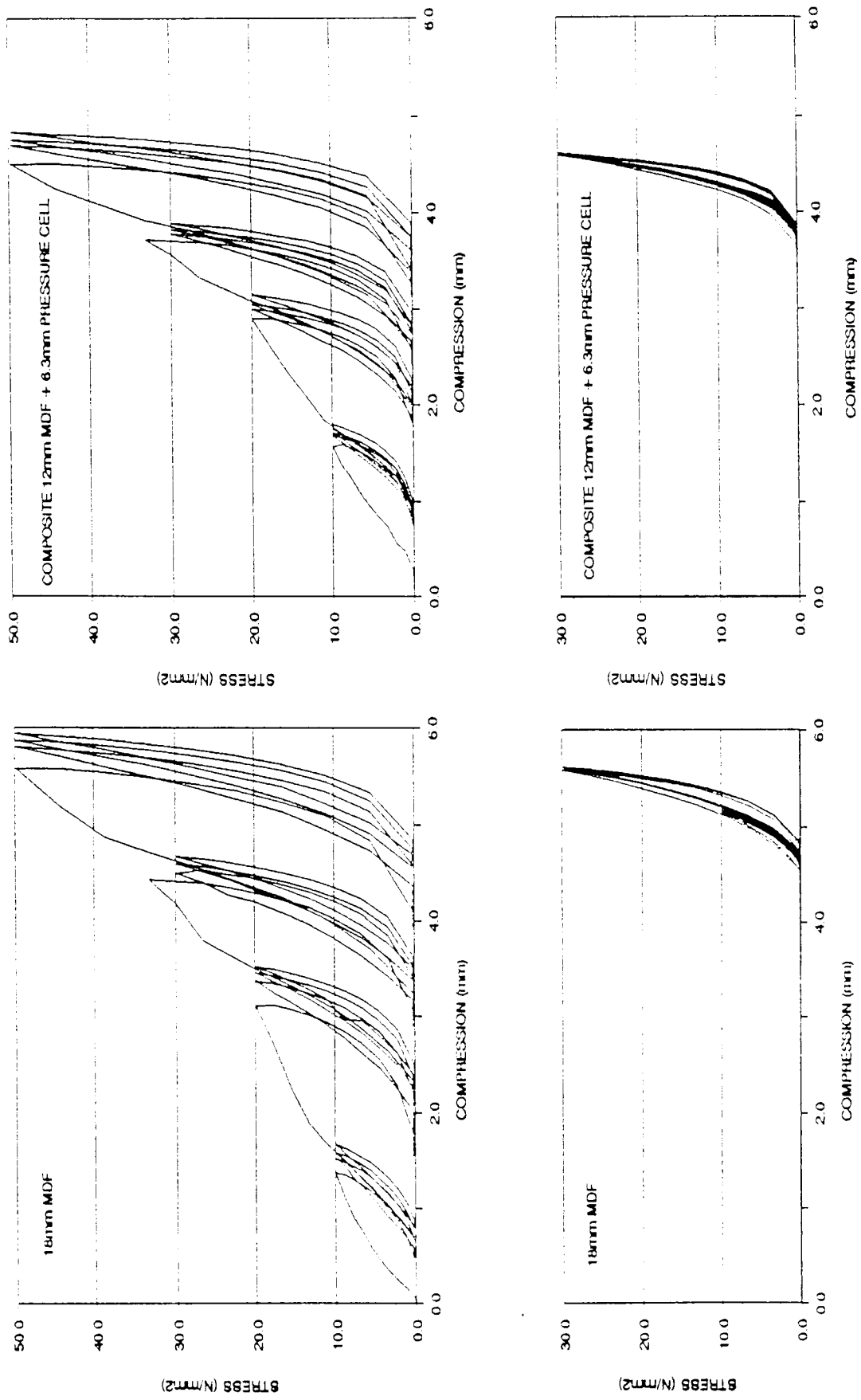


Figure 10. Stiffness response of joint packer.

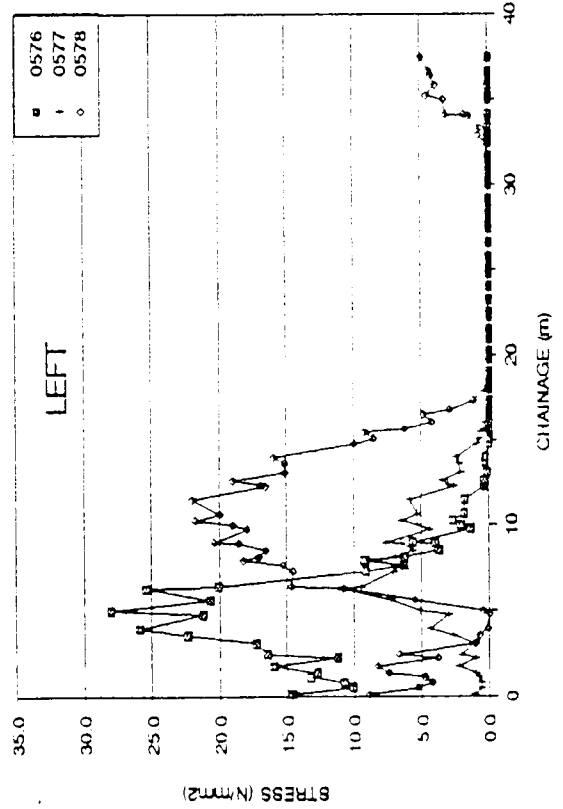
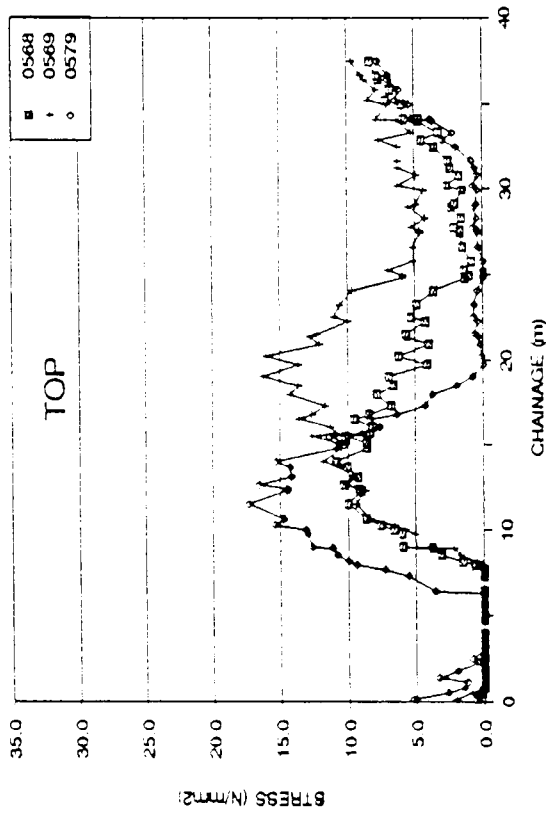
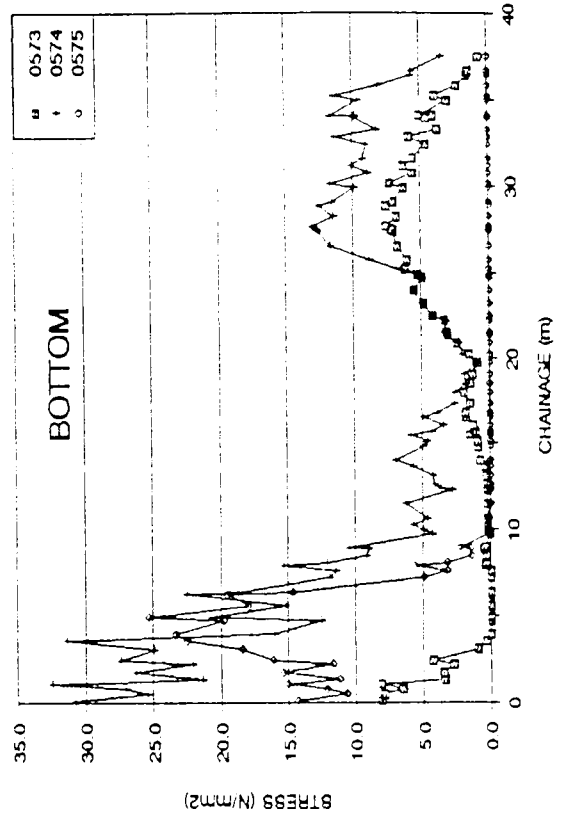
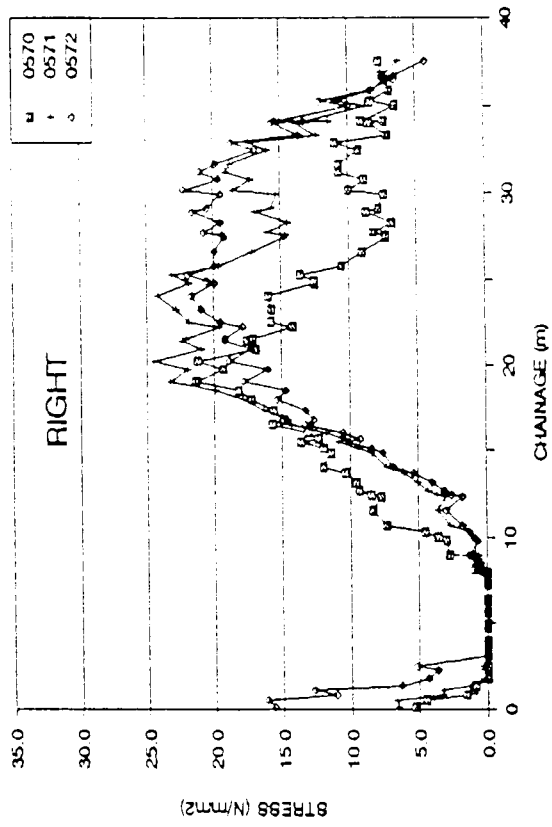


Figure 11. Variation in joint pressure distribution for Honor Oak.

INSITU STIFFNESS EVALUATION COMPOSITE 12mm MDF + PRESSURE CELL

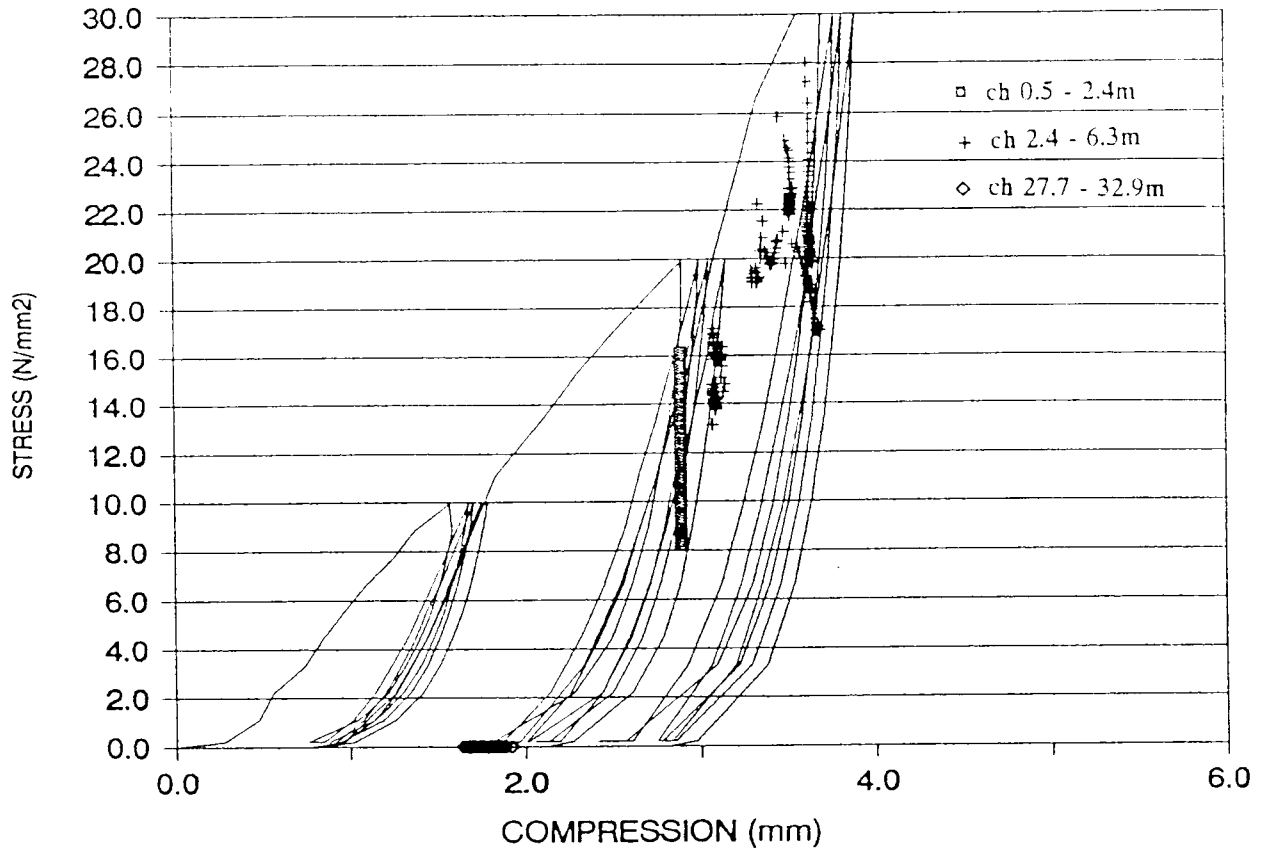


Figure 12. Joint stress-compression values during scheme 3 at cell BM0576.

Frictional resistance of jacked concrete pipes at full scale

Paul Norris & G.W.E. Milligan

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ABSTRACT: For the past 6 years a programme of research on the pipe jacking technique has been in progress at Oxford University to clarify how pipes interact with each other and the ground. This paper is concerned with pipe-soil interface data obtained from the incorporation of an instrumented pipe on five construction sites. The results show that large localised radial and frictional stresses can be generated during jacking, which appear to be a function of pipeline misalignment.

1 INTRODUCTION

An important consideration for pipe jacking is the amount of friction generated when the pipe is pushed into the ground. This friction contributes to the jacking resistance and is a major factor in determining the required capacity of the main thrust rams and whether intermediate jacking stations will be required. The magnitude of the pipe friction depends on the pipe size and material, type of soil, its moisture content and grading, depth of cover and the details of the construction equipment and procedures employed. Factors such as the amount of overcut by the shield, misalignment of the pipes, excavation methods, duration of work stoppages, and whether or not a bentonite injection system is used will also affect the amount of friction.

This paper presents some of the results obtained from instrumentation and monitoring of pipejack tunnels under construction on five sites. The instrumentation system is shown in Figure 1. The instruments of particular significance to the paper include:-

1. Contact stress cells which measure both radial and shear interface stresses between the pipe and the ground;
2. Pore pressure probes, close to the stress cells;
3. Jack load cells attached to the rams in the

jacking pit;

4. A displacement transducer to measure the forward movement of the pipe string.

Details of the five schemes are given in Table 1, where it will be seen that the instrumented pipe was located close behind the shield in the first scheme, and positioned progressively further back in the pipe string in subsequent schemes. Data has been selected from schemes 1, 3 and 4 to investigate global and local friction values in the various soil types and the effects of construction factors on their magnitude.

Ground conditions at these three sites are summarised in Table 1.

2 TOTAL JACKING RESISTANCE

The provision of sufficient jacking capacity is largely based on previous experience from schemes in similar ground conditions. The total jacking load depends upon both the force required to push the shield into the excavation, referred to as face resistance and the frictional resistance along the pipe length, Table 2. The face resistance varies between 6-9% of the total jacking load for the two cohesive drives. The high face resistance for scheme 2 is a result of the shield trimming mudstone in the invert. Scheme 4 provided an opportunity to deliberately

Table 1. Details of the instrumented schemes

	1	2	3	4	5
Pipe i.d (mm)	1200	1350	1800	1500	1200
Cover (m)	1.5-1.7	7-11	11-21	5-7	4-7
Length (m)	65	110	80	160	350
Test Pipe	No.3	No.10	No.15	No.15	85m
Lubrication	No	No	No	No/Yes	Yes
Excavation	Hand	Hand	Hand	Hand	Slurry TBM
Ground Type	Stiff Glacial Clay	Weathered Mudstone	London Clay	Dense Silty Sand	Loose Sand and Gravel
Particle Size Dist. (clay, silt, sand)	Not determined		36, 62, 2	15, 55, 30 Laminated silty sand 5, 10, 85 Silty sand	
Soil density (Mg/m ³)	2.1-2.3	Mudstone properties not available	2.0-2.1	1.7-1.9	Scheme not considered in this paper.
Index Tests w PL % LL	12-15 17 22-29		24-29 23-30 67-80	17 NP Analysis for LSS 22	
Triaxial s_u (kPa)	150-300		225-600	-	
In situ s_u (kPa)	130-260		Too Stiff	N/A	
Consolidated undrained ϕ' deg	5 33		50 31	-	

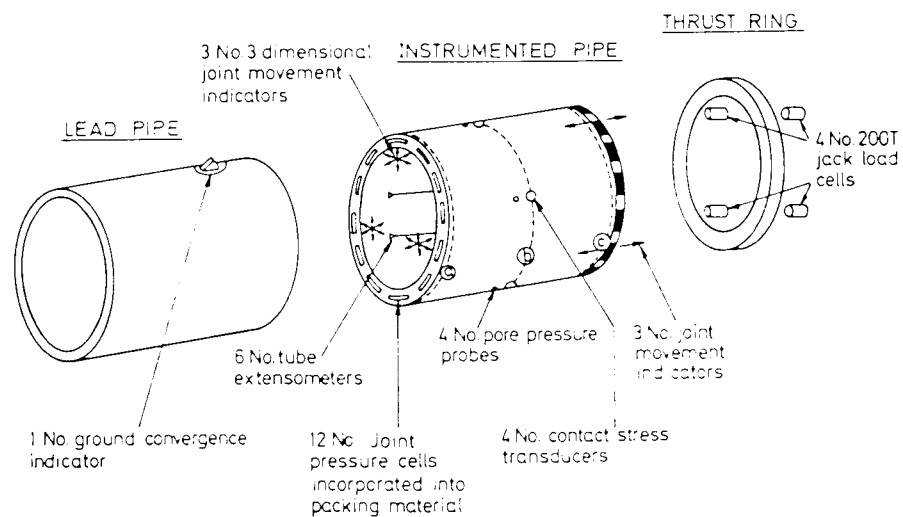


Fig.1. Schematic of instruments

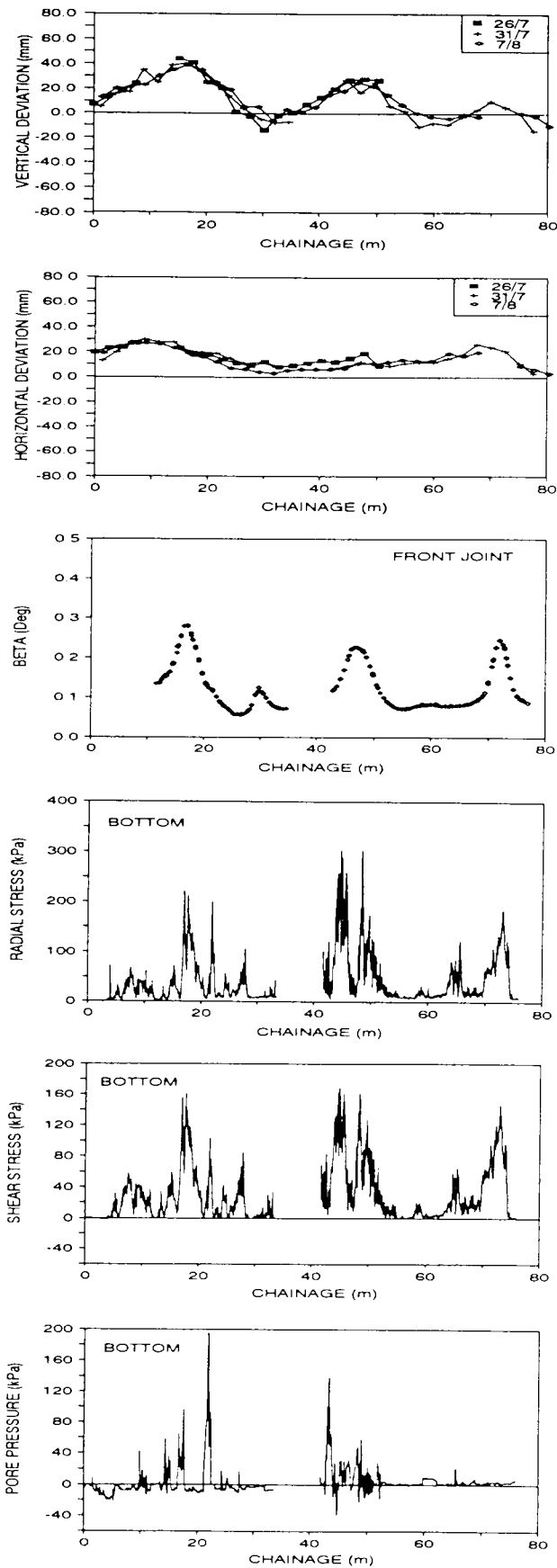


Fig.2 Local contact stresses from scheme 4.

monitor the effects of changes in shield trimming and hence face resistance. The large variation of 3% to 25% is a result of changing the extent of excavation from outside the shield to 20mm or so inside with extra trimming in the invert. The frictional resistances are generally at the lower end of the Craig limits in Table 2, which is probably a function of the competent nature of the ground and the well controlled alignment of the drives.

3 LOCAL INTERFACE STRESSES

Local contact stresses have been recorded using contact stress cells and pore pressure probes positioned at crown, invert and pipe axis. The major contact stresses were mobilised at the bottom of the pipes, Table 3, apart from scheme 3 through heavily overconsolidated London clay in which lateral ground stresses up to 650kPa were sufficient to damage the instruments on the tunnel axis. A typical set of responses along the bottom interface of scheme 4 are presented in Figure 2. The plots clearly show that peak values of radial, frictional and pore water pressures are obtained over short lengths of the drive with a lower overall average value for the total length. The local peak skin friction values are nearly two orders of magnitude larger than the average friction values in Table 2. The radial stresses exhibit peak values up to 300kPa over short lengths of 1-2m.

4. GROUND RELATED FACTORS

4.1 Soil type

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 have been produced for schemes 1 and 4. The data from the bottom probes of both schemes are presented in Figure 3. A number of salient features are evident. Best line fits to the plots indicate only small differences between the total and effective stress behaviour. This suggests that a partially drained state exists at the pipe-soil interface with

Table 2. Average face resistance and pipeline friction values

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

* Value based on 40m monitored length. Total drive length 100m

Table 3. Maximum local interface stresses

Scheme	Full overburden (γh)	Radial (kPa)			Shear (kPa)			Pore Pressure (kPa)		
	(kPa)	Top	axis	bottom	Top	axis	bottom	Top	axis	bottom
1	37	5	10	550	5	10	250	-5	NI	400
2	242	NI	NI	550	NI	NI	160	NI	NI	700
3	300	450	650	IF	150	150	IF	IF	250	250
4	126	100	100	300	60	60	60	10	10	190

NI - No instrument IF - Instrument failure

Table 4. Predicted and measured coefficients of skin friction

Scheme	Internal angle of friction ϕ' (deg)			Skin friction coeff.	Predicted δ' (deg)			Field δ (deg)
	Peak	Critical State	Residual		Peak	Critical state	Residual	
1	33av	30 Bolton (1979)	25.3 (Lupini et al 1981)	0.68 (Potyondy 1961)	22.4	20.4	17.2	19
4	47 Bolton (1979)	32 Bolton (1979)	-	0.87	40.9	27.8	-	37.7

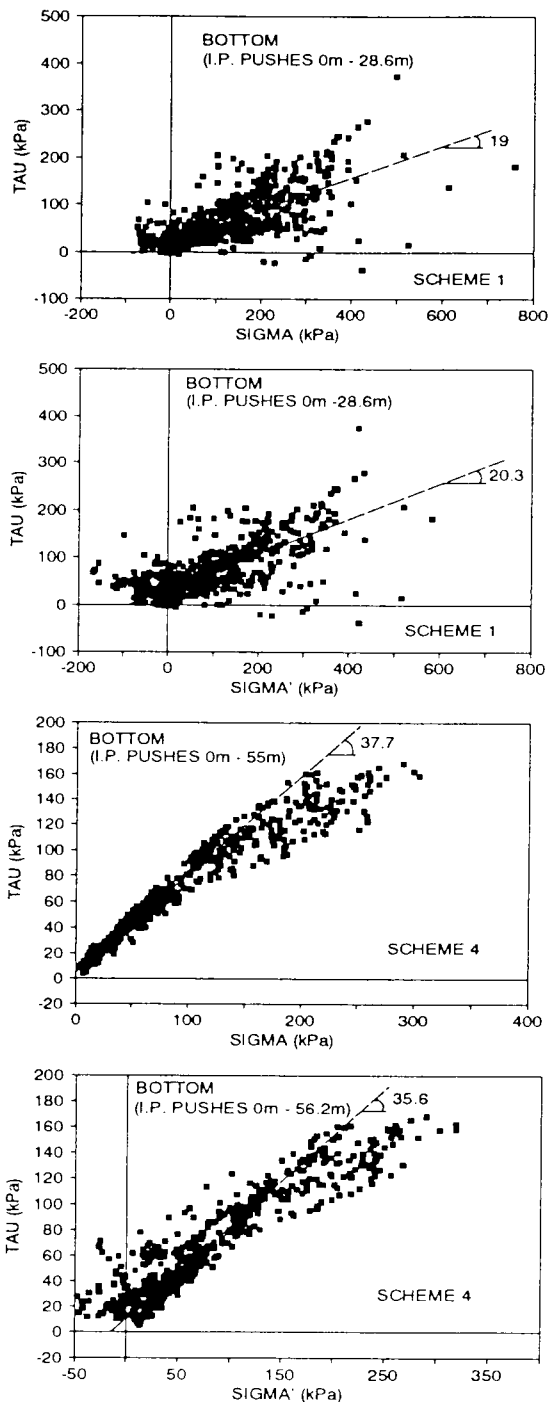


Fig.3 Shear stress/radial stress relationships.

the concrete pipe acting as a local drainage path. Both materials exhibit frictional responses with coefficients of skin friction between the pipe and soil of 19° for the glacial clay and 37.7° for the dense silty sand. These values fall within the range of typical peak, critical and residual friction angles in Table 4.

Given the degree of scatter of the field results, there is some evidence to suggest that the

repeated passage of pipes subjects the clay to large shear displacements which reduces the friction angle below the critical state value while the silty sand material illustrates a post peak response which has not reached the critical state.

During the later stages of scheme 1 the moisture content of the soil was increased following a period of heavy rain and temporary inundation of the tunnel due to a fractured water main. As a result, the measured angle of skin friction was reduced to 13.5° ; this is consistent with values of skin friction between smooth concrete and cohesive granular soils at varying moisture content measured in laboratory tests by Potyondy (1961). However the overall resistance to jacking increased considerably (see Table 2) due to larger radial effective stresses as the soil swelled, closed the overbreak and made contact with the pipe around its full perimeter.

4.2 Pipe self weight friction

Comparison of the pipeline self weight friction with measured average frictional resistance is presented in Table 5.

Reasonable agreement is obtained for schemes 1, 2 and 4 which are through relatively stable bores. The larger recorded values are possibly 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 required to obtain closer agreement for scheme 3.

5. CONSTRUCTION RELATED FACTORS

5.1 Misalignment

Comparison of tunnel alignment data and local interface stress profiles, Figure 2, illustrates reasonable agreement between the positions of peak stress values and maximum angular deviation (β). The relationship is reproduced in Figure 4 as a scatter plot. Although there is considerable variation in the relationship, larger stresses generally occur at larger angular deviations. The scatter is probably a function of the precise profile of the ground in relation to the locus of pipe movement, the necessary ground

Table 5 Pipe self weight friction

Scheme	Field skin friction δ	W tan δ (kN/m)	Av friction (kN/m)
1	19°	6.1	7.2 wet 29.8 dry
2	16°	6.6	8
3	14°	8.8	54.4
4	37.7°	18.7	23.1 unlub 9.4 lub

support varying along the pipe length.

5.2 Time factor

Increases in pipejacking forces in clay soils following a lapse in jacking are routinely recorded. The effect was evident in the jacking records for the low plasticity drive of scheme 1 although a more dramatic effect was observed in the high plasticity London Clay of scheme 3 which forms the basis for discussion. The jacking record for a short section of the drive and the associated total radial stresses acting on the pipe at the start and end of each stoppage are presented in Figure 5. 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. The total radial stress plot however indicates reductions during the rest periods which are not a function of changes in jack ram loading during the stoppage. Unfortunately there is no reliable pore pressure data from the drive to establish whether the reduction is a result of large pore pressure dissipation.

The occurrence of large restart forces after short pauses have been observed during laboratory controlled 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 and highlight a peak resistance which drops rapidly to a mini-

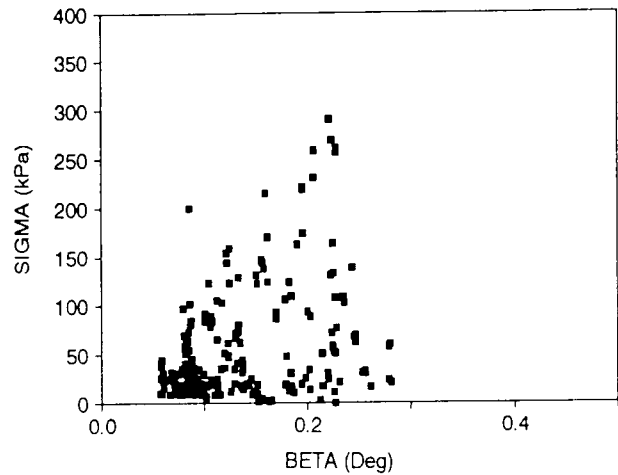


Fig.4 Relationship between local radial stress and Beta; Scheme 4.

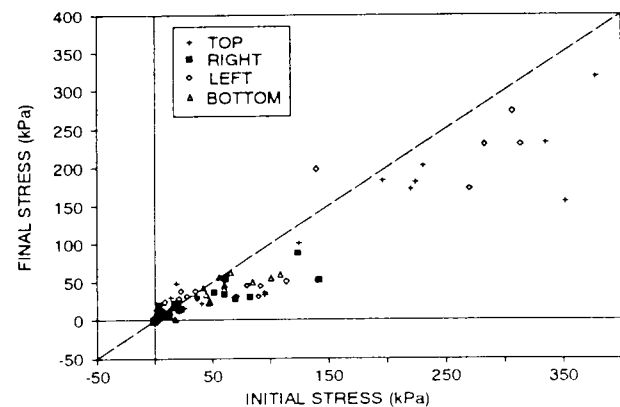
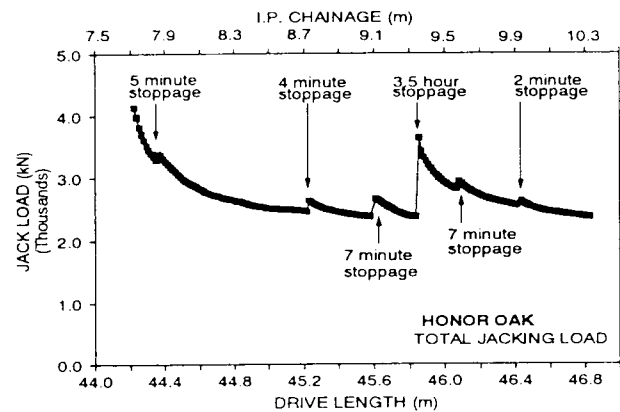


Fig.5 Time dependent changes in jacking load and local interface stresses.

imum value with fast displacement; the magnitude of frictional resistance increasing with increasing rate of shear. Typical jacking rates fall between these two values, scheme 3 was 335mm/min (2 rams) and 185mm/min (4 rams). Tika suggests that the fast rate of shearing may

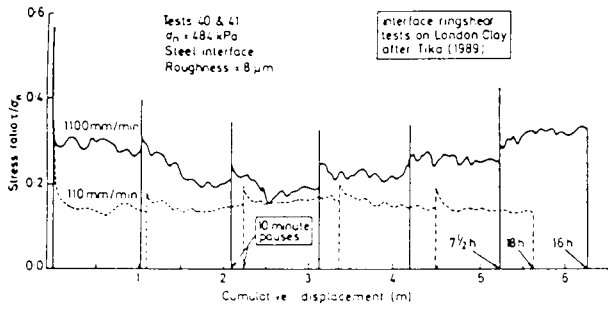


Fig.6 Interface ring shear tests on London clay.

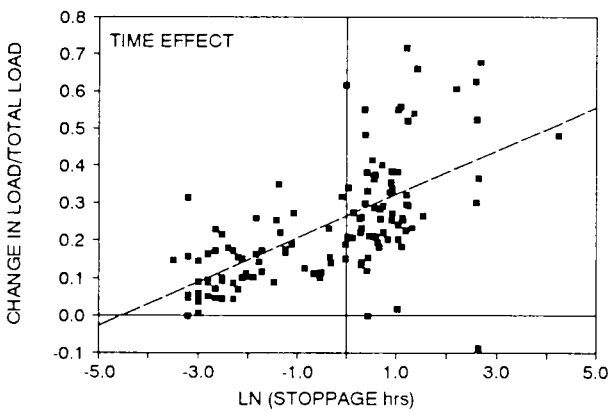


Fig.7 Time factor for scheme 3.

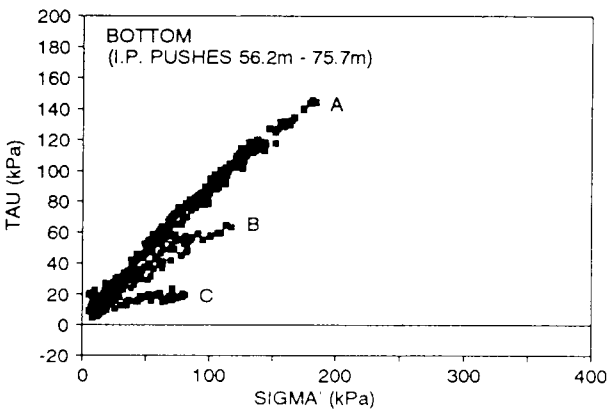


Fig.8 Effective skin friction during lubrication of scheme 4.

change the basic shearing mechanism of the soil from pure sliding to sliding with some turbulence of the platy clay particles combined with a possible transient (viscous) effect. It will be noted that although the global response of the pipejack of scheme 3 seems to match the laboratory trends, the shear mechanism may be further complicated by both the radial and shear stresses varying during pushes.

The data from all the stoppages have been compiled in Figure 7. Although there is some variation which is probably a function of changing face resistance, the general trend fits the relationship:

$$\frac{\delta P}{P} = 0.259 + 0.058 \ln(t)$$

and demonstrates that a major part of the increase occurs within minutes of stopping.

5.3 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 8 provides some indication of the local variability of its effectiveness. The data is 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 unlubricated 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 data from scheme 5 suggests that the lubricant around the pipe annulus was pressurised, supporting the surrounding ground and causing pipeline buoyancy and low interface frictional stresses.

6. CONCLUSIONS

The field data clearly demonstrates that large localised radial, shear and pore pressures can be generated during jacking. Peak values typically occur over short 1m to 2m lengths and generally correspond to positions of maximum angular misalignment. Critical pressure distributions, which are highly non symmetrical around the pipe circumference, interact with non-uniform pipe end load transfer, Norris & Milligan (1992), requiring consideration in the pipe design process.

A partially drained frictional material response is exhibited at the pipe interface in both cohesive and non-cohesive ground. There is limited evidence that the material within the rupture zone approaches a critical state condition. 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. Further field measurements are required to establish the suitability of using Marston trench loading conditions which are the basis of UK design.

If stoppages occur in cohesive ground then a greater force is needed to restart the process than would have been required to maintain continuity. The "time factor" effect is pronounced in the high plasticity clay and may be a function of jacking speed rather than increases in the radial ground pressure during the stoppage.

Lubrication is an effective method of reducing frictional resistance but care is needed in matching lubricant properties to existing ground conditions, providing sufficient pumping capacity and adopting suitable injection sequences.

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SUMMARY

The Oxford pipe jacking research project has now been in progress for over seven years, supported by a mixture of government and industrial funding. Two stages have been completed, involving laboratory studies followed by monitoring of five full scale pipe jacks on active sites in a variety of ground conditions. Some typical results from the field monitoring are presented, pertaining to pipeline alignment, tunnel stability, pipe/soil interface behaviour, total jacking forces, pipe stresses and ground movements. The full results have been reported to the sponsoring organisations and will be incorporated into a new code of practice for pipe jacking. Research is now in progress on two further stages, one continuing the field monitoring, the other using numerical modelling of pipe joints to evaluate possible improved pipe joint details.

INTRODUCTION

The research project in pipe jacking at Oxford University has now been in progress for over seven years. It originally grew out of an initiative by the Pipe Jacking Association (PJA) and the Construction Industry Research and Information Association (CIRIA), and the overall objectives follow the recommendations in the report by Craig (1):-

- (a) prediction of friction loads in different ground conditions;
- (b) load/deflection characteristics of the joints with different packing materials;
- (c) the effect of cyclic loading on the pipes at intermediate jacking stations;
- (d) the effect of lubricants in reducing friction along a pipe;
- (e) the development of a site investigation test suitable for the prediction of frictional forces.

The measurement and prediction of ground movements associated with pipe jacking has been added to this list of objectives.

Phase 1 of the project, from 1986 to 1989, involved laboratory testing of model concrete pipes and joint packing materials, with Kevin Ripley as Research Assistant (2). Phase 2, from 1989 to 1992, involved the monitoring of full-scale pipe jacks on five active sites, with Paul Norris as Research Assistant. Phase 3 started in November 1992, and Phase 4 in May 1993; the former continues the site monitoring, while the latter comprises finite element analysis and improved design of pipe joints. Research Assistants are Mark Marshall and Jian-Qing Zhou respectively. The whole project is supervised by the first author, with the assistance of a management committee drawn from the various sponsoring organisations.

An overview of the procedures and progress of Phase 2 was given at the First International Conference on Pipe Jacking and Microtunnelling (3); details of the instrumentation used (4), and preliminary results of the monitoring (5,6), have been presented at other specialist conferences. Further details of the instrumentation and complete details of the site monitoring work are available in a report and thesis by Norris (7,8). The purpose of this paper is to give an indication of the quality and extent of the results of Stage 2, and to explain the purpose of Stages 3 and 4. The main practical lessons of Stages 1 and 2 have been encapsulated in a summary report for the sponsoring organisations, and will be incorporated into the new Code of Practice (9) and other documents to be produced by the PJA.

RESULTS OF RESEARCH - STAGE 2

Funding and Progress

The total budget for Phase 2 was approximately £220,000, divided between the Science and Engineering Research Council (SERC), the PJA, and five water utility companies - Northumbrian, North-West, Severn-Trent, Thames and Yorkshire. Additional costs amounting to about £15000 per site were borne mainly by the water companies. Work was completed within budget, even though the original three-year programme was extended by six months due to delays in obtaining suitable sites and to allow fuller analysis of the large quantity of data collected. Details of the five sites monitored are given in Table 1, and a schematic of the instrumentation employed in Figure 1. At the end of the monitoring phase most of the instruments were still in good working order and will be used in Stage 3. Overall the project was remarkably successful in obtaining good data from a difficult working environment, and represents an outstanding example of co-operation between academia, clients, designers, contractors and pipe suppliers for the common benefit of the industry.

Pipeline Alignment

In all pipe jacks small angular deviations occur between pipes as a result of the tunnelling machine or shield veering off line, for a variety of reasons, and the steering corrections made to maintain correct line and level. These angular deviations have two major effects; they tend to increase the local contact stresses between pipes and the ground and cause serious stress concentrations at the joints between pipes. The former increases the overall jacking load and the latter reduces the ability of the pipe joints to transmit the jacking force without damage to the pipes. Normal specifications giving allowable deviations from line and level are an indirect and ineffective way of attempting to control the angular misalignments between successive pipes.

Pipe joint angles were measured indirectly from tunnel line and level surveys, and more accurately and directly by the pipe joint movement indicators. A typical set of data is shown in Figure 2. From the five sites it was found that typical joint angles on well-controlled drives were generally less than 0.3° , but occasionally reached 0.5° , and as much as 0.8° when steering corrections were over-rapid. The correlation between local contact stresses and pipeline alignment is also apparent in this figure.

The site monitoring showed up a tendency for alignment control to be poorest at the start of drives. Successive tunnel

surveys and joint angle measurements also showed that the pipeline alignment did not improve as jacking continued, nor did the joint angles change significantly between the loaded and unloaded conditions. Thus initial misalignments persisted and those at the start of a drive later coincided with the pipes carrying the greatest jacking force.

Pipelacking contractors' experience suggests that in some conditions, particularly in soft soils, some straightening of the pipeline may occur as jacking proceeds; it is hoped to monitor a drive in such conditions in the current stage of the research.

Tunnel Stability

The tendency of the tunnel face or bore to collapse in cohesionless or soft cohesive soils can be assessed by established soil mechanics calculations (10). On three of the monitored drives, through stiff clays or soft rock, the tunnel was basically self-stable, while the fourth through silty sand above the water table was stable in the short term due to capillary suction. In such cases the pipes will mainly be sliding along the base of the tunnel bore, with the least resistance to jacking, except where contact between pipes and ground occurs due to tunnel alignment as discussed above. Contact stresses were then measured mainly at the base of the pipe.

On scheme 3, although the tunnel bore was theoretically stable, the ground closed onto the pipe due to the elastic relief of the high stresses in the heavily overconsolidated London clay. Simple elastic analyses (11) confirm that the inward movement of the tunnel walls would be sufficient to close the initial overbreak from the shield. Unfortunately only a few contact stress measurements were obtained before the stress cells were damaged or displaced by the unexpectedly high pressures. It is hoped to monitor a similar scheme in Stage 3 with instrumentation of greater capacity. On scheme 5, through sand and gravel below the water table, the tunnel bore was stabilised by the use of bentonite slurry under pressure. Prior to the use of slurry, the ground collapsed immediately onto the pipe as expected.

Pipe/soil Interface Behaviour

The contact stress cells incorporated into the pipe walls are able to measure both the radial (normal) and shear stress at the interface between the pipe and the ground. Values of the interface friction or adhesion are therefore obtained directly. It was found that radial stresses varied very widely, no doubt due to the irregular contact between pipe and ground, but that surprisingly clear relations between normal and shear stresses were obtained; results from schemes 2 and 4 are shown for example in Figure 3. These plots are in terms of total stresses; plots in terms of effective stresses, using the pore pressures measured adjacent to the stress cells, showed greater scatter but little difference in the overall relations between shear and normal stresses, even in high-plasticity clay. The pore pressures may not be reliable, partly because of the difficulty of measuring pore pressures when contact with the soil is intermittent and the soil surface may be unsaturated, and partly because the pore pressure and stress measurements were not made at precisely the same place. The latter will be rectified in Stage 3.

For the first four sites the basic shear stress/total normal stress relation could be interpreted as being frictional in nature, with the friction angles as given in Table 2. In two cases the behaviour seemed to be tending more towards undrained adhesion: in scheme 1 when the stiff glacial clay was wetted by heavy rain and a burst water main; and in scheme 3 in London clay when the radial stresses became large. This possible change in mechanism in softer clays or at higher normal stresses will be investigated further in Stage 3.

Figure 3 also shows the effects of the lubrication used in the later stages of scheme 4. When freshly injected it considerably

reduced the interface friction angle (line C); however after a short time the effect was greatly reduced (line B), and eventually disappeared altogether, so that the behaviour (line A) was no different from that prior to lubrication. On scheme 5, full lubrication was adopted - the overbreak annulus was kept completely full of slurry under sufficient pressure to stabilise the tunnel bore. The pipeline then became buoyant within the slurry, and a layer of lubricant was maintained at all times between pipe and soil. The contact stresses between soil and ground were negligible, the stress and pore pressure cells both recorded the slurry pressure, and the surface shear stresses were very small.

Total Jacking Forces

A typical jacking force record, as measured by the load cells on the main jack rams, is shown in Figure 4. The intercept at zero length of drive represents the face resistance at the shield, while the gradient of the line obviously gives an average measure of the frictional resistance along the pipeline. Face loads were found to vary between 100 and 1200kN, being large for the slurry tunnelling machine and when the shield was used to trim the excavation in strong cohesive soils, but small in stable cohesive soils when the face was excavated to the full diameter of the shield.

The interpreted average frictional stresses were generally near or below the lower end of the typical ranges quoted by Craig (1), probably reflecting the good directional control on the monitored sites. However it is somewhat misleading to talk in terms of average stresses around the pipes when in fact they are only in contact with the ground at the base of the tunnel bore. In such cases it was found that the jacking resistance could be calculated approximately from the selfweight of the pipes and the friction angles given in Table 2, adding about 25% for the effects of typical misalignments.

In the cohesionless soil of scheme 5, before the use of bentonite lubrication, the jacking resistance could be calculated by assuming that contact stresses around the pipes corresponded to Terzaghi's "trench" analysis (12). Once lubrication was in use, the jacking resistance dropped by a factor of 10 to a mean shear stress over the surface area of the pipes of 2.2 kN/m²

Pipe Stresses

The joint angle measurements coupled with the joint pressure cell measurements allowed the load-paths through the instrumented pipe to be determined. These were found to vary from being along one edge of the pipe to almost "diagonal", that is from top to bottom or one side to the other over the length of the pipe. However the latter generally occurred at points of contraflexure in the pipeline (see Figure 5a, pipe No. 2) and coincided with relatively small misalignment angles at the joints. The former occurred at the apex of curves in the pipeline (Figure 5b, pipe No.3), and could coincide with relatively large joint angles (Figure 2).

No sign of cracking was observed in the pipe barrels in any of the monitored schemes; this was confirmed by the strains measured by the tube extensometers, which were always within the elastic range for the concrete. Pipe barrels could therefore be designed simply as elastic stocky columns for the full range of possible load paths.

In contrast, the joint instruments confirmed that contact stresses at the joints were highly concentrated for joint angles as small as 0.3° (see Figure 6). The Australian Concrete Pipe Association's design approach was found to give reasonable predictions of joint stress distributions, provided an appropriate value was used for the stiffness of the packing material. Using this approach it is possible to establish design curves giving allowable jacking loads on pipes for different maximum joint angles with different packing material types, widths and thicknesses (see Figure 7 for example). Note that even if a

relatively high local stress is allowed in the concrete the total jacking load is severely curtailed at a joint angle of 0.5° , and that considerable benefits are obtained by using thicker and softer packing. In scheme 5 the contractor's pipes, fitted with plywood packers, suffered some damage at points of sharp curvature and large jacking loads, while the instrumented pipe negotiated the same conditions without damage, having been fitted with fibreboard packers.

Ground Movements

Surface settlements were measured on scheme 1, and were very small despite the low cover depth. An instrument array of three inclinometer tubes with magnetic settlement plates was used in scheme 5 to detect both lateral and vertical ground movements. The passage of the pressurised tunnelling machine caused upward movements of up to 7mm in the medium dense to loose gravelly sand above the face, with negligible movements at the surface. Subsequent settlements on the tunnel centreline were about 10 to 15mm and were slightly greater than predicted by the empirical methods in common use (13) if the ground loss was taken to be equal to the overbreak volume. This seemed to be due to a general settlement of some 4mm over quite a wide area, possibly caused by soil densification due to the machine vibrations.

FURTHER RESEARCH - STAGES 3 AND 4

Stage 3 - Site Monitoring

Stage 3 of the research work is a continuation of the site monitoring, but with a somewhat different emphasis from Stage 2. The structural performance of pipes and pipe joints is now reasonably well understood, but considerable uncertainties still exist over the contact stresses between pipes and soil in some ground conditions, the effects of pipeline misalignments on these stresses, and hence the best methods of calculating expected jacking loads. The instrumented pipe will be fitted with up to 12 contact stress cells with pore pressure probes in the cell face, so that much more information can be obtained on the stress distributions along and around the pipe. Pipe joint instrumentation will probably be restricted to the joint at one end of the pipe, but measurements may also be made at the thrust ring and at an interjack station to determine the pipe end stresses in these situations, which may be onerous.

More measurements will also be made of the ground movements in different ground conditions. Electro-levels will probably be used, in addition to inclinometers and settlement plates, as they can give continuous readings during the passage of the tunnelling machine or shield. The precise schemes for ground and pipe instrumentation are being finalised, and suitable sites sought, with the intention of restarting site work towards the end of the year. Particular emphasis is placed on obtaining a site in London or similar clay, as only limited data was obtained from such conditions in Stage 2, and in soft clay, as no such scheme has been monitored to date.

Stage 4 - Pipe Joint Design

The work to date, and many years of practical experience, have shown that the pipe joints are the weakest point in the system for the transmission of the jacking loads along the pipe line. The steel-banded joint, now commonly used whenever significant jacking loads are expected, is undoubtedly better than the in-wall joint, while correct use of good packing material of adequate thickness also increases the joint load capacity. However there is every possibility that quite minor changes to joint geometry, use of more sophisticated packing material, modified reinforcement arrangements or localised prestressing at the joint could significantly enhance joint load capacity. Experimental evaluation of all such possibilities would be very expensive and time-consuming; the approach in Stage 4 will therefore be to model the joint numerically using finite elements and then use the numerical model to try out variations in joint design. The most promising modifications

will then be constructed and tested, first in a simplified form in the laboratory and then under field conditions.

CONCLUSIONS

Some seven years of research work on pipe jacking, involving both laboratory testing and field monitoring at full scale on active construction sites, has produced much valuable information. This includes the measurement of pipe line alignments and resulting joint misalignment angles; pipe soil interface behaviour; the effectiveness of slurry for tunnel stabilisation and pipe lubrication; total jacking forces in various ground conditions; pipe stresses, packing material behaviour and joint stresses in misaligned pipe joints; and ground movements associated with pipe jacking. In some cases the results have provided scientific backing to previously empirical knowledge, in others they have given rise to serious debate within the industry. In addition to the purely technical advances made, the research has provided a valuable forum for co-operation between client and contracting organisations, has helped to raise the level of technical awareness in the industry, and stimulated the interest of designers in pipe jacking as a means of constructing tunnels both small and large. The continued backing of SERC, the PJA and the five water companies is allowing two further stages of work to proceed, to expand the database of results from field monitoring and to investigate possible improvements in pipe joint design.

ACKNOWLEDGEMENTS

The success of this research depends on the co-operation and involvement of many organisations and persons too numerous to mention individually. The authors wish to acknowledge and thank all the sponsors of the project for their generous support; the members of the management committee for their interest, encouragement and assistance; the Water Companies and their agents and consultants for providing sites and other assistance with the work; the pipe manufacturers for their co-operation and for absorbing some of the additional costs of the specially prepared pipes; the contractors on each site for their patience and help with the research work; and various staff, technicians and students in the Engineering Department at Oxford who have helped with the project.

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	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	Barbale	Barbale	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 1 Details of schemes monitored

Scheme No.	Soil type	Friction angle (°)
1	Glacial clay	19
2	Mudstone	17
3	London clay	12.7
4	Silty sand	38
4	Sandy silt	30

Table 2 Measured local interface friction angles

Figure 1 Schematic of instrumentation

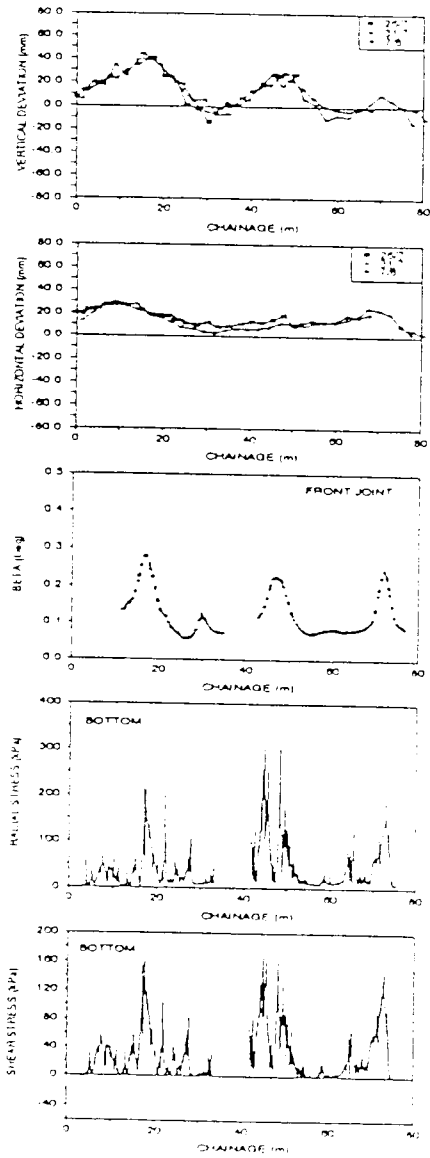
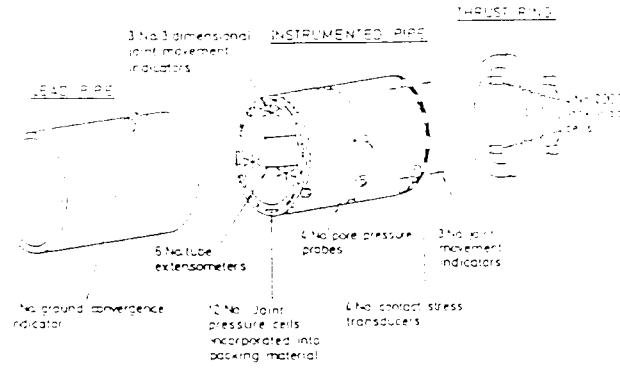


Figure 2 Tunnel alignment data and local interface stresses (scheme 4)

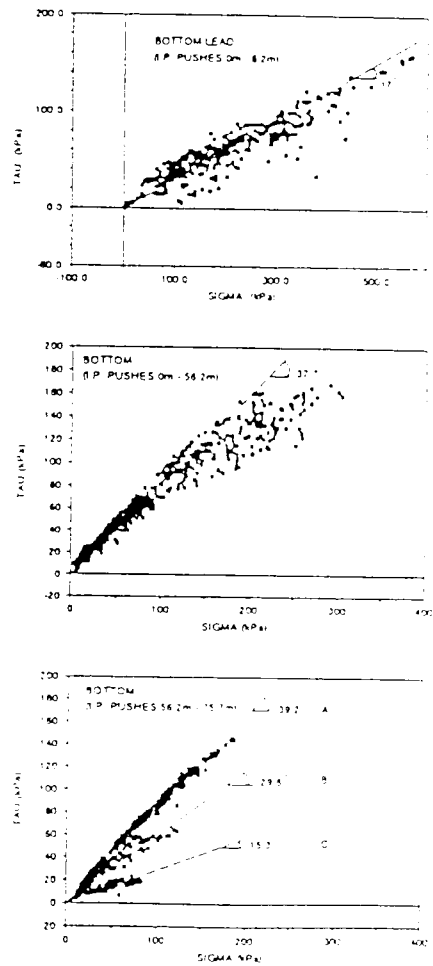


Figure 3 Shear stress/radial stress relations:
 - from scheme 2 (top)
 - from scheme 4 prior to lubrication (middle)
 - from scheme 4 after lubrication (bottom)

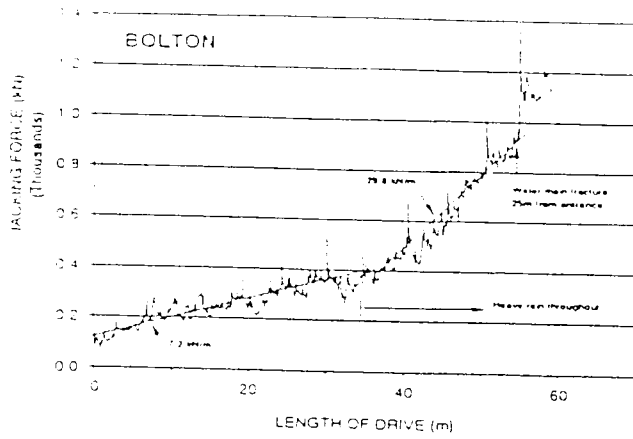


Figure 4 Jacking record for scheme 1

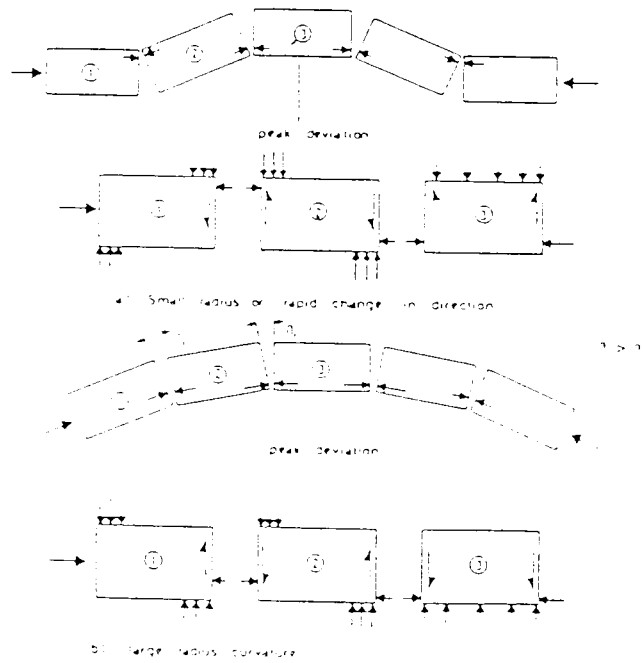


Figure 5 Theoretical misalignment forces

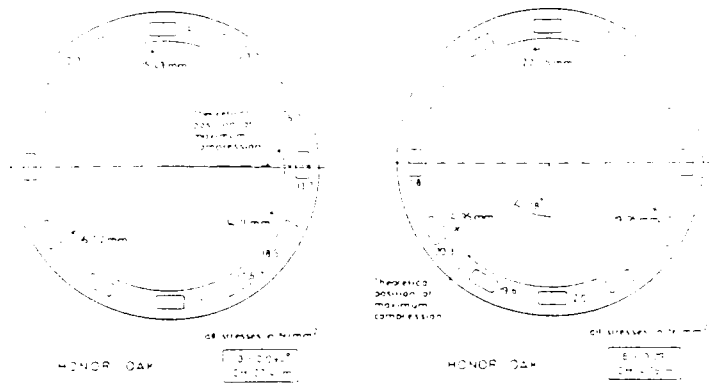


Figure 6 Joint pressure distributions for different joint angles β ; scheme 3

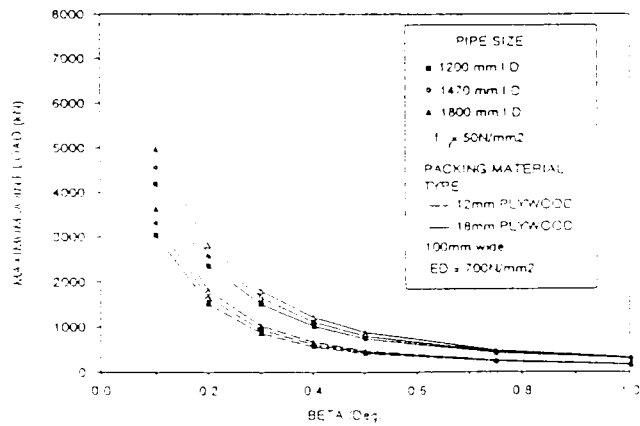
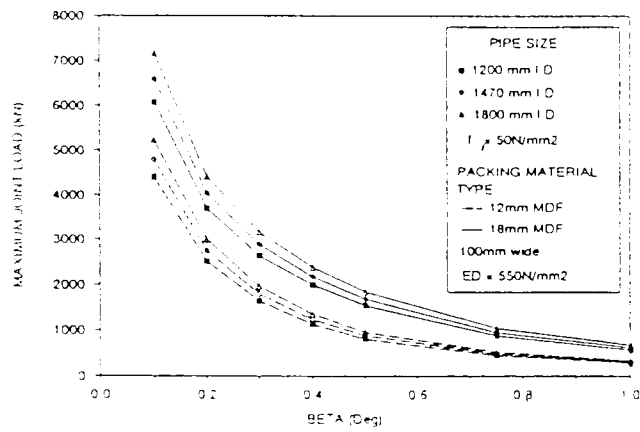


Figure 7 Permissible pipe end loading at various angular misalignments:
 - 100mm wide medium density fibreboard (upper)
 - 100mm wide exterior grade plywood (lower)

Practical examinations

A decade's research at Oxford University on pipe jacking forms the core of a new best practice guide. Dr George Milligan talks to *Ground Engineering* about the research and its main findings

For over 10 years Oxford University has been carrying out research on pipejacking, through an innovative joint industry and public funded project. The work forms the core of the recently published 'Guide to best practice for pipe jacking and micro tunnelling' and has provided industry with what is proving to be an invaluable assessment and explanation of pipe stresses and pipe-soil interaction.

The research was initiated by the UK trade organised Pipe Jacking Association. When the PJA approached Oxford's Department of Engineering Science, Milligan confesses that neither he nor the department's professor, the late Peter Wroth, had much knowledge of pipejacking; but this was a time 'when the term trenchless technology had not been coined – or was certainly not in use,' says Milligan.

From the start the PJA was keen for the work to be based around a live project. However Milligan and his first research assistant Kevin Ripley – an engineer on secondment from Delta (the contracting arm of tunnelling equipment manufacturer Decon whose chairman Arthur Moss is one of the driving forces of the PJA, and the tunnelling industry in general, decided that a preliminary laboratory operation must come first. There followed an initial three year project modelling in a 3D test chamber (which allows in situ ground stresses to be reproduced in the laboratory).

This work, co-sponsored by the Concrete Pipe Association, provided useful and immediately practical information on packer materials (a compressible material, historically plywood, used between the joints of pipes). The laboratory work found that simply by using chipboard or plywood in place of plywood as a packer material the pipes could accommodate greater deviations off line before damage occurred.

However the work was not entirely

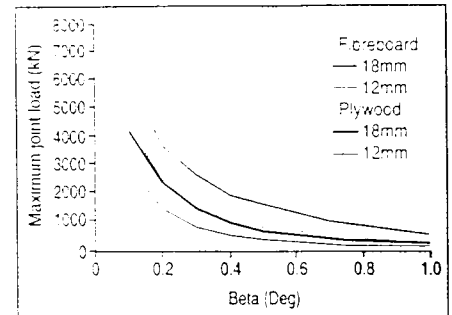
successful – boundary conditions and end effects in the laboratory model were difficult to understand, and the work proved the need to instrument on site.

For this second phase, more industry partners became involved. In addition to public funding through SERC (now EPSRC) and the PJA, five water authorities (this was before water industry privatisation), gave financial support, provided access to their sites where pipejacking work was under way, and covered additional site costs resulting from the research.

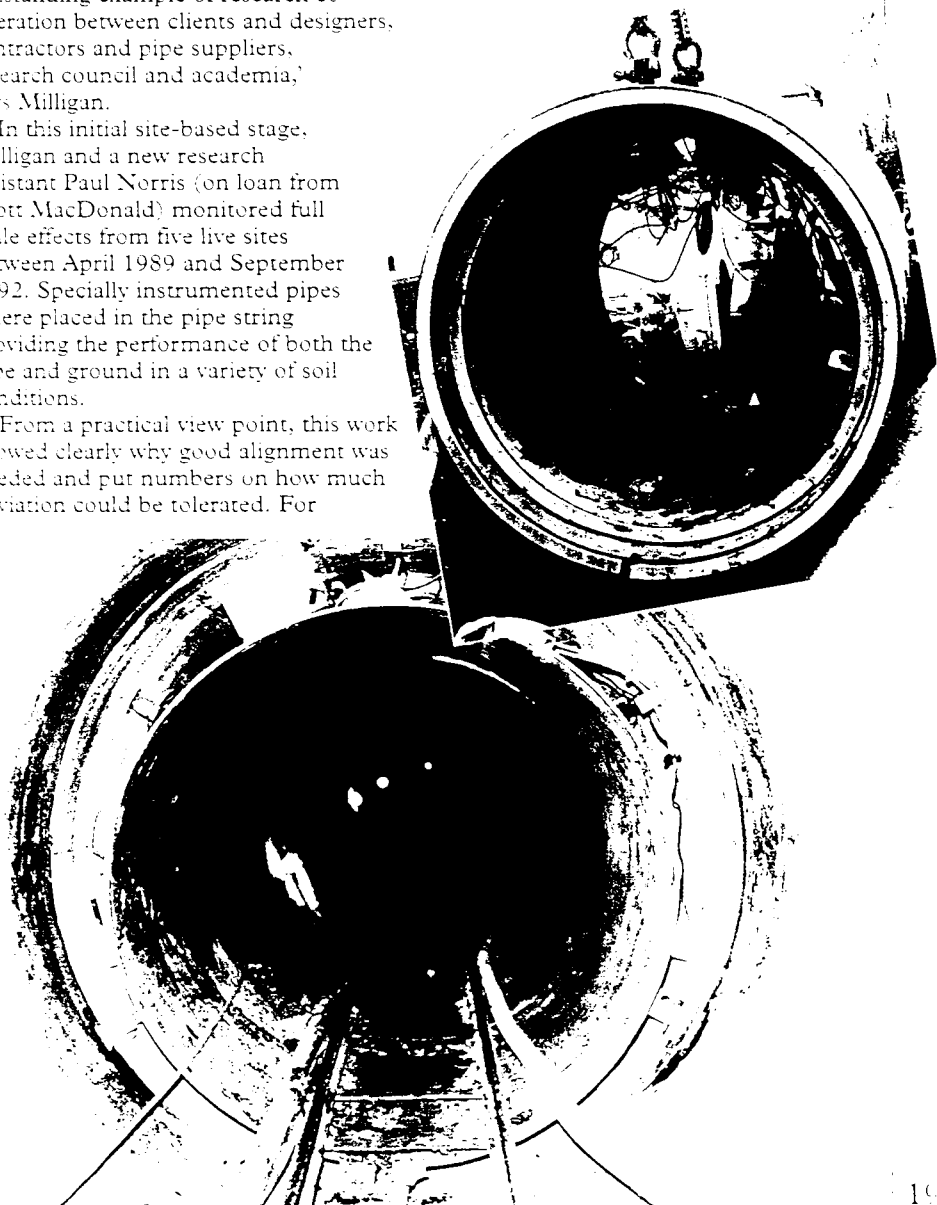
'The project represented an outstanding example of research co-operation between clients and designers, contractors and pipe suppliers, research council and academia,' says Milligan.

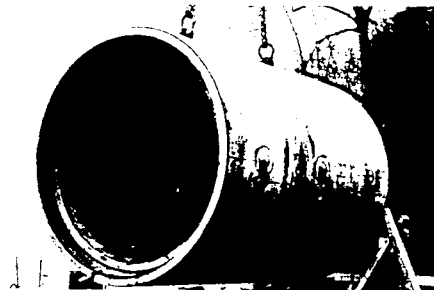
In this initial site-based stage, Milligan and a new research assistant Paul Norris (on loan from Mott MacDonald) monitored full scale effects from five live sites between April 1989 and September 1992. Specially instrumented pipes were placed in the pipe string providing the performance of both the pipe and ground in a variety of soil conditions.

From a practical view point, this work showed clearly why good alignment was needed and put numbers on how much deviation could be tolerated. For



Permissible pipe end loading at various angular misalignments (Beta) for different packer materials. Example given for 1.2mID pipe with 100mm wide packer.





Instrumented pipe. The three pressure cells can be seen clearly.

example says Milligan, the work highlighted shortcomings in working to a specification based around line and level performance. 'The key factor - provided hydraulic considerations are satisfied - is not the overall alignment, but the deviation between one pipe and the next. So if the drive starts to go off line it should be brought back as gradually as possible. In a way a lot of what was found was common sense, but by getting in and taking measurements we were able to put sensible numbers to it,' continues Milligan.

However, other aspects of the work dispelled some commonly accepted industry practices. Most pipejacked tunnels wiggle a bit in the first few metres, and tunnellers will have you believe that this is not a problem. The alignment they say, straightens out as loads increase. From the Oxford research it was clear that if one pipe kinks, then the rest follow. 'Extra care is needed at the start of a drive, and contractors need to avoid the pressure to get a few pipes in the ground as quickly as possible at the start of the drive' suggests Milligan.

Geotechnical aspects of the work were equally surprising. Cambridge stress transducer pressure cells, capable of measuring both normal stress and shear stress, were built in flush with the external face of the pipe wall, and load cells placed on the jacking rams, giving an accurate record of the total jacking resistance.

The pipe soil interaction was observed to be essentially frictional in all cases, even when going through clayey materials. 'We had expected - at least in London Clay - to get undrained behaviour,' says Milligan.

This is as yet not fully explained, and what may at first appear to be a simple problem is controlled by some fairly complicated mechanics. An intimate

contact is not made between the pipe and ground. During jacking the pipe passes over an irregular cut surface and as the jacking load increases these asperities are flattened out.

There is evidence that at very high normal stresses, something approaching undrained behaviour occurs and phase three of the research (currently under way with Mark Marshall as research assistant) includes a site in much softer clay which confirms undrained behaviour as an upper bound to behaviour.

Emphasis in the current phase is on the geotechnical aspects of pipe jacking and in particular ground movements. The instrumented pipe is now equipped with 12 stress cells to get a better understanding of the distribution of stresses, while the ground surrounding the pipejacks is being monitored using an array of instrumentation including inclinometers, spade cells and electrolevels.

This work is focusing on identifying and evaluating the main factors which are most likely to influence soil-pipe interaction, such as the amount of overcut (the space between the outside dimensions cut at the face and the diameter of the pipe), soil type and the use or not of pipe lubrication.

For example if a drive is made through stable ground with sufficient overcut the jacked pipes will only come into contact with the soil along the bottom of the pipe. In this case the resistance to sliding of the pipes is related simply to the self weight of the pipes. If, however, the drive is through soft clay the overbreak may not stand intact, and the ground may pressurise the pipe around the whole of its circumference. Equally, if the clay is heavily overconsolidated it will relax upon excavation, possibly filling the overcut and 'squeezing' (as tunnellers put it) the ground.

In granular soils, below the water

table, the excavation will not stand unsupported, and so bentonite is often pumped into the overbreak. In these cases the pipe tends to float in an annulus of bentonite and extremely low contact stresses result. The down side of using bentonite is that it is very difficult to then displace the bentonite to allow satisfactory grouting of the annulus. And in heavily overconsolidated materials, water in the bentonite slurry may result in rapid rehydration, expansion and softening of clayey ground.

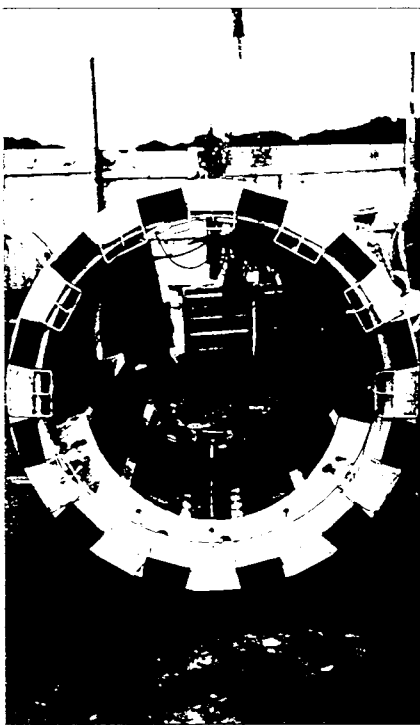
Pipe-soil interaction work is still in its early stages, says Milligan, and a current shortage of pipejacking contracts is making it difficult to find suitable sites to progress this work. Milligan was hoping to get access to a chalk site in Kent in the new year, but the contract may be put on hold, robbing for the time being the research database.

Main geotechnical conclusions from pipe jacking research at Oxford University

- Contact stresses between the pipe and the ground depend on the stability of the tunnel bore, the initial stresses in the ground and the stiffness of the soil. In cohesionless materials they are well predicted by the Terzaghi model.
- Pipe-soil interface sliding is frictional in nature, even in cohesive soils. The critical state friction angle gives reasonably good predictions, except in high plasticity clays where residual friction strength better first approximation. The undrained strength however provides an upper bound in cohesive soils.
- In stable tunnel bores the resistance to sliding of the pipes is related simply to the self weight of the pipes.
- Effective lubrication requires complete filling of the overbreak, and in cohesionless soils sufficient pressure to maintain the stability of the tunnel bore. In stiff clays, aqueous slurries (ie bentonite) may accelerate swelling of the clay leading to increased contact stresses between pipe and ground. Polymer based lubricants may be better and newly developed stabilising foams may have good lubrication properties.
- Face loads are likely to be relatively high with slurry or earth pressure balance tunnelling machines and care is needed to ensure that the face is not overloaded leading to excessive ground movements.
- *Guide to best practice for pipe jacking and microtunnelling, is available from the Pipe Jacking Association, 56 Britton Street, London, EC1M 6NA, tel: 0171-252 2268; fax: 0171-251 1939. Price £35 (plus £3 p+p outside UK).*

PIPEJACKING RESEARCH

The basic principles of pipejacking is a method of producing a tunnel lining up from pipe sections laid out in practice to involve complex interactions between the tunnelling equipment, the jacking machine or shield, main bearings and interjack stations, the pipes and the ground. For instance the contact stresses between the pipes and the ground, along with the characteristics of the soil, determine the resistance to jacking and hence the equipment required, the method of resisting the jacking force and the loads to be



Typical instrumented pipe prior to installation.



Instrumented pipe installed in tunnel.

	1	2	3	4	5
Date	August 1990	January 1991	March 1991	July 1991	December 1991
Location	Bolton, Lancs	Gateshead, Tyne-side	Honer 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	Barbale	Barbale	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

carried by the pipes during installation. But in fact these stresses are related to the details of construction procedures, such as the method of excavation, the amount of overbreak, the amount

Table 1. Details of projects monitored.

of the drive, and the use of slurries to support and lubricate the tunnel bore.

For the past seven years a research project at Oxford University has explored these various interactions, first by using model pipes in laboratory investigations and then by monitoring actual construction work on site with specially instrumented full scale pipes. The work has been directed and supervised by the author. Dr Kevin Ripley (Delta Civil Engineering) was the Research Assistant for the first stage, and Dr Paul Norris for the second, both registered as research students at Oxford and obtained their degrees from their work on these projects. Financial support for the work has come from a strong alliance of public and industrial money, with support from the Science and Engineering Research Council, the Pipe Jacking Association, the Concrete

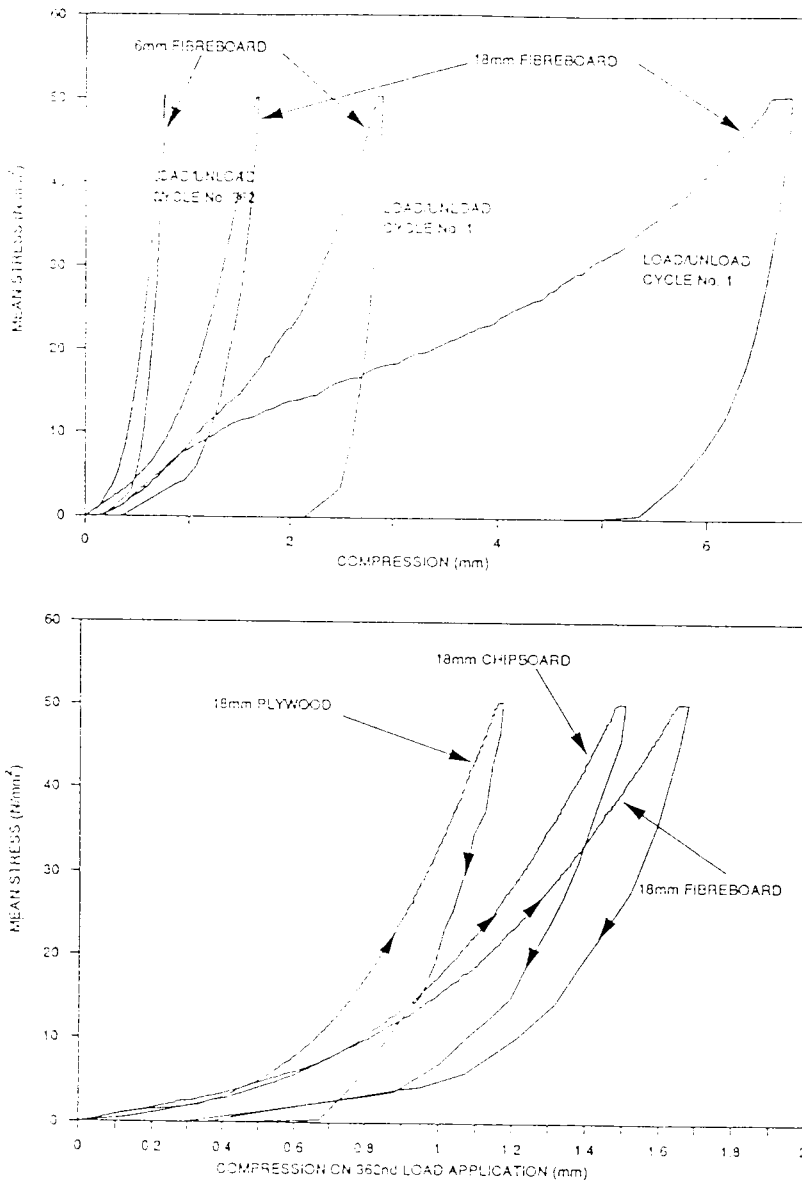


Fig.1 Typical behaviour of packing material.

Pipe Association and, for the second stage, five water utility companies - Northumbrian, North-West, Severn-Trent, Thames and Yorkshire. In addition to direct financial support, the water companies provided suitable sites and carried some of the additional site costs resulting from the research work. Support in kind was also provided by pipe manufacturing companies and contractors involved with individual projects.

LABORATORY MODEL TESTS

The laboratory testing highlighted the importance of the stress concentrations that occur at the joints between pipes while being jacked due to the small angular deviations between successive pipes which inevitably arise as a result of manufacturing tolerances.

corrections to line and level, and even deliberate curvature of the pipeline. Initial failures in pipes nearly always appeared as local crushing or spalling of the joint; the latter may not be detectable in a finished pipeline as the spalling will be on the outside of the pipe, but may lead to problems in the long term if pipe reinforcement is exposed or joint sealing details disrupted. It is therefore very important to be able to design the joint correctly.

The use of packing material in the joints was shown to be essential for any pipeline sustaining significant jacking forces, and substantial differences in the effectiveness of different commonly-used materials demonstrated. Medium density fibreboard is better than chipboard which in turn is better than plywood or solid timber, and the thicker the material the better. However even with packers the stress

concentrations can be severe and it was recommended that maximum jacking loads should be related to realistic local concrete stresses and joint misalignment angles rather than based on low nominal stresses over the full cross-sectional area of the pipe. In doing so it must be appreciated that the packing material behaviour is quite different after several cycles of loading from the behaviour under initial loading (Figure 1).

SITE MONITORING OF CONSTRUCTION

The site monitoring work involved incorporating into active pipe jacks an instrumented pipe which allowed measurement of joint misalignment angles, joint stress distributions, overall longitudinal pipe strains, and radial stresses, shear stresses and pore pressures at the interface between soil and pipe. Pipeline movement and total jacking forces were measured at the jacking pit, and all measurements were related to site activities, periodic surveys of line and level of the full pipeline, face logs and soil sampling and testing (Figure 3). Five schemes in total were monitored in this way (Table 1).

SIGNIFICANCE OF PIPELINE ALIGNMENT

The joint instrumentation validated at full scale the results of the model tests, and provided accurate information on the actual joint angles achievable in practice on well-controlled drives. The tunnel alignment was found in all cases to follow closely the path traced out by the tunnelling shield, with negligible tendency to straighten either under load or with the passage of successive pipes. Conventional line and level measurements may then be reinterpreted to provide reasonable estimates of the three-dimensional misalignment angles. To allow safe transmission of significant jacking loads, the maximum joint angles should not exceed 0.5° or better 0.3° (Figure 2). Current specifications based only on limits to deviations from line and level do not ensure this, and are counter-productive if they result in over-rapid corrections and hence large joint angles. The requirements for line and level could in most cases be relaxed to those required for hydraulic performance, for example, combined with specification clauses linking allowable jacking loads to maximum joint angles, pipe joint design, packing material characteristics, and concrete strength. These joint angles must of course include the effects of out-of-squareness of the pipes themselves, and it should be noted that pipes made to the British Standard limits of allowable tolerances could theoretically give joint angles of 0.3°, leaving practically no tolerance at all for pipeline construction. Fortunately it was found that the UK-manufactured pipes used on the monitored sites were well within tolerances and the effects of end squareness were negligible.

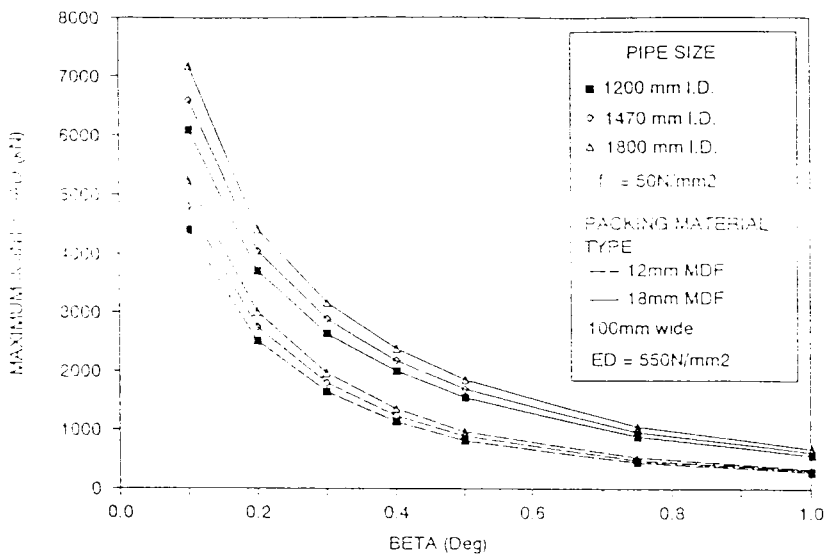


Fig. 2. Typical reductions in allowable jacking load with pipe joint angle.

INDUCED PIPE LOADING

Pipe loadings were generally most severe when the pipe was loaded essentially along one edge, with the areas of concentrated stress in the same location at each end of the pipe. Loading conditions approaching diagonal loading, for example from the top of the pipe at one end to the bottom at the other, were observed, but only with relatively small joint angles at points of contact/burial in the pipeline. In all cases the longitudinal strains within the main barrel of the pipe were small and stresses could be calculated from simple elastic "stocky column" theory; only nominal longitudinal reinforcement is needed in the central part of reinforced concrete pipes for normal jacking loads.

JACKING RESISTANCE

The total jacking force at the jacking pit (or jacking jack) is the combination of the load at the

tunnel face on the shield or tunnelling machine and the integrated frictional resistance along the ever-increasing length of pipeline. The face resistance will be relatively high with a slurry or earth pressure balance tunnelling machine, or when the shield is used to trim the excavation in hand-dig drives in strong soils, but may be very low in stable soils if the face is slightly over-excavated. With tunnelling machines excessive face pressure may cause local heave above and ahead of the machine in soft clays or loose granular soils.

The frictional resistance along the pipe depends on the characteristics of the soil and the contact stresses between pipe and soil, which in turn depend on the stability of the tunnel bore, the initial stresses in the ground, and the stiffness of the soil. The pipe-soil interface sliding behaviour was observed to be essential-

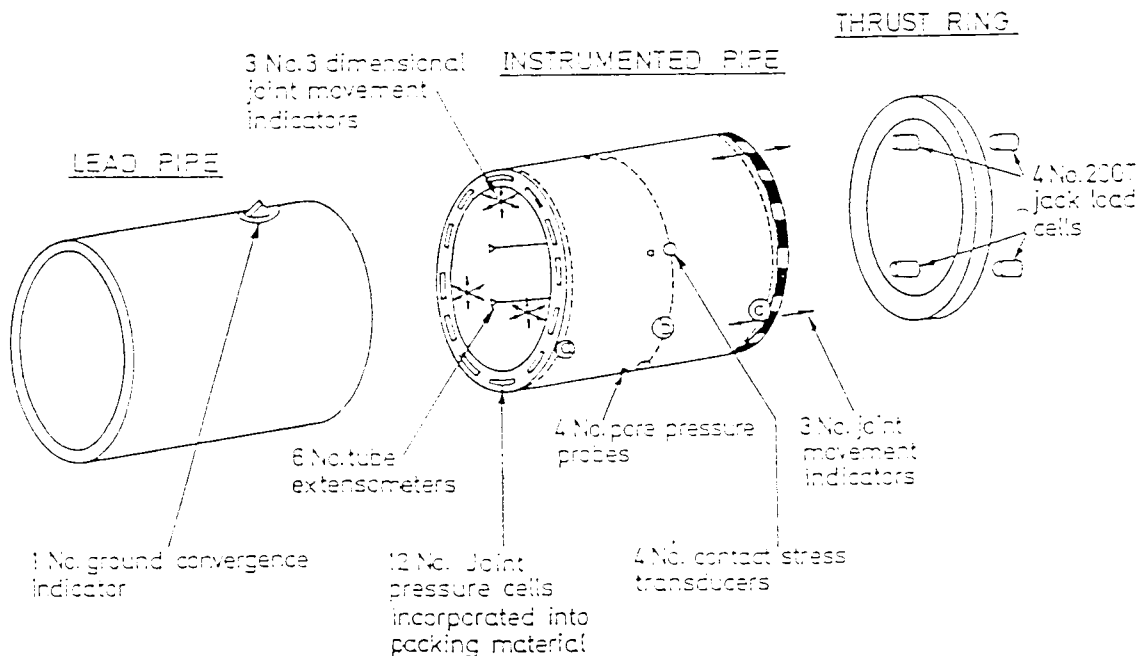
ly frictional in nature, even in cohesive soils, although the undrained strength provided an upper bound. The measured local friction angles varied from 12.7° in highly plastic over-consolidated London clay, to 38° in silty sand, with values of 17° to 20° in weathered mudstone and low-plasticity glacial clay.

When pipes are being jacked to a well-controlled alignment along a stable tunnel bore, with the overbreak remaining open, the resistance to sliding was found to be related to the self-weight of the pipes and hence quite low. Resistance was much higher in the London clay, despite the low friction angle, because the ground squeezed on to the pipe and combined with stress increases due to steering corrections to produce extremely high local contact stresses. Pipe misalignment generally causes the pipes to be forced against the side wall of the tunnel, increasing local contact stresses and hence frictional resistance. The mechanisms of this are complex and will be further investigated in the next stage of the research project now under way.

LUBRICATION

Bentonite slurries and various polymer materials are often used to lubricate the pipes to reduce jacking resistance. Monitoring of one pipeline in gravelly sand beneath the water table, driven with a slurry TBM and with full lubrication, showed clearly that the primary function of the slurry in such a situation is to stabilise the tunnel bore and maintain an open overbreak. The pipes then become buoyant in the slurry and a lubricating layer is maintained around them, with negligible contact stresses between pipe and ground; the lubricant is then fully effective and resistance to jacking very small. When slurry

Fig. 3. Schematic view of instrumentation for site monitoring.



was not used, the soil collapsed onto the pipe and the (very much greater) resistance was well estimated from contact stresses between pipe and ground determined using the "trench" loading - originally proposed by Ternaghi, Shree which does not completely fill the overbreak and cannot support a supporting pressure in the trench - is more "less effective".

SITE CONTROL

It is clear from the research work to date, as well as from much practical experience, that maintaining accurate alignment of a pipe jack is essential. Even quite small angular misalignments between pipes greatly increase local frictional resistance to jacking and reduce the ability of the pipes to withstand the jacking forces. Should the drive go off line, due to changes in ground conditions or other factors, it is usually more important to bring it back on line gradually to minimise pipe joint angles than to attempt to keep to tight specified tolerances on line and level by rapid steering corrections. It is particularly important to check alignment carefully at the start of the drive; for various reasons this is often the hardest part of the drive to control well, but it is also the point at which the force in the pipeline will be largest as the jacked length increases, unless an interjack is employed. Careful monitoring of actual alignment should allow more rational decisions about when and if to deploy interjack stations, particularly if

combined with routine accurate measurement of jacking forces by load cells (as used in the research work) or calibrated pressure gauges on the jacks.

FURTHER RESEARCH

As noted above the research work is continuing, with the same sponsors and two new research assistants. Stage 3 of the project will involve further site monitoring with the emphasis moving away from pipe joint behaviour to more detailed measurements of pipe-soil interaction and the ground movements resulting from pipe jacking operations. Stage 4 aims to develop finite element computer models of the pipe joint to allow different joint materials and configurations to be assessed with the aim of producing a joint detail which will be less sensitive to small misalignment angles. All the work is aimed ultimately at improving the reliability and economy of pipe jacking, including microtunnelling, as a construction method.

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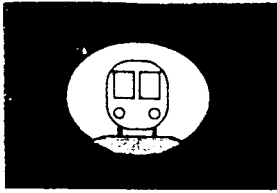
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TUNNELS OF SMALL DIAMETER USING THE PIPE JACKING TECHNIQUE

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1. INTRODUCTION

Pipe jacking is one of the techniques for installing tunnels and underground pipelines, ducts and culverts. For internal diameters between about 1.0 and 3.0 metres it provides an alternative to other methods of lining tunnels, such as segmental linings, or to the placing of large pipes or culverts in open cut. Pipe jacking adapted to internal diameters too small for man entry, typically from 900mm down to about 250mm, is usually referred to as microtunnelling and provides one of the trenchless technologies for installing pipelines without excavation of open trenches.

The technique involves pushing pipes through the ground using hydraulic jacks from a thrust pit to a reception pit, with excavation taking place inside a shield at the front of the line of pipes. Figure 1 shows a typical arrangement using hand excavation inside a simple shield. For larger tunnels the excavation is usually done by machine; Figures 2(a), (b) and (c) respectively show full-face tunnel boring machines, a cutter boom shield and a back-acter shield. In less stable ground, slurry or earth pressure balance tunnel boring machines may be used, while compressed air may be used to balance the ground water pressures. Microtunnelling systems are essentially miniaturised versions of pipe jacks using full-face tunnelling machines; excavated spoil is usually brought back to the thrust pit either by an auger system or by slurry. Some machines for use in soils containing cobbles and small boulders incorporate a crusher head to break down the large particles to a size which can be handled by the spoil removal system. Microtunnelling systems are often packaged so that they can be transported in a single standard container unit, which is sited close to or over the thrust pit and houses the control panels for the machine operator.

Directional control is usually achieved at the shield using steering jacks within an articulated tunnelling machine, or for simpler shields between the shield and the leading pipe. It is possible to pipe jack along large-radius curves as well as in a straight line. Some microtunnelling systems establish the line of drive using a directional drilling technique before over-reaming the hole and jacking in the pipe line. Control of alignment is either by conventional surveying techniques, or commonly now by using a laser to establish a reference line. With tunnelling and microtunnelling machines the laser usually shines on to a graduated target at the shield, giving immediate indication of errors in line and level; in a microtunnel the target is typically viewed via closed-circuit television on a screen in the control cabin.

At the thrust pit the jacking force is usually provided by two or four hydraulic rams, with their hydraulic systems interconnected so that the forces in all jacks are always equal. The ram loads are distributed onto the end of the pipe being jacked through a substantial thrust ring. Reaction to the ram loads is provided by a thrust wall at the back of the jacking pit, transferring the loads into the ground; in poor ground conditions more complex arrangements, for instance using piles, may be necessary. To keep the diameter of the thrust pit to a minimum, the jacking rig is sometimes set back into a short length of "back-shunt" tunnel. Once one pipe jack has been undertaken from a shaft, the rig may be turned round to drive a tunnel in the opposite direction, using the pipe line already constructed to provide the reaction.

The pipes used in pipe jacking may be manufactured in various materials, including reinforced or unreinforced concrete, steel, ductile iron, glass-reinforced plastic (grp), or vitrified clay. For large diameters, reinforced concrete is commonly used, though special composite pipes, such as of concrete with a grp lining, may be used in special circumstances. Standard pipes in large diameters are typically 2.5m long, but half- or other shorter-length pipes may be used for jacking around curves (to minimise the angular deviation at each joint) or when operating out of small-diameter thrust pits. The two types of pipe joint used in the U.K. are the rebated m-wall type and the butt joint with a steel or grp collar (see Figure 3); the latter is greatly to be preferred when substantial jacking forces are expected. Both incorporate sealing rings and should include compressible packing material to help distribute the jacking forces transmitted through the joint.

The maximum pipe length that can be jacked depends on many factors such as ground conditions, size and strength of pipes, type of shield and available reaction. About 80m for microtunnels and up to about 150m for large-diameter pipe jacks are commonly achieved. Lengths can be greatly increased by the use of lubrication with bentonite or polymer slurry materials, and by the incorporation of intermediate jacking stations (interjacks). These are steel cylinders containing a set of jacks which essentially create a telescopic joint within the pipeline; the interjack rams are operated to push the pipeline in front it, and the main rams then used to push the remaining pipe length and close up the interjack. By incorporation of a number of interjacks, the theoretical drive length is infinite, but muck-removal becomes increasingly less efficient as the pipe length increases, and economic lengths for machine drives are commonly of the order of 300-400m.

This opening section has served as a brief introduction to the techniques of pipe jacking and microtunnelling. Fuller details of methods and equipment may be found in a recent book by Thomson (1993), whilst the UK Pipe Jacking Association will shortly issue a new "Guide to Best Practice" (Cole 1993). The techniques have a number of potential advantages over other methods of installing pipes and small-diameter tunnels, particularly for the construction of sewerage systems, ducts for other services, culverts and underpasses. Benefits may be listed as:

- strong, rigid lining installed immediately after excavation
- completely enclosed operation possible for safe working in unstable or dangerous ground
- ground movements controlled
- high-quality internal finish without need for secondary lining
- in comparison with segmental linings, far fewer joints to allow potential leakage, safer working conditions
- in comparison with trenching, greatly reduced surface disruption, environmental disturbance, interference with other services, reinstatement costs etc.

However the full benefits can only be obtained if the methods are used correctly, with properly designed pipes installed using suitable machinery and with adequate control. Acceptance of the methods by clients was initially inhibited by a number of problems resulting in drives which failed to reach their destination, pipes which cracked during installation, or alignments which fell outside specified tolerances. Although improvements were made gradually through experience, more rapid advance was prevented by lack of understanding or information on many of the factors affecting installation. The remainder of this paper describes a major research project set up to investigate these factors, and presents some of the results obtained to date from the research.

2. OXFORD PIPE JACKING RESEARCH PROJECT

Pipe jacking as a technique was introduced in the USA about 100 years ago, but its use was sporadic up until about the 1960's. Since then, quite rapid development has taken place, mainly in Germany and Japan. The method is now widely used in Japan and Western Europe, it is again finding increasing recognition in the USA, and has been taken up in the Middle East and elsewhere. However improvements to increasingly sophisticated technology were not matched by advances in basic understanding of many factors of pipe jacking, and designs were essentially empirical and relied heavily on the experiences of contractors. In 1983, Craig's report for the UK's Construction Industry Research and Information Association (CIRIA) reviewed the method and proposed a list of areas requiring research. This was taken up by the Pipe Jacking Association and Concrete Pipe Association and research was initiated at Oxford University, with overall objectives in line with Craig's recommendations:-

- (a) prediction of friction loads in different ground conditions;
 - (b) load/deflection characteristics of the joints with different packing materials;
 - (c) the effects of cyclic loading on the pipes at intermediate jacking stations;
 - (d) the effect of lubricants in reducing friction along a pipe;
 - (e) the development of a site investigation test suitable for the prediction of frictional forces.
- The prediction of ground movements associated with pipe jacking has been added to this list.

The research is now in its fourth phase. Phase 1, from 1986 to 1989, involved laboratory testing of model concrete pipes and of various joint packing materials. The crucial influence of the inevitable small angular misalignments between pipes on the localised stresses at the pipe joints under axial loading, and the importance of the packing material in helping to alleviate these stress concentrations, was clearly established (Milligan and Ripley, 1989). Phase 2, from 1989 to 1992, involved the monitoring of full-scale pipe jacks on five active sites: the nature of this work and some of the results obtained are discussed later in this paper. Phase 3, started in 1992, is continuing the field monitoring on five further sites, three of which have been completed to date, while Phase 4 comprises finite element analysis of pipes and pipe joints. The intention here is to develop improved joint designs initially by computer modelling and then substantiate these by laboratory and field testing. Improved joints should reduce the likelihood of joint failures under load and increase the maximum forces that can be transmitted safely.

The work has been supported by the PJA and the Engineering and Physical Sciences Research Council (EPSRC; formerly SERC, the Science and Engineering Research Council). The fieldwork has had substantial additional funding from five major regional water companies - Northumbrian, North-West, Severn-Trent, Thames and Yorkshire. The water companies have also provided the research sites, mainly on new sewerage schemes within their areas, and borne site costs averaging about £15000 per site. Management of the project has been greatly assisted by a management group with two representatives from the PJA, one from each of the water companies, and the author and research assistants from Oxford University. Overall the project has been an excellent example of successful co-operation between academia and industry, including clients, designers, contractors and pipe manufacturers.

3. FIELD MONITORING - INSTRUMENTATION AND SITES

The overall research objectives are being met by observations of the behaviour of pipes, pipe

joints and the surrounding ground. The instrumentation developed to do this is shown diagrammatically in Figure 4. It has all worked satisfactorily except for the ground convergence indicator, which was abandoned after the first site. The main instrumented pipe is inserted at a predetermined location within the pipe line, and for Phase 2 contained the following instruments:-

- (i) four contact stress cells, to measure both radial and shear total stresses at the pipe-soil interface, with their active face flush with the pipe surface and having a similar surface finish;
- (ii) four pore pressure cells close to the contact stress cells to measure the local pore pressure and hence determine the effective radial stress;
- (iii) three joint movement indicators at each end of the pipe, to measure the movements across the joint gaps;
- (iv) six extensometers fitted to the internal surface of the pipe to measure the compression of the pipe under load;
- (v) twelve thin joint pressure cells built into the packer in the joint at either end of the pipe, to measure the magnitude and distribution of the pressures transmitted across the joint.

In the jacking pit the total jacking load is monitored continuously using load cells on the jack rams, and the overall movement of the pipe string by a rotary displacement transducer. Details of the design, construction and calibration of the instruments are given in Norris and Milligan (1991). All instruments were designed to operate successfully in the aggressive tunnel environment, be sufficiently accurate but robust and reasonably simple to calibrate, and disrupt normal site operations as little as possible. The instruments are mostly fitted to the pipe before insertion into the pipe line, to minimise delays to the contractor, the necessary holes and fixings having been built in during the manufacture of the pipe. All instruments are retrieved at the end of each scheme for re-use; many are now on their eighth site and still operating satisfactorily. Calibration, and where necessary refurbishment, of instruments takes place between sites.

The cables from the instruments are connected to data acquisition boxes, one in the bottom of the jacking pit and the others mounted inside the instrumented pipe. The signals are converted from analogue to digital, and returned by a single cable to the computer logging the information, which is housed in a container positioned alongside the jacking pit. The only other cable along the pipe line is the power cable to the instruments. These cables are broken and fed through each new pipe in turn in the same way as the various services for the contractor. A schematic of the arrangement is shown in Figure 5. The instruments and data acquisition boxes in the instrumented pipe are protected from the contractor's normal site operations by a telescopic liner. This restricts the pipe diameter locally by about 300mm, but the exact size and placement of the liner are arranged with the contractor to cause as little disruption as possible; site records have shown that loss of productivity after installation of the instrumented pipe is usually marginal.

In addition to the instrument readings, a detailed site log is kept, full surveys of the pipe line and levels are made regularly, and face logging and soil sampling are used where possible to supplement the initial information on ground conditions. Ground movements around the tunnel were measured on the final scheme of Phase 2, and are being measured on all sites in Phase 3. A typical measurement array is shown in Figure 6. For Phase 3, the number of contact stress cells in the instrumented pipe has been increased from 4 to 12, with sets at each end of the pipe as well as around the middle. Also the pore pressure cells have been incorporated into the contact stress cells to make interpretation of the results easier when the readings from

both vary rapidly as the pipe line is pushed.

Details of the Phase 2 sites are given in Table 1, and of the sites monitored to date in Phase 3 in Table 2. The principal variables have been the ground conditions, depth of cover, location of the test pipe in the line, and use or absence of lubrication. Pipe diameters varied between 700 and 1800 I.D., but all were of reinforced concrete; joints were of the steel-banded type except in the second scheme of Phase 3. Medium density fibreboard (MDF) was specified for the packers since this had been found to be the best of the available materials in Phase 1, and calibration of the joint pressure cells was carried out in conjunction with this material. Excavation in the early drives was always by hand, but on later sites excavation has been by slurry or open-face tunnel boring machines. exact details of the instrumentation have varied somewhat from site to site, depending on specific scheme requirements; for instance the joint pressure cells were not used in the first two schemes of Phase 3, in the first case because they could not be fitted into the in-wall joints, in the second because the pipe walls were too thin to accommodate the cells.

4. FIELD MONITORING - RESULTS

Very large quantities of high-quality data have been and are being acquired from the field work. Complete evaluation of the results will take several years, and the data will no doubt be returned to on many occasions to check theoretical advances made in the future. Results from Stage 2 have been studied in some detail, and a short summary of the most important findings follows below. More detail may be obtained from the report by Milligan and Norris (1994). This document started as a set of notes for a full-day workshop held by the Pipe Jacking Association to alert their members to the findings of the research project. Results have also been disseminated through seminars and technical meetings, and papers to relevant conferences - Norris and Milligan (1992a and 1992b), Milligan and Norris (1993).

Starting with the instruments in the thrust pit, the jacking records showing the progress of the drive and the increase in total jacking force are presented in Figures 7 (a) to (e). These generally show an initial load corresponding to the face load at the shield, and an approximately linear increase in frictional load with the length of the pipe line. Notable changes in the rate of increase of frictional load occurred in schemes 1 and 2, the former due to increased interaction between pipe and ground following very heavy rain which softened the clay to the depth of the pipe, the latter following a change in soil type from weathered mudstone to glacial clay. The very peaky jacking force plot for scheme 3 reflects increases in jacking load whenever the pipe line was stationary, the load dropping off as each push of the pipe line occurred; this behaviour is typical of cohesive soils, and is much more marked in high-plasticity clays (London Clay in this case). A detailed record in Figure 8 of the installation of one pipe length shows that increases are apparent after only a few minutes, while increases of over 70% can occur over a few hours.

The peaks of jacking force in the record for scheme 4 (Figure 7d) are the result of using the shield to trim the tunnel excavation by up to 20mm, giving very substantial increases in face load. The use of lubrication towards the end of the drive was clearly effective until problems were encountered with running sand at the very end. In scheme 5 the high initial rate of increase in jacking load reflects the immediate collapse of the ground on to the pipe line and the consequently high frictional resistance. Introduction of lubrication reduced the rate of increase of load by a factor of 10; the bentonite slurry was under sufficient pressure to

stabilise the tunnel bore and then act as an effective lubricant in the small annular space around the pipes caused by the overbreak from the tunnelling machine shield. The double trace towards the end of the drive results from the use of an interjack.

Figures 9 (a) to (d) show the tunnel alignment surveys for four of the schemes, also the local pipe joint angles (β) derived from the line and level measurements and much more accurately from the special instrumentation. It is notable that successive tunnel surveys show little if any tendency to change as jacking progresses: thus once a misalignment has been established by the passage of the shield, it will affect all subsequent pipe joints passing that point. Control of alignment is often most difficult and poorest at the start of a drive, yet this will be the point subjected to the greatest jacking forces at the end of the drive. Detailed study of the joint angles during jacking has also shown that there is little tendency, except perhaps in loose soils, for the pipe line to straighten significantly under load. Other points of interest are that joint angles in well-controlled drives (within line and level specification) were typically in the range 0 to 0.3 degrees, with occasional values up to 0.5. Some loss of alignment in scheme 5 led to joint angles up to nearly 1.0 degrees; significant pipe damage was experienced at the locations at which this occurred. The change in pipe levels in this scheme of nearly 20mm between the first two surveys was due to the introduction of full bentonite slurry lubrication, causing the pipe to float within the excavated bore.

Typical contact stresses between pipe and ground measured by the stress cells are shown in Figure 10 (from scheme 4). In this case the ground was stable in the short term due to capillary suction in the silty sand above the water table, and the pipe line was essentially sliding along the base of a stable tunnel. Local stresses on the bottom of the pipe fluctuated rapidly with maximum radial stresses around 300 kPa and shear stresses about 160 kPa. By comparison with Figure 9 (c) it can be seen that there is a close correlation between the points of high radial stress and the level alignment survey; under load the pipeline tends to act as a prestressed beam spanning between the high points in the tunnel. A similar effect was observed due to misalignments in the horizontal plane in scheme 5, the pipeline being forced against the soil on the inside of bends.

The measurement of both shear and radial stress allowed the actual interface friction angle on the pipe wall to be evaluated. Results for scheme 4 are shown in Figure 11; both effective and total stress plots show essentially frictional behaviour. When lubrication was introduced, the local interface friction at first reduced markedly (from line A to line C in Figures 11 (c) and (d)); subsequently, since the annulus was not kept full topped up with lubricant, the effectiveness of the lubrication lessened and the interface friction tended towards its original value (line B). In the fully lubricated section of scheme 5, very low interface friction of around 2 kPa was measured. Rather surprisingly, frictional behaviour in terms of total stress was also observed in the schemes in cohesive soils, except that the undrained shear strength seemed to provide an upper bound; more work in softer clays is needed to check this effect. The observed friction angles are summarised in Table 3.

Table 4 summarises the measured face load and overall frictional resistance for the five schemes. In the final two columns the average frictional resistance is compared with the empirical values quoted by Craig (1983), based on previous experience, and the resistance due purely to the self-weight of pipes sliding inside a stable bore, for the schemes for which this model was appropriate, calculated using the measured local friction values. If this latter value is increased by about 25%, to allow for typical effects of misalignment, a good estimate of

the frictional force is obtained. For design purposes, suitable values of local friction angle may be obtained from shear box tests taken to large displacements. Theoretical models which appear to fit the field behaviour are also available when either soft clay or cohesionless soils make close contact with the pipe; however the case of stiff over-consolidated clays which swell due to stress relief and may generate high local contact stresses on the pipes, as measured in scheme 3, is a complex problem in soil-structure interaction requiring further attention.

Finally, much information was also obtained on the structural performance of the pipes and their joints under the action of jacking forces. The extensometers showed that the strains and stresses in the pipe barrel were within the elastic range and could be estimated using short-column elastic theory. On the other hand, stresses at the joints were large and locally concentrated due to the small joint misalignments. Thus the pressure cell readings shown in Figure 12 prove that the joint stresses are concentrated into a small section of the pipe circumference, even at modest joint misalignment angles, while local values up to 60 N/mm² were recorded in scheme 5. Theoretical calculations based on actual behaviour of the joint packing material provide a good correlation with the field data and allow design curves such as shown in Figure 13 to be drawn up. The importance of controlling maximum joint angles to allow adequate jacking forces is very obvious.

5. SUMMARY

Pipe jacking and microtunnelling are potentially powerful techniques for the installation of pipes and small-diameter tunnels. To help realise the full potential of the methods, a major research programme is in progress at Oxford University, with support from the research council, the association of specialist contractors, and five water companies. It has now been in progress for some eight years, and has included laboratory testing, field monitoring and finite element analysis. Results of direct importance to industry have been obtained, and are being disseminated through papers, reports, seminars and workshops. Input has also been provided to a new Guide to Best Practice for the industry, and to a new European Standard currently under production. In addition, the research has provided a forum for co-operation within the industry, and a focus for the drive towards technical awareness and improvement by all involved.

6. ACKNOWLEDGEMENTS

The financial support provided by the Engineering and Physical Sciences Research Council, the Pipe Jacking Association, Northumbrian Water, North West Water, Severn Trent Water, Thames Water and Yorkshire Water, is gratefully acknowledged. The five water companies also provided the necessary sites for the field monitoring, and carried additional site costs. Successful progress of the work has depended on the hard work of the research assistants - Kevin Ripley, Paul Norris, Mark Marshall and Jian-Qing Zhou - and the co-operation of many individuals, too numerous to name, from representatives on the Management Group to operatives on the various sites. It has been a great pleasure to work with so many positive, enthusiastic and helpful people.

Figures 1 to 3 are reproduced with the kind permission of the Pipe Jacking Association from their publication "A guide to pipe jacking and microtunnelling design"

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Date	1	2	3	4	5
Location	Botton, Lancs.	Gatcliffe, Tyneside	Hamer Oak, SE London	Crookley, Lancs	Cheltenham, Glos
Client	HW Water Bolton MBC	Horthumbrian Water	Thames Water	HW Water Clonley D.C.	Severn Trent Water Cheltenham BC
Contractor	Laverbore	DECT	Bathale	Bathale	Litley
Pipe Supplier	Buchan	ARC	Buchan	Spun Concrete	Spun Concrete
Pipe ID (mm)	1200	1350	1000	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 Ho. 3	Pipe Ho. 10	Pipe Ho. 15	Pipe Ho. 16	85m from front end.
Lubrication	H ₂ O	H ₂ O	H ₂ O	No/Yes	Yes
Packer +	MDF	MDF	MDF	MDF	MDF
Excavation	Hand	Hand	Hand	Hand	Slurry TBM.

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

Table 1 Details of Stage 2 sites

Scheme No	Soil type	Friction angle (deg.)
1	Glacial clay	19
2	Weathered mudstone	17
3	London Clay	12.7
4	Silty sand	38
4	Sandy silt	30

Table 3 Local interface friction angles

Date	1	2	3
Location	East London	Southport	Seaham
Client	Thames Water	North West Water	Northumbrian Water
Contractor	Clancy	Miller	Donelon
Pipe supplier	ARC	Spun Concrete	Buchan
Pipe I.D. (mm)	1500	1000	1800
Ground type	London Clay	Dense silty sand	Glacial till
Cover (m)	6 - 8	5 - 8	6
Drive length (m)	75	160	310
Position of test pipe	Pipe No. 11	Pipe No. 2	Pipe no. 74
Lubrication	No	Yes	No/Yes
Packer material	Medium density fibreboard (MDF)	MDF	MDF
Excavation method	Hand	Slurry TBM	TBM

Table 2 Details of Stage 3 sites

Scheme	Measured face load (kN)	Average friction force (kN/m)	Average friction stress (kPa)	Craig (1983) (kPa)	Friction from self-weight (kN/m)
1	120	7.2	1.5	5 - 18	6.1
2	950	8.0	1.5	2 - 3	7.0
3	300	54.4	7.6	5 - 20	
4	100 - 800	23.1	4.2	5 - 20	18.7
5	1200	10	2.2	10 - 15	6.5

Table 4 Face resistance and pipe line friction

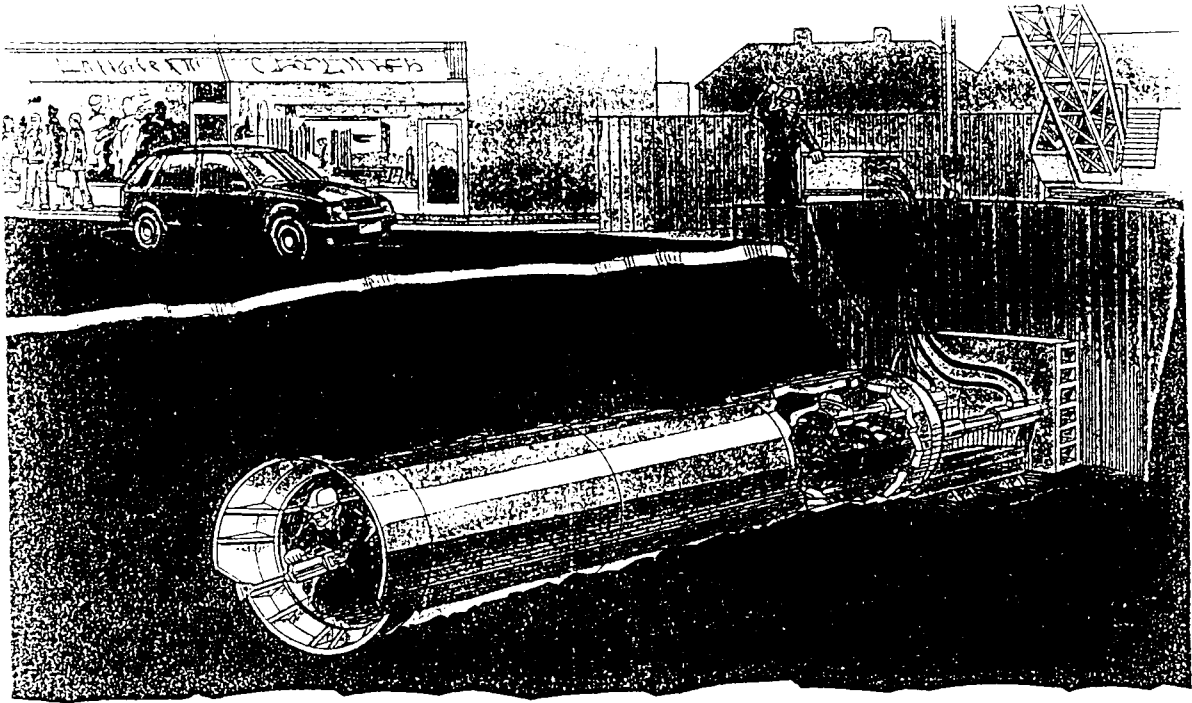


Figure 1 General arrangements for pipe jacking with hand excavation.

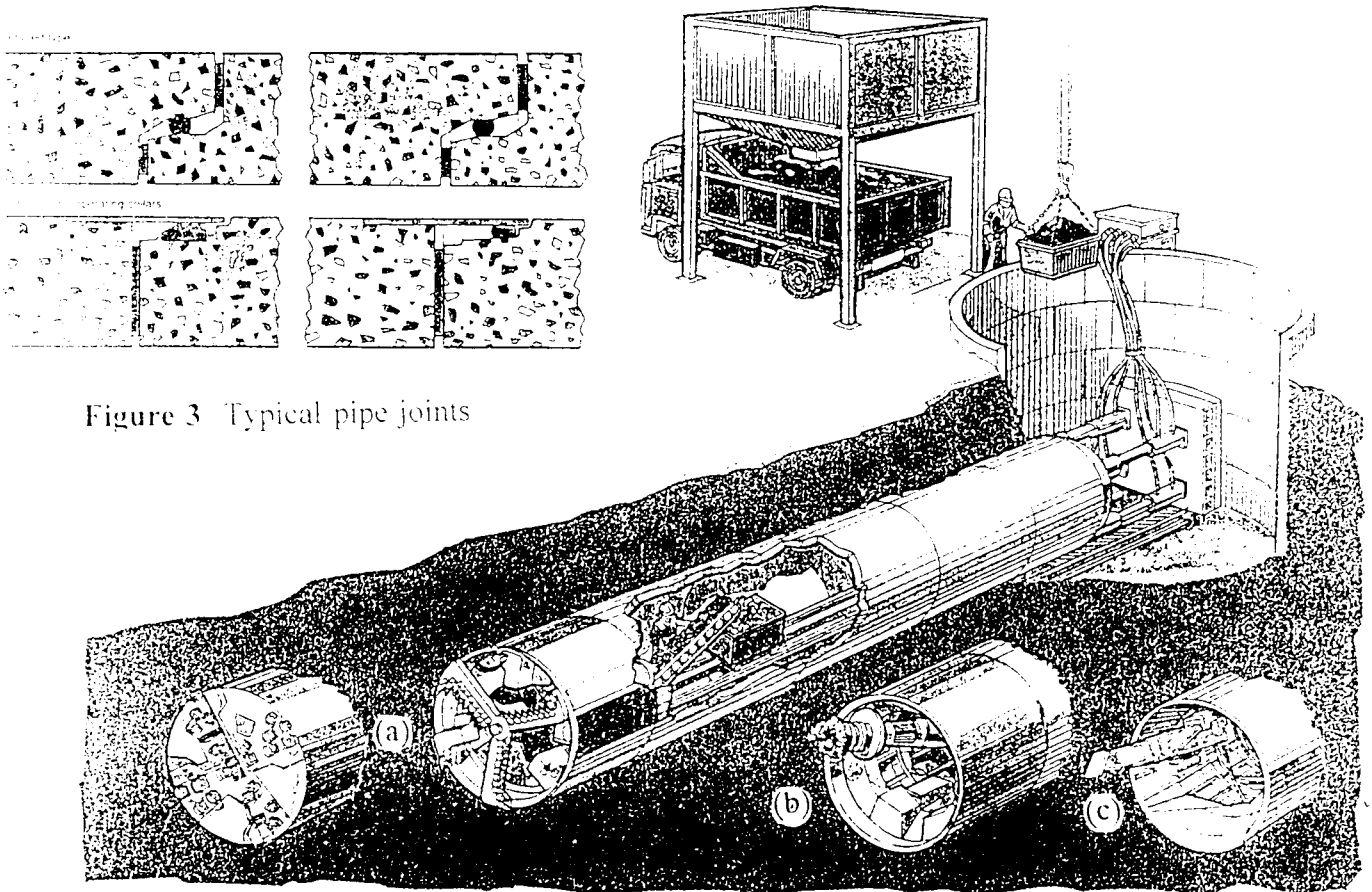


Figure 3 Typical pipe joints

Figure 2 Methods of excavation

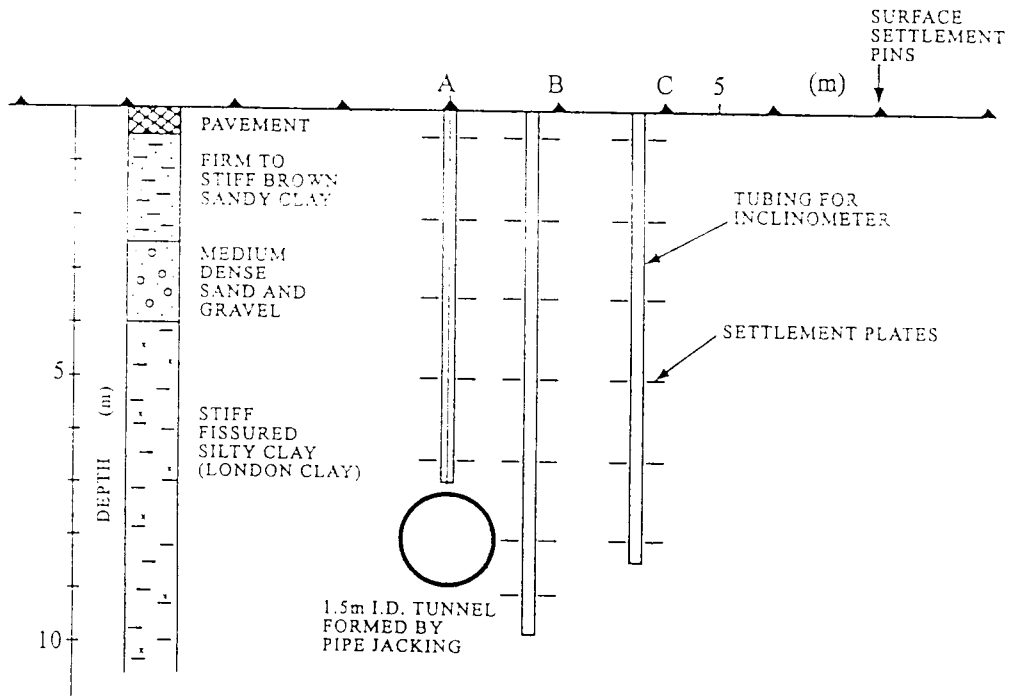
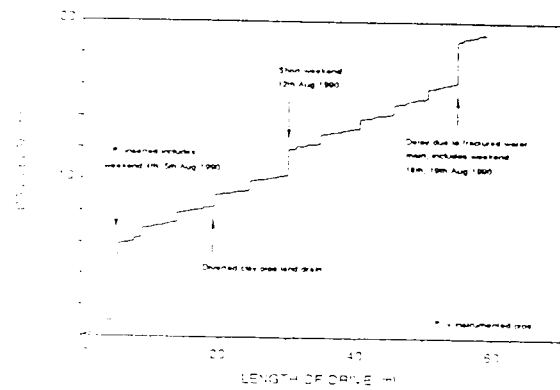
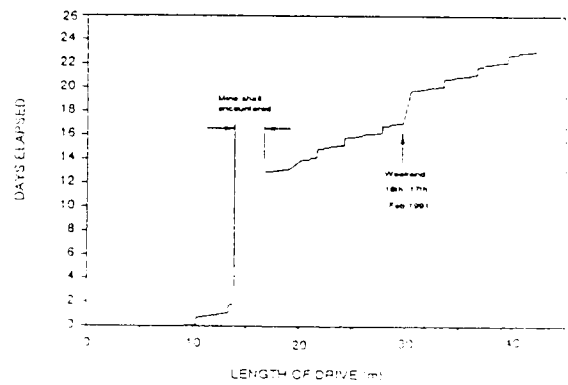


Figure 6 Typical ground instrument array

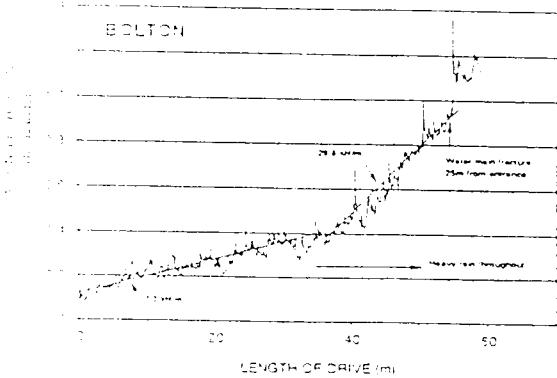
(a) Scheme 1



(b) Scheme 2



BOLTON



NEWCASTLE

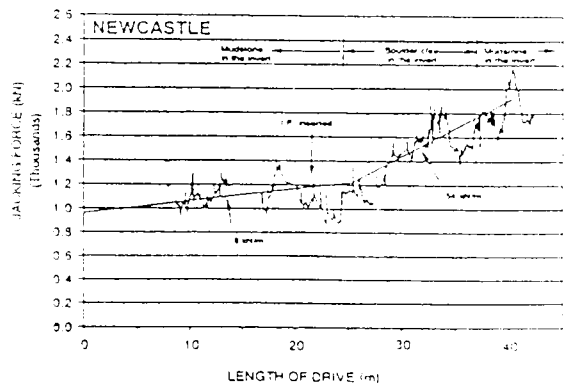
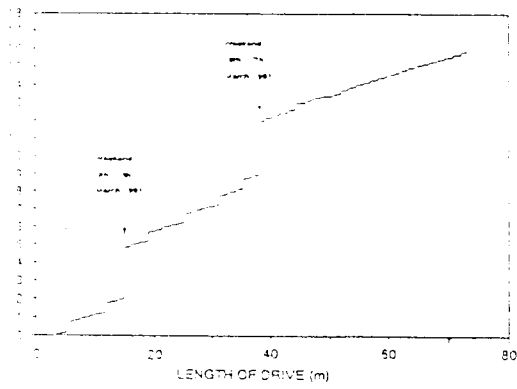


Figure 7 Jacking records for schemes 1 to 5

(c) Scheme 3



(d) Scheme 4

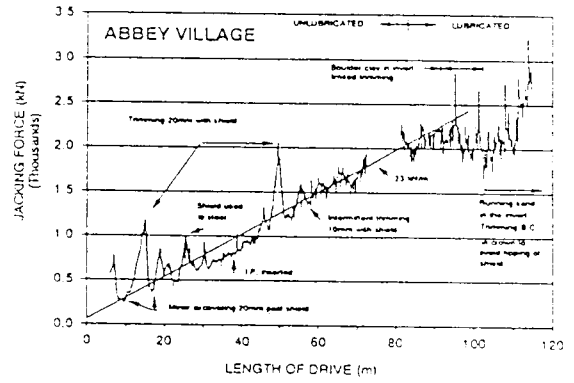
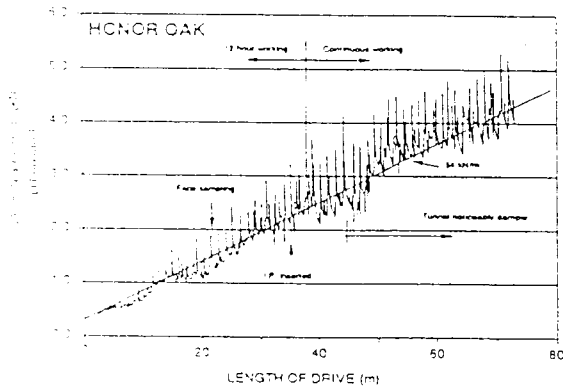
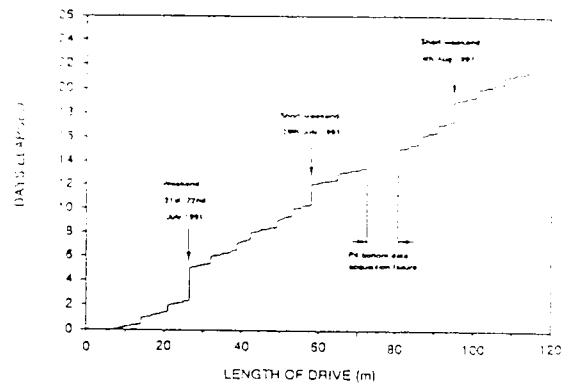


Figure 7 Jacking records for schemes 1 to 5 (cont.)

(e) Scheme 5

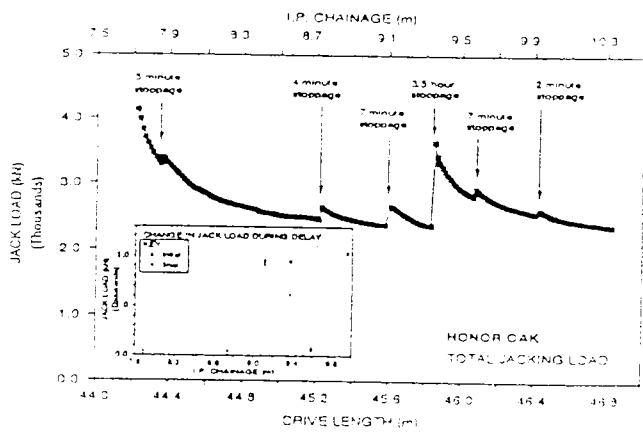
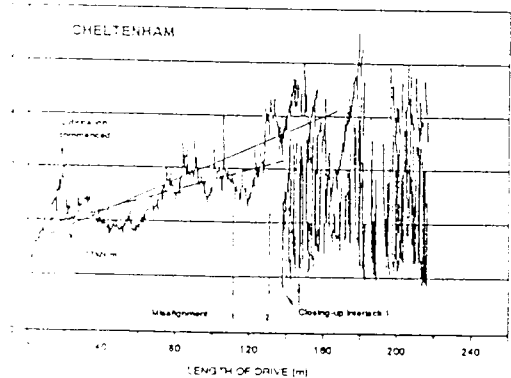
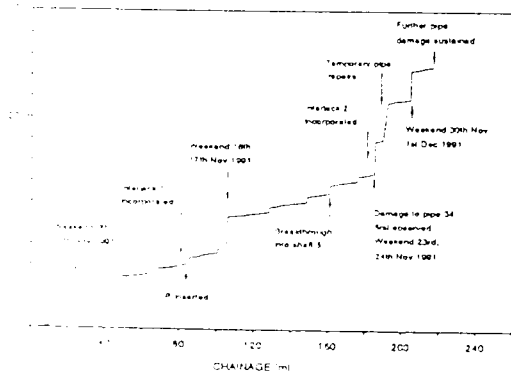
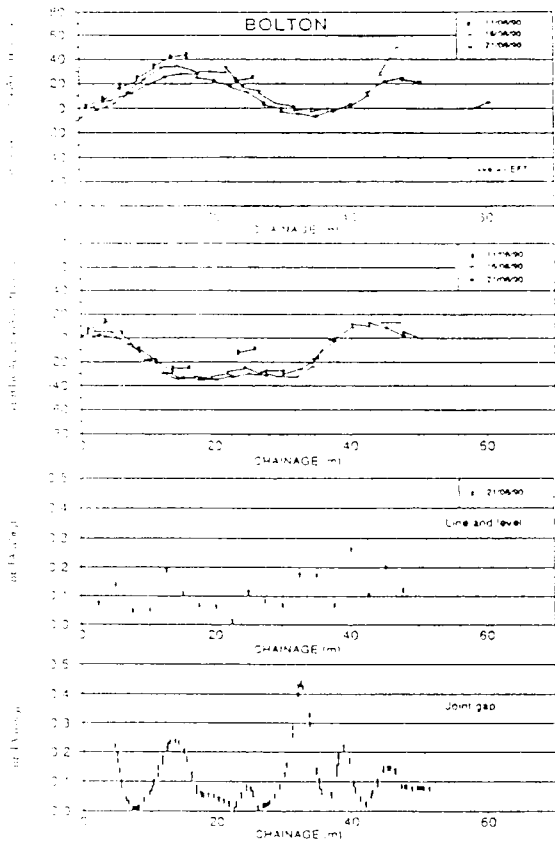


Figure 8 Time effects on jacking force

(a) Scheme 1



(b) Scheme 3

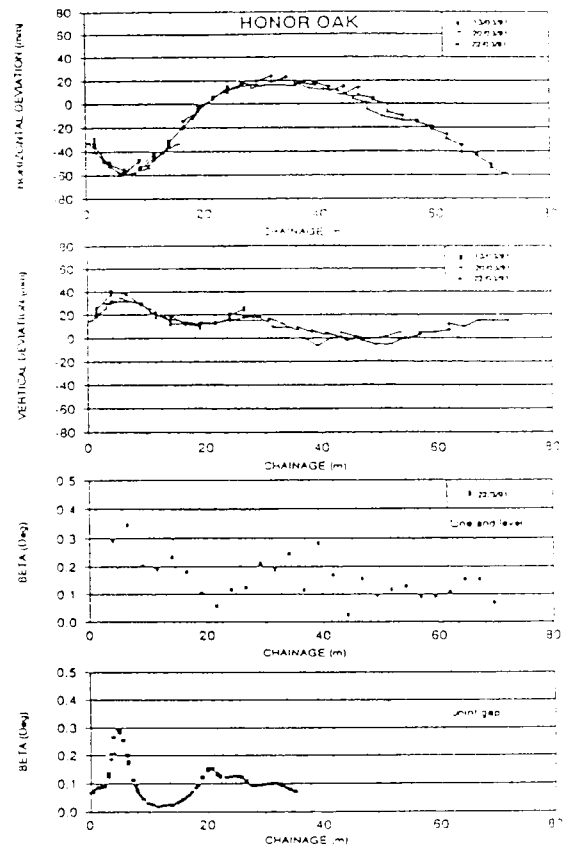
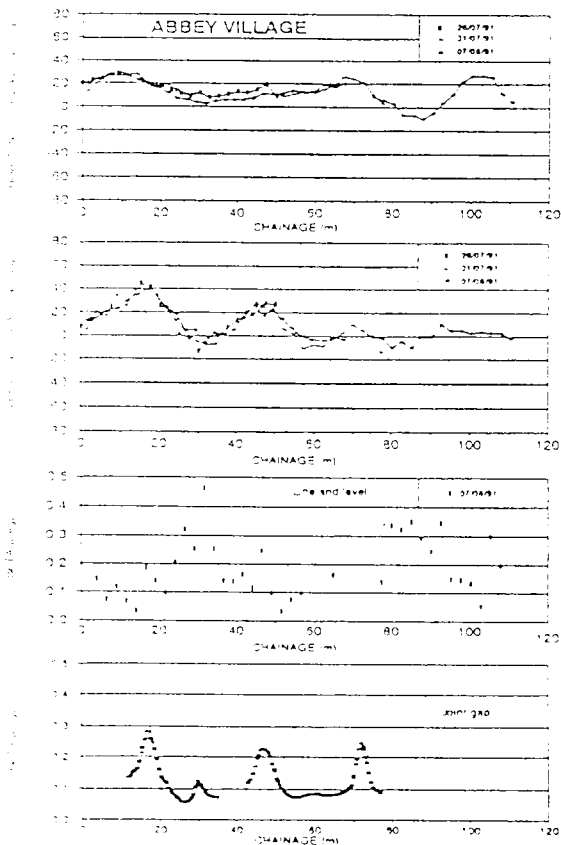
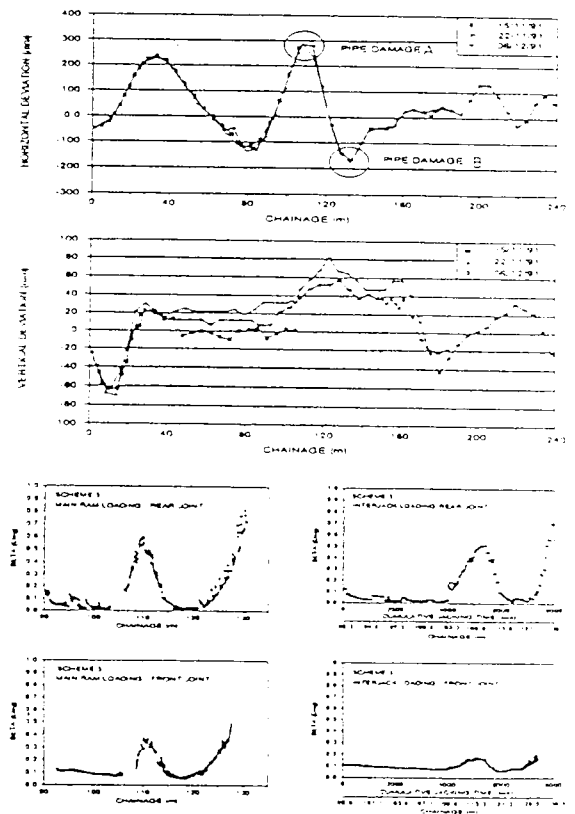


Figure 9 Tunnel alignment surveys

(c) Scheme 4



(d) Scheme 5



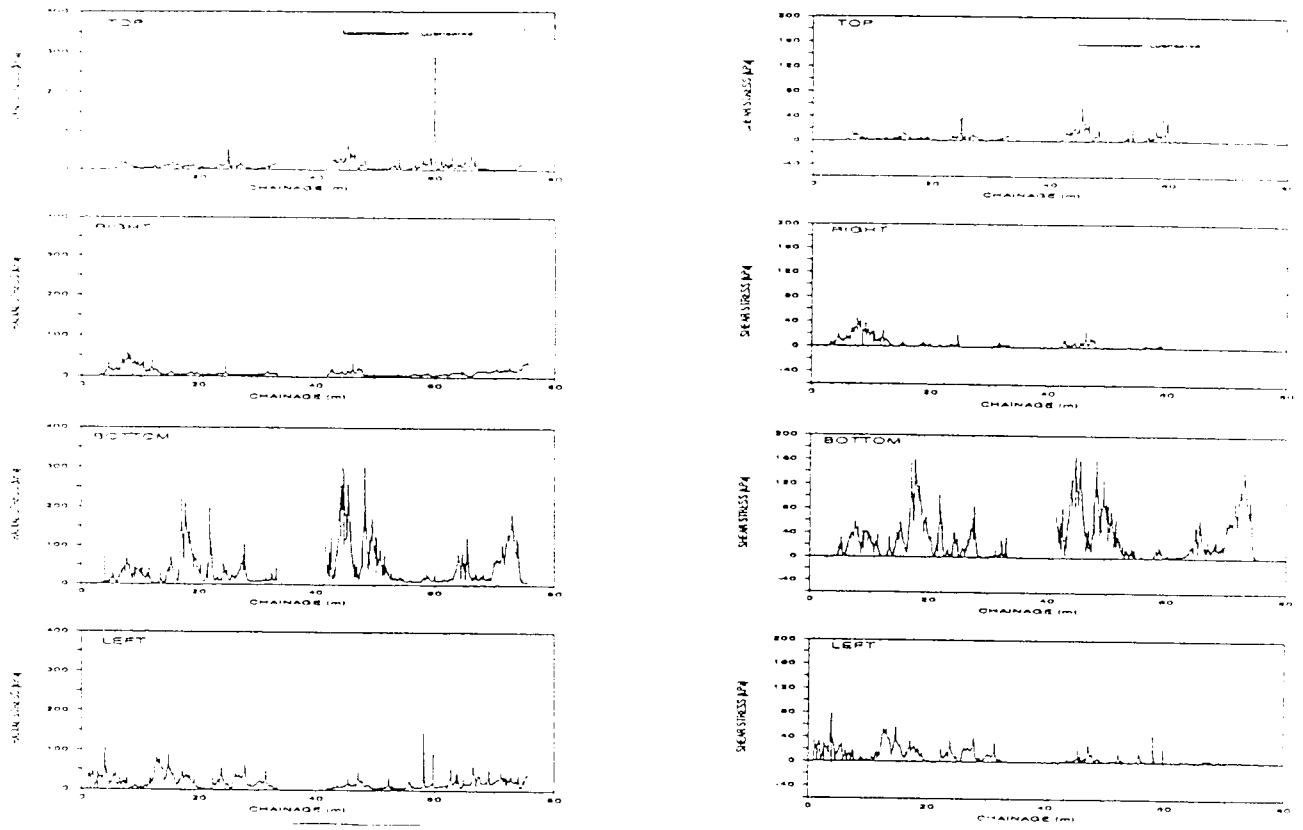


Figure 10 Radial and shear stresses - scheme 4

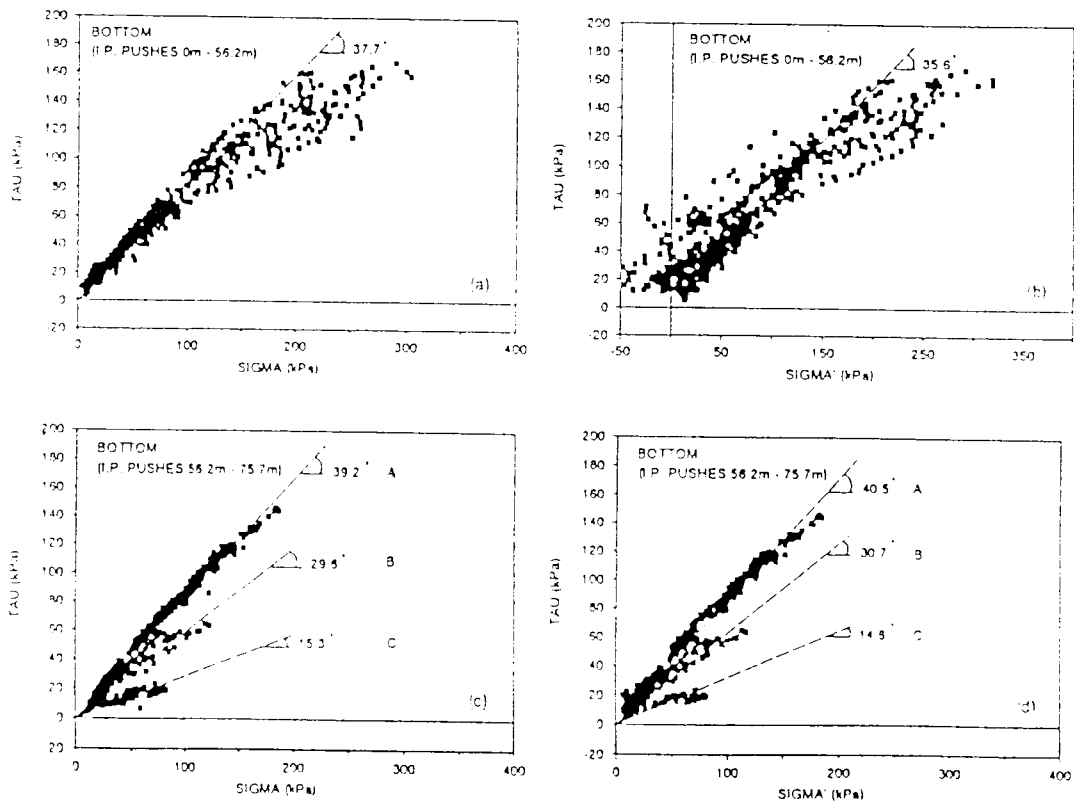


Figure 11 Plots of shear stress against radial stress - scheme 4

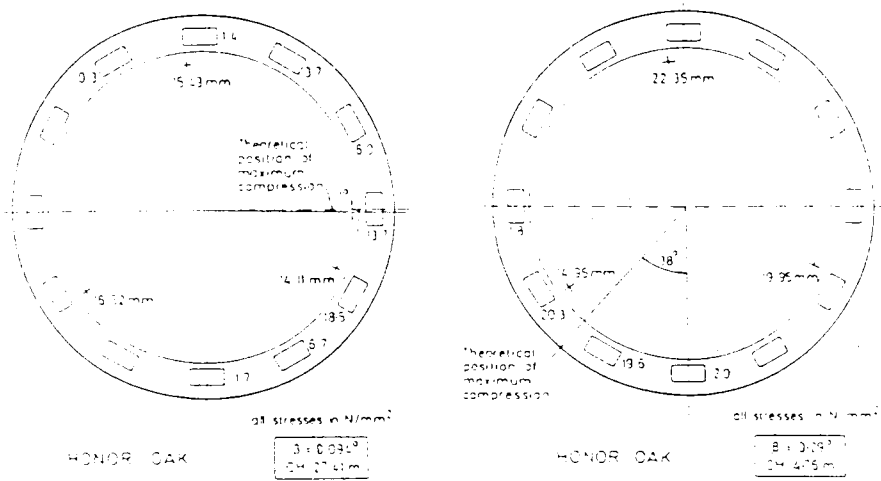


Figure 12 Pressure distributions in pipe joints

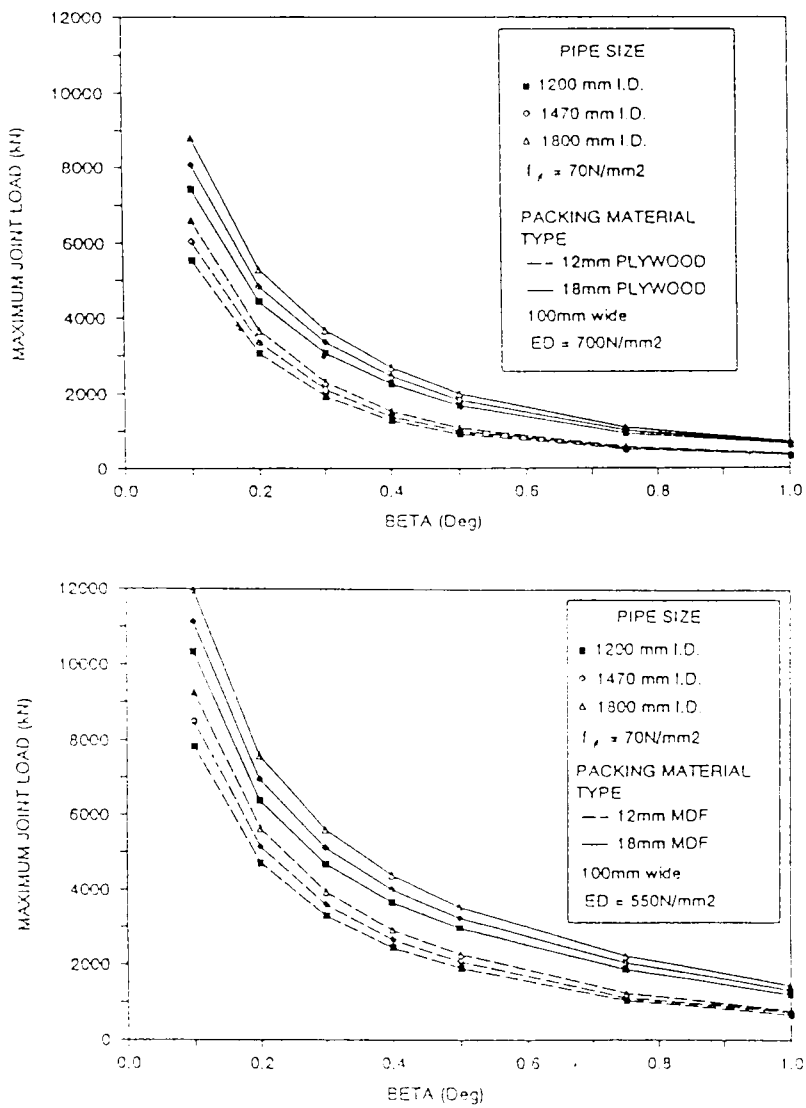


Figure 13 Permissible pipe end loading related to joint misalignment angle

Ground movements due to construction of pipe jacked tunnels

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Ground movements due to construction of pipe-jacked tunnels

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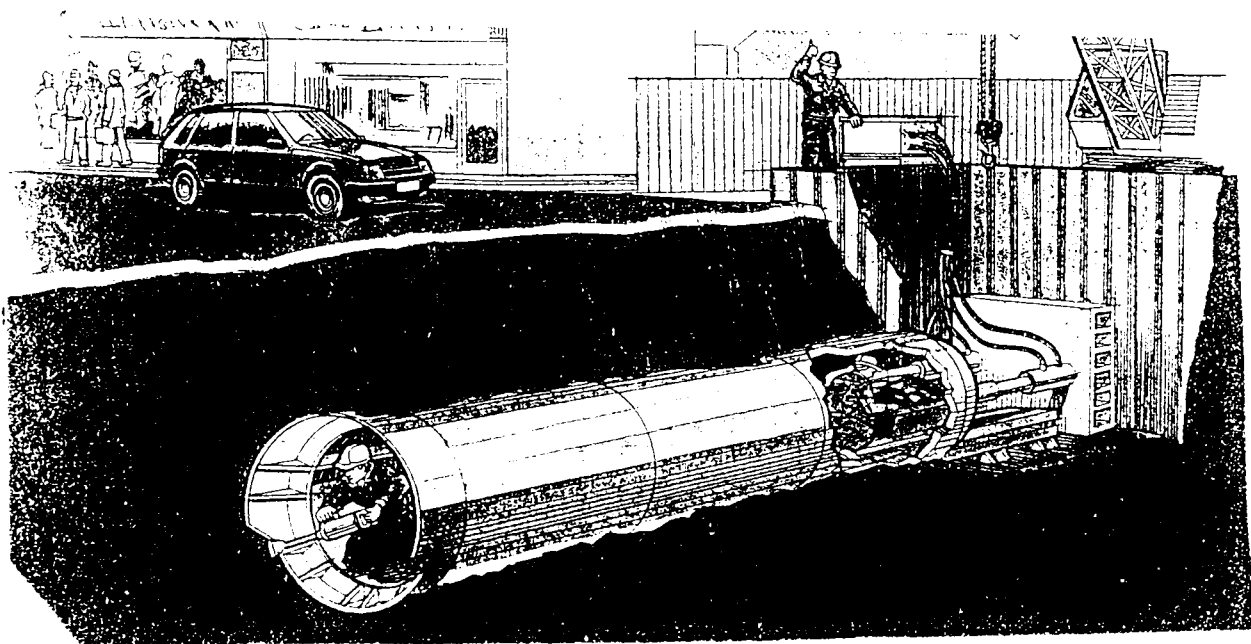
SYNOPSIS: Ground movements due to tunnelling in urban areas may cause problems with existing buildings and services. The pipe jacking technique appears to offer a good opportunity for limiting ground movements by the installation of a rigid lining immediately behind the tunnelling shield, but the requirements for ease of jacking and minimisation of settlements may be in conflict. Measurements of ground movements made on three sites in different ground conditions are reported and compared with settlements predicted by accepted empirical methods. In all cases the measured movements were small, and less than predicted.

1. INTRODUCTION

1.1 *Pipe jacking*

The technique of pipe jacking involves the construction of lined tunnels or pipe lines by pushing a "string" of pipes through the ground from a thrust or jacking pit to a reception pit, using large hydraulic jacks (see Figure 1).

Additional pipes are added at the thrust pit as the pipe line is advanced. Excavation takes place at the front end, by methods ranging from hand excavation within a simple shield through various mechanical systems to full-face tunnel boring machines; in poor ground these machines may use slurry pressure or earth-pressure balance to support the tunnel



*Figure 1 Pipe jacking with hand excavation
(with permission of the Pipe Jacking Assoc.)*

face. Conventional pipe jacking is typically used to instal pipes with internal diameters in the range 1.0 to 2.5m, but large box-section culverts may also be constructed. Pipes too small for man-entry (less than 900mm I.D.) may be installed as microtunnels, using miniaturised mechanised systems, operated by remote control.

1.2 Ground movements

Pipe jacking is often used for the construction of new sewer pipes in urban areas, as it leads to far less disruption than laying pipes in open trench. It can be economically competitive with other tunnel lining techniques, such as bolted segments, particularly in difficult ground conditions, and has the advantages of requiring no secondary lining and containing relatively few joints. In urban areas in particular, a major concern may be the control of ground movements which could cause damage to buildings, road pavements, and other buried services. With pipe jacking the tunnel lining is installed immediately behind the tunnelling shield; in addition the pipes themselves, whether of reinforced concrete or other material, must be relatively strong and stiff to withstand the jacking forces. After installation, their deformations due to ground loading are likely to be negligible. Pipe jacking therefore appears to offer a good approach to minimising ground movements, provided no large displacements occur due to stress relief or instability at the tunnel face. However, in stiff clay it is normal to excavate at the shield to a slightly larger diameter than the outside of the pipes, to reduce contact with the ground and prevent jacking forces from becoming excessive; in cohesionless soils a similar "overcut" or "overbreak" is created which may be filled with bentonite slurry to support the ground and prevent collapse onto the pipes. In either case, slurry within the overbreak annulus may act as a lubricant to reduce further the frictional forces between soil and pipes. The closure of this void must lead to some settlement.

1.3 The Oxford research project

For the past eight years a major research programme on pipe jacking has been in

progress at Oxford University, supported by the Pipe Jacking Association, the Engineering and Physical Sciences Research Council, and five U.K. water companies (North-West, Northumbrian, Severn-Trent, Thames and Yorkshire). The programme of research is described by Milligan and Norris (1993), and results of the first two stages of the work summarised by Milligan and Norris (1994). Stage 2 and the current Stage 3 have involved site monitoring of pipe jacks during construction by incorporation of a specially-instrumented pipe into the pipe string. In addition, ground movements have been measured on the first three sites so far monitored in Stage 3; results from these three sites are presented in this paper.

2. PREDICTION OF SETTLEMENTS

For a major tunnelling scheme, such as for a railway or metro system, it would be appropriate to conduct extensive ground investigations and analyses to determine the expected ground movements in sensitive areas. Pipe jacks for sewers are usually relatively minor schemes in comparison and it is normally appropriate to carry out only routine site investigation and use empirical methods to calculate settlements. The most commonly used method depends on the fact that the shape of the surface settlement trough developing above a tunnel approximates to a Gaussian distribution curve (Peck 1969) - see Figure 2. The surface settlement S_o is described by

$$S_o = S_{o, \max} \exp\left(-x^2/2i^2\right) \quad (1)$$

where $S_{o, \max}$ is the maximum settlement above the centre line of the tunnel and i is the distance from the centre line to the point of inflexion of the trough. From collected field data O'Reilly and New (1982) produced the following expressions for the parameter i :-

For granular soils,

$$i = 0.28z_o - 0.12(\text{m}) \quad (2)$$

and for cohesive soils

$$i = 0.43z_o + 1.1 \text{ (m)} \quad (3)$$

where z is the depth of the tunnel axis below ground area.

Mair et al. (1993) have shown that, for clays, Eq (3) may be reasonably approximated by

$$i = 0.5z_o \quad (4)$$

and have also shown from site data that the variation of i with depth z below the surface may be represented by

$$i_z = 0.175z_o + 0.325(z_o - z) \quad (5)$$

The value of $S_{o, \max}$ at the surface or $S_{z, \max}$ at depth z below the surface, may then be obtained by equating the volume of the settlement trough V_s to the volume of "ground loss". By integration, it can be shown that

$$V_s = S_{\max} i \sqrt{2\pi} \quad (6)$$

$$S_{\max} = \frac{V_s}{i\sqrt{2\pi}} \quad (7)$$

The ground loss due to tunnelling has two main sources - movements at the tunnel face and closure of the ground between the tunnelling shield and the final tunnel liner. In this paper the calculations assume that the former is negligible, either because the face is continually supported by slurry pressure or because the excavations are in stiff clay. For the relatively small diameters involved the tunnel faces in stiff clay are highly stable and only very small movements due to elastic unloading of the ground will occur. The only significant contribution to ground loss is then due to the ground being excavated to a slightly larger diameter D_e than the external diameter of the pipe D_p . Hence it is assumed here that

$$V_s = \frac{\pi}{4} (D_e^2 - D_p^2) \quad (8)$$

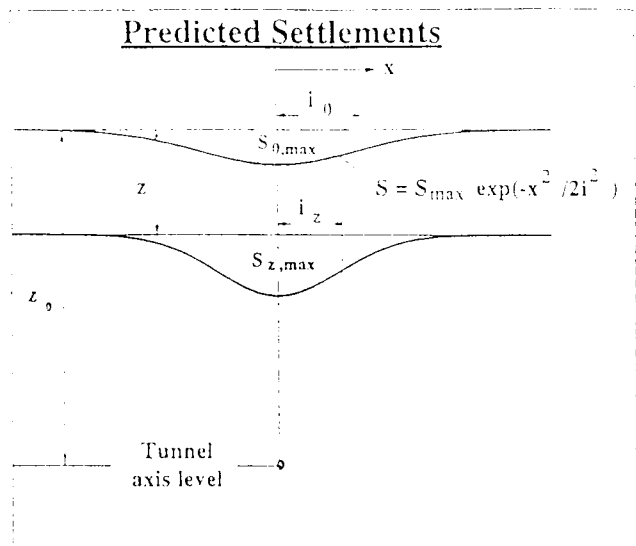


Fig.2 Calculation of settlements

3. FIELD MEASUREMENTS

3.1 Site details

Table 1. Details of sites

Site No.	1	2	3
Location	Walthamstow, East London	Southport, Lancashire	Seaham, Co. Durham
Drive length (m)	75	160	310
Dates	17/11/93-9/12/93	24/2/94 - 8/3/94	2/7/94 - 5/9/94
Ground type	London clay	Dense silty sand	Glacial clay
Pipe O.D. (m)	1.800	1.200	2.160
Shield O.D. (m)	1.800	1.220	2.170
Cutter diam. (m)	1.850 (av.)	1.235	2.200
Excavation by	Hand	Slurry TBM	TBM
Axis depth (m)	3.5	5.6	6.95

Brief details of the schemes at the three sites are given in Table 1. At Site 1 the tunnel was excavated by hand through London clay using pneumatic tools within a simple steerable shield. The overbreak on excavation was provided by the miner cutting the clay with a clearance around the outside of the shield. This clearance varied with position around the shield, from approximately 10mm at the crown to a maximum of about 50mm at the shoulder and zero at the invert, the average value being

about 25mm. Some bentonite slurry was injected as lubricant around the pipe, but the overbreak annulus was not usually completely filled nor was the slurry pressurised.

At Site 2 the pipe line was in fine sand below the water table. A Miller-Markham "Super-Mighty" microtunnelling system was used, with slurry pressurisation of the face and slurry transport of excavated material to the surface. The overbreak at the cutting head was 15mm (on diameter) relative to the shield, which in turn was of 10mm larger diameter than the pipes. The overbreak annulus between shield and pipes was filled with bentonite slurry which was pressurised to support the excavated tunnel and then provide a layer of lubricant. In this situation the pipes were theoretically buoyant within the tunnel bore; indeed, readings from the contact stress cells in the instrumented pipe indicated that contact between pipes and ground was mainly near the crown of the pipe.

At Site 3 the excavation through stiff glacial clay was undertaken with an open-face tunnelling machine. The overbreak varied somewhat along the drive as the cutter teeth tended to wear due to the presence of gravel and boulders in the clay, the value given in the table being an estimate for the position corresponding to the instrument array. The

overbreak was again completely filled with lubricating slurry.

In all three cases any overbreak remaining at the end of the drive was simply left to close onto and seal the outside of the pipe line, with no attempt to grout up the void.

3.2 Ground conditions and instrument arrays

The instrumented array used at each of the three sites are shown in Figures 3, 4 and 5 respectively, along with the ground conditions revealed by the site investigation and confirmed during installation of the instruments. The layout used reflected the financial restraints imposed, the time needed for reading the instruments, and individual local site conditions. Thus Site 2 was in a busy street and this limited the number of boreholes allowed

The measurements made at all three sites were of surface settlements by precise levelling of nails driven into the road pavement, and subsurface movements using inclinometers for measurement of horizontal displacements and magnetic plates for vertical displacements. Vertical movements of the tops of the inclinometer tubes were determined by precise levelling, while horizontal movements of the tops of the tubes were assumed to be negligible due to the stiffness of the road

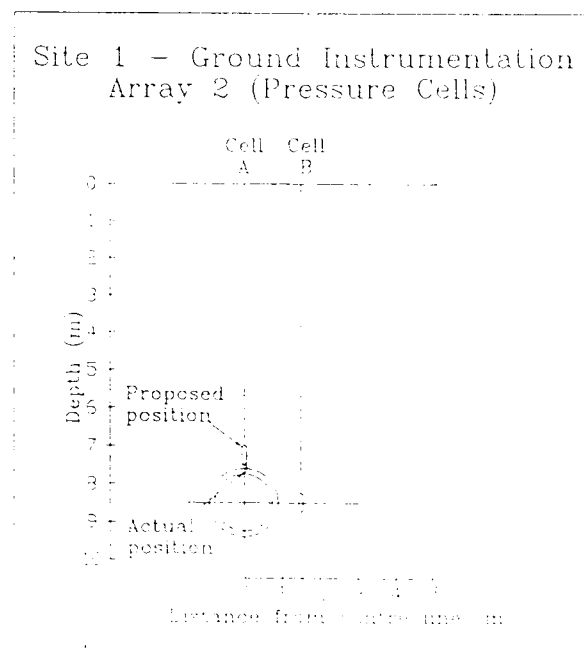
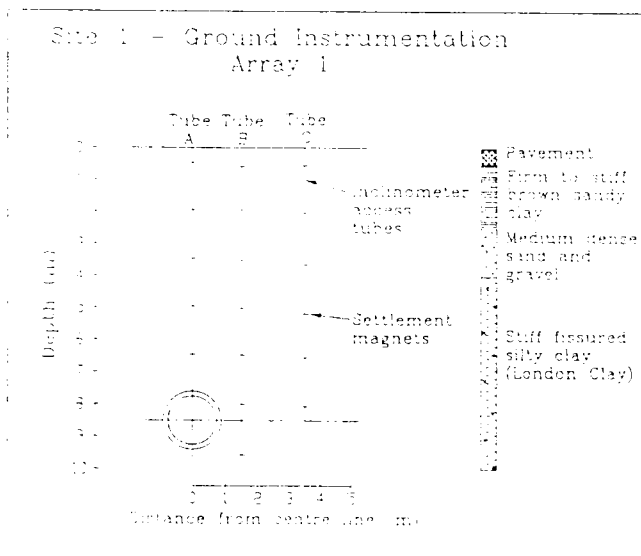


Fig.3. Instrument arrays at Site 1

pavement present in each case. The measured patterns of movements suggested that the latter assumption was in fact reasonable.

At each of sites 1 and 3 some additional instrumentation was used. At Site 1 this consisted of two spade cells with piezometers pushed in to the ground at the bottom of boreholes; the thin "blades" of the spade cells were aligned along the line of drive to measure ground pressures perpendicular to the line of the tunnel. Some pressuremeter testing was also done at this site to supplement the

normal site investigation and obtain information on the initial horizontal stresses in the London clay and the elastic stiffness of the clay.

At Site 3 the surface road nails were supplemented by more substantial settlement stations consisting of a length of steel bar cast into a concrete block at a depth of 950mm below the surface and sleeved between the top of the block and the road surface. In addition a set of electro-levels was used, in a near-horizontal tube grouted into a borehole just

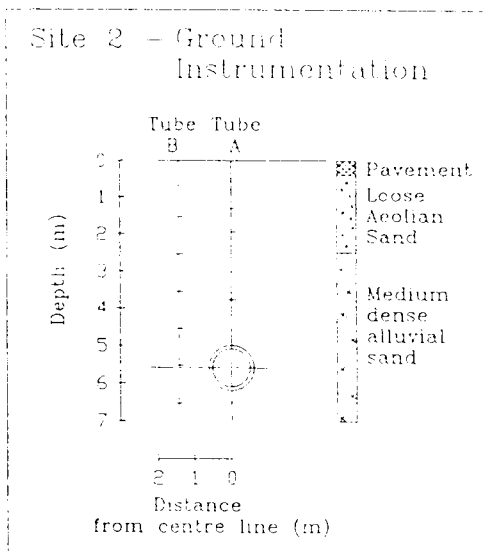
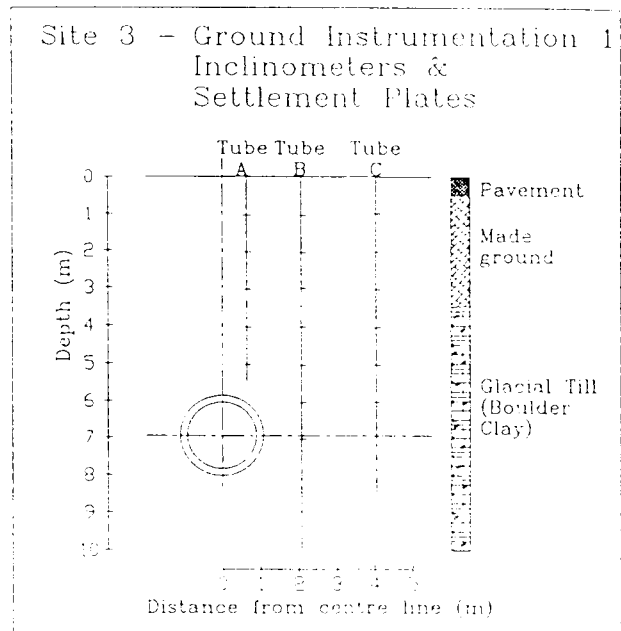


Fig.4. Instrument array at Site 2



Site 3 - Ground Instrumentation 2
Electro-levels

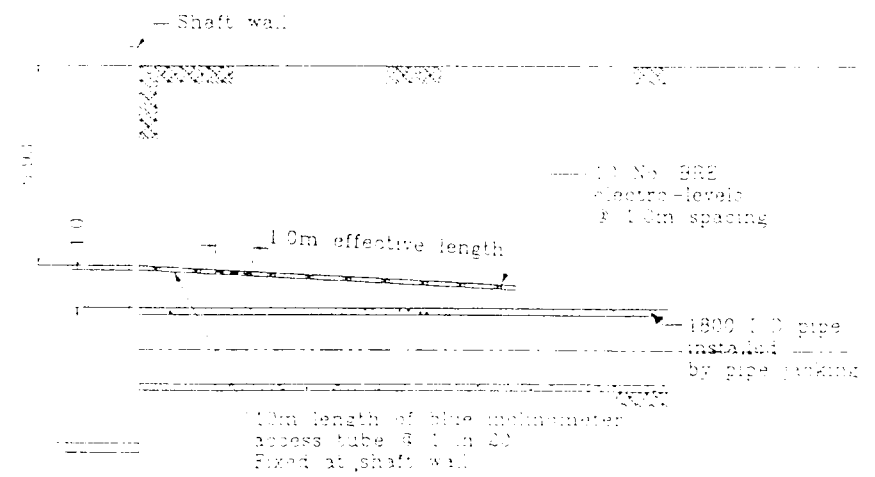


Fig.5. Instrument arrays at Site 3

above the crown of the tunnel at the reception shaft. The electro-levels operate like a series of short, continuously reading inclinometers, giving settlements by integration of the slopes measured at intervals along the tube.

4. RESULTS AND DISCUSSION

4.1 Site 1

The calculated settlements, including their variation with transverse distance from the centre line at the ground surface and their variation with depth on the centre line, are given in Table 2. From O'Reilly and New (1982) the value of i at the surface is 4.76m, assuming the ground to be cohesive (in practice there is a significant depth of sandy gravel above the London clay), and the maximum predicted surface settlement 12mm. Measured surface settlements are plotted in Figure 6 and compared with the calculated values. The measured values are somewhat erratic, and surprisingly show the centre line settlement to be rather less than to either side of the centre line; however they all lie within the calculated settlement profile, with a maximum settlement of 8-9mm.

Table 2. Site 1 calculated settlements

x (m)	0	1	2	3	4	5	6	8	10
S_0 (mm)	12.0	11.8	10.9	9.3	8.4	6.8	5.4	2.9	1.3
z (m)	0	1	2	3	4	5	6	7	
i_z (m)	4.25	3.93	3.60	3.28	2.95	2.63	2.30	1.98	
S_z (mm)	13.4	14.5	15.8	17.3	19.3	21.6	24.7	28.7	

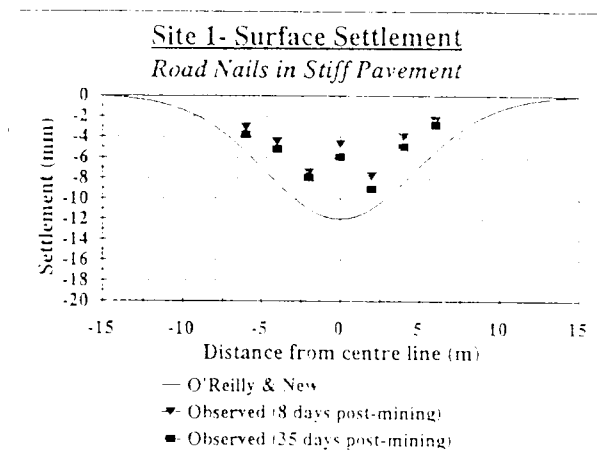


Fig.6. Site 1 Surface settlements

The prediction using Mair et al (1993) suggests that the settlement should approximately double between the surface and just above the crown of the tunnel. In practice the settlements indicated by the magnetic plates were never more than 3mm greater than measured at the surface, and most readings were within the accuracy for repeatable reading of about 2mm.

Inclinometer profiles from instruments 2 and 3 are plotted in Figure 7, showing maximum horizontal movements of 5mm perpendicular to the line of the tunnel at about the level of the tunnel. Horizontal movements of the centre-line inclinometer, and movements parallel to the line of the tunnel were all small, (<2mm) and somewhat erratic. Figure 8 shows information from the inclinometer and settlement plates combined to give ground movement vectors in the cross-sectional plane.

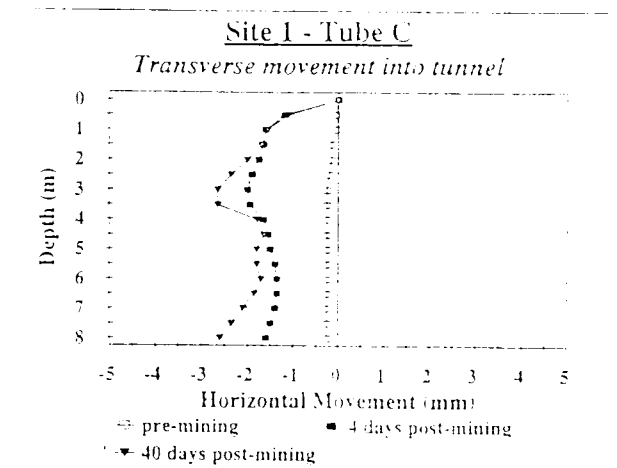
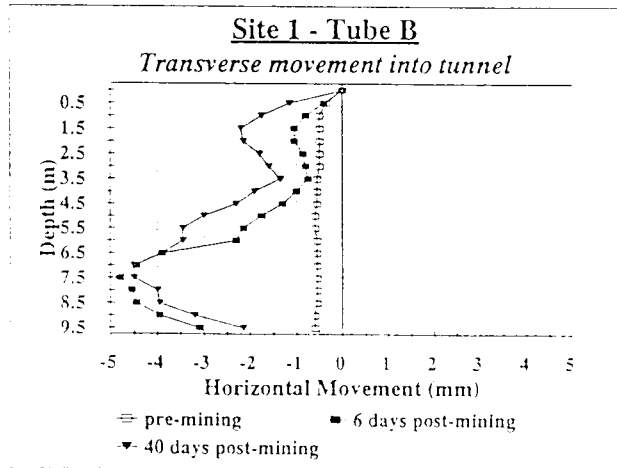


Fig.7. Site 1 inclinometer readings

In general the calculations provided a reasonable upper bound to the measured settlements; horizontal movements near the surface were small, but more significant (though still small) at the level of the tunnel.

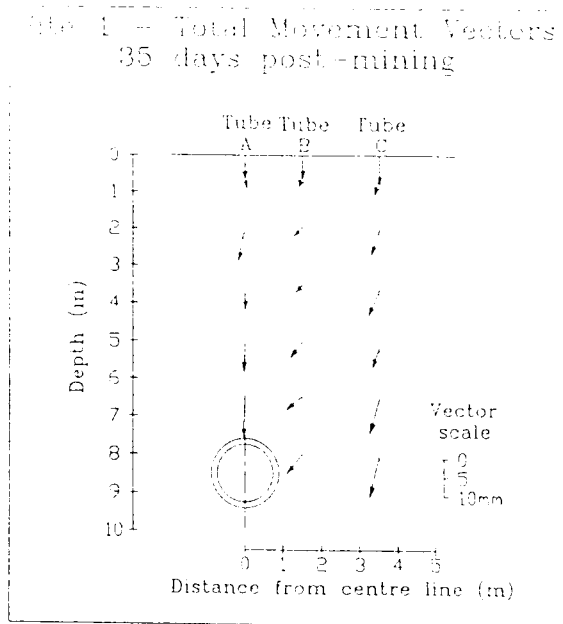


Fig.8. Site 1 movement vectors

The inward movement of the tunnel walls due to elastic stress relief may be calculated from the initial ground stresses and the insitu stiffness of the London clay. Results from the pressuremeter testing are presented in Table 3. At a depth of 8.5m the total vertical stress is assumed to be 190 kPa, while the measured total horizontal stress was measured at about 330 kPa; the initial shear modulus G_i averaged about 50 MPa.

Table 3. Results of pressuremeter tests

TEST	z (m)	P_0 (kPa)	s_u (kPa)	G_i (MPa)	G_{ur1} (MPa)	G_{ur2} (MPa)	P_l (kPa)
B1T1	6.4	287	94	90	18	14	997
B1T2	7.4	327	92	55	27	20	1001
B1T3	10.1	320	97	56	26	22	1012
B1T4	11.1	340	141	50	28	24	1121
B2T2	6.8	328	112	35	20	18	1050
B2T3	8.0	362	89	45	17	18	1011
B2T4	9.0	289	159	14	25	20	1124
B2T5	10.0	292	150	95	24	24	1189

p_0 = total horizontal stress G_i = initial shear modulus
 G_{urn} = shear modulus from unload/reload loop No. 'n'
 s_u = undrained shear strength P_l = limit pressure

The reduction in horizontal and vertical diameter are given by Poulos and Davis (1974) as

$$\delta_h = \frac{(1 - \nu^2)}{E} D (3\sigma_h - \sigma_v) \quad (9)$$

$$\text{and } \delta_v = \frac{(1 - \nu^2)}{E} D (3\sigma_v - \sigma_h) \quad (10)$$

where E and ν are Young's modulus and Poisson's ratio, $\nu = 0.5$ for undrained deformation and $E = 2G(1 + \nu) = 3G$. The resulting values in this case ($D = 1.85\text{m}$) are $\delta_h = 7.4\text{mm}$ and $\delta_v = 2.2\text{mm}$. A significant part, of the order of 3.5mm, of the horizontal movement close to the tunnel can therefore be attributed to elastic unloading.

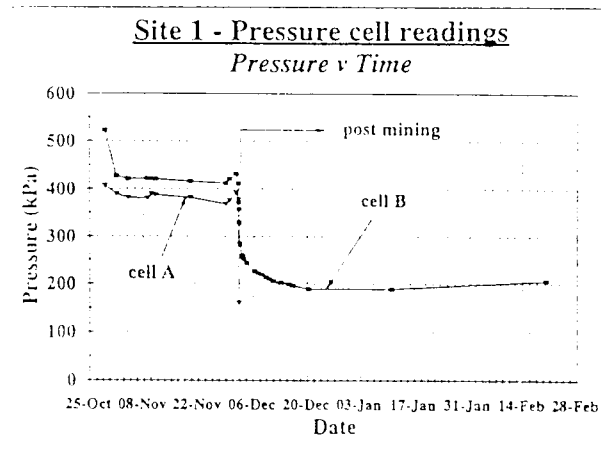


Fig.9. Site 1 pressure cell readings

The readings from the embedded stress cells are shown in Figure 9. Following installation, once the readings have stabilised, both instruments indicate horizontal pressures of about 400 kPa. It has been suggested (Tedd et al 1984) that for installation in stiff clay these cells over-read by about half the undrained strength of the clay, s_u . Here s_u (Table 3) is about 90-100 kPa, while the cell readings are about 70 kPa higher than the total horizontal stress from the pressuremeter tests. With the passage of the tunnelling shield the reading of the cell 1.5m from the axis of the tunnel dropped by about 220 kPa to about 215 kPa. The cell which was intended to be just above

the crown of the tunnel, and theoretically should have measured a substantial increase in stress, was in fact exposed in the tunnel face having been installed at the wrong level, its reading dropping by about 240 kPa to a final reading taken of 160 kPa. It is suggested that these readings are best interpreted as showing an initial horizontal stress of about 350 kPa, slightly higher than given by the pressuremeter. It is interesting to note that both cells started to show increases in stress as the tunnel face approached, presumably due to the soil arching in both horizontal and vertical planes around the front of the tunnel excavation.

4.2 Site 2

The surface and subsurface settlements for Site 2 are shown in Figures 10 and 11 respectively. None of the horizontal displacements measured by the inclinometers exceeded 1.5mm, nor did they show any consistent pattern. The calculated value of i was 1.45m, and the maximum settlement 18mm assuming that the full overbreak volume between the cutter diameter and the pipes' external diameter is converted into surface settlement. However the maximum surface settlement is only about 6mm, though the distance to the point of inflexion of the settlement profile is in good agreement with the prediction. Two main factors probably account for the discrepancy in the magnitude of settlement. Firstly, shearing of the soil during settlement will cause some dilation of the dense sand, so that the volume of the settlement trough will be somewhat smaller than the volume of the overbreak. Secondly, the use of bentonite slurry support and lubrication prevented the collapse of the ground onto the pipes. An alternative calculation assuming that the overbreak volume is given by the difference in diameter between the cutter and the shield gives a maximum settlement of 8mm, with the same value of i ; allowing for some dilation this is in reasonable agreement with the measured value.

If this hypothesis is correct, it leaves some uncertainty over whether further settlements will occur in the long run as the annulus of bentonite slurry, presumably mixed with sand during the jacking operation, gradually consolidates under pressure from the

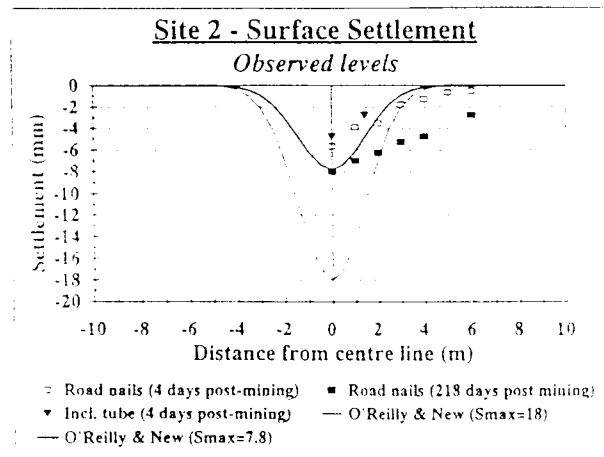


Fig.10. Site 2 surface settlements

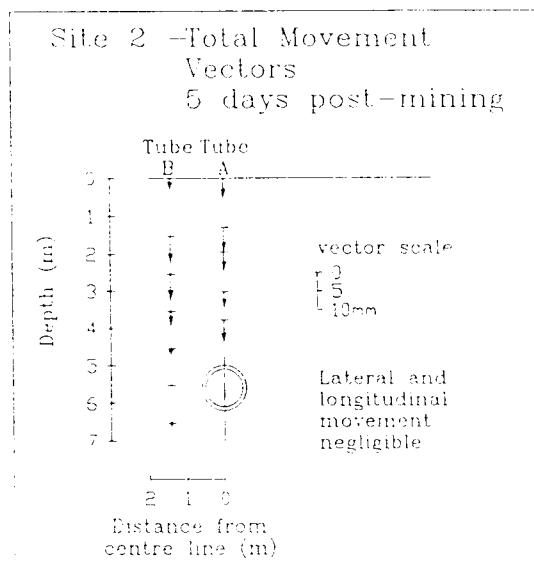


Fig.11. Site 2 movement vectors

surrounding ground. In this case the settlement increased after about seven months to give a maximum of 8mm; the profile is much flatter than expected, probably due to the stiff road pavement.

4.3 Site 3

Surface settlement measurements from Site 3 are shown in Figure 12, while calculated values are given in Table 4 and also plotted in Figure 12. Again two values of $S_{o, max}$ have been determined, the value of 13.3mm resulting from the closure of the full overbreak between cutter and pipes, the lower value of 10mm from the closure between cutter and shield, assuming that the remaining annulus is held open by the pressure of bentonite slurry.

Both values are larger than measured, though the shape of the settlement trough is again well predicted. The settlements measured on the road nails were unexpectedly somewhat greater than on the settlement points founded beneath the pavement.

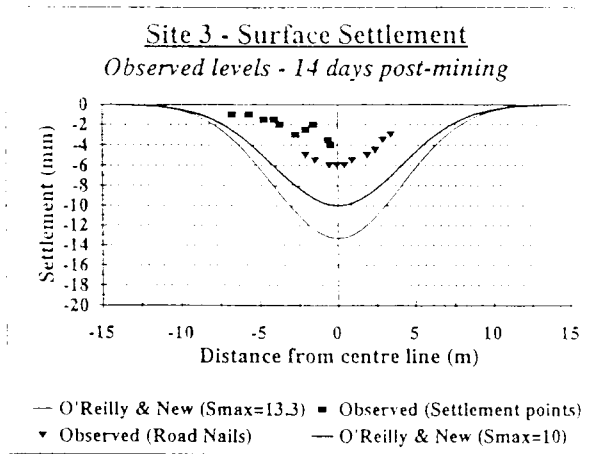


Fig.12. Site 3 surface settlements

Table 4. Site 3 calculated settlements

x (m)	0	1	2	3	4	5	6	8
S_0 (mm)	13.3	12.9	11.8	10.1	8.2	6.3	4.5	2.0
z (m)	0	1	2	3	4	5	5.5	
i_z (m)	3.47	3.15	2.82	2.50	2.17	1.85	1.53	
S_z (mm)	15.7	17.3	19.4	21.8	25.2	29.5	35.7	

Inclinometer plots for the transverse deflections of the tubes to the side of the tunnel are shown in Figure 13; small movements towards the excavation were detected, reaching a maximum at about the level of the tunnel but still significant for some depth below it. Movements over the centre line, and in the direction of drive for all three tubes, were very small.

Figure 15 shows profiles of settlement just above the crown of the pipe and along its centre line, obtained from the electro-levels. These show a maximum settlement of about 10mm, greater than measured by the settlement plates but still much less than calculated for this depth. Settlement starts to be just detectable about 1.5m ahead of the tunnel face, but most of the settlement occurs after the passage of the cutting head.

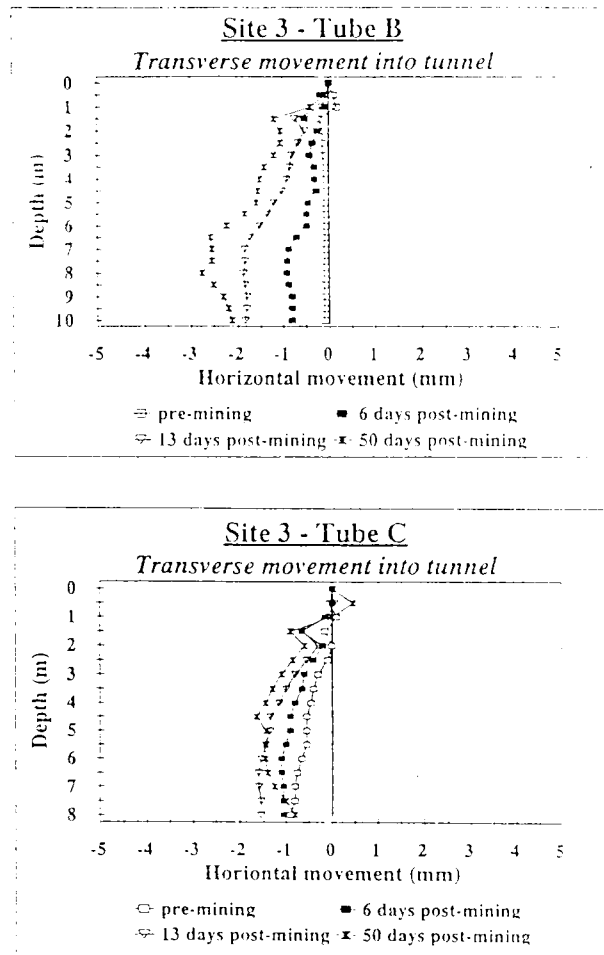


Fig.13. Site 3 inclinometer readings

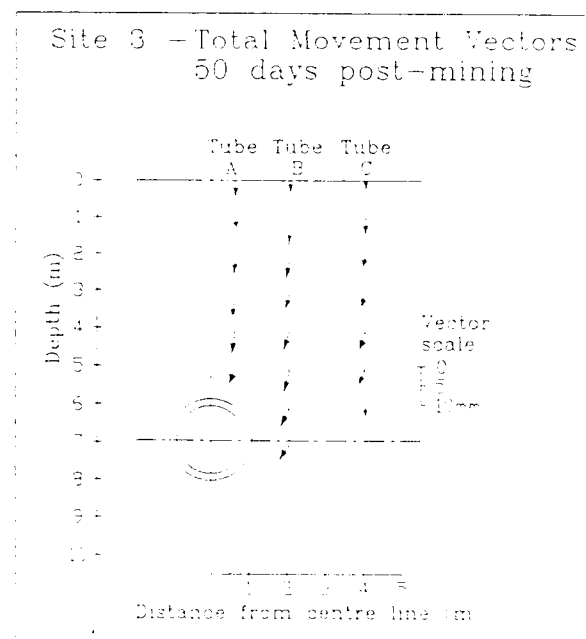


Fig.14. Site 3 movement vectors

This confirms that ground loss due to inward movement of the tunnel face is negligible. Settlements had almost ceased by 10 days after completion of the drive, as shown for a point some 7m from the shaft wall.

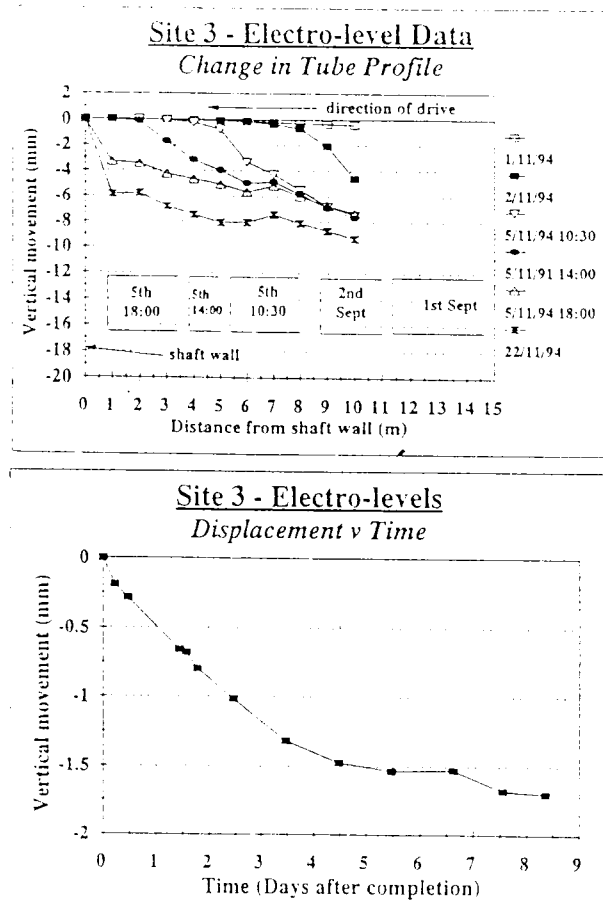


Fig.15. Site 3 electro-level readings

5. CONCLUSIONS

The ground movements measured on three sites during construction of sewer pipes by pipe jacking have all been very small. In very stable ground conditions, or if face pressure is accurately balanced, the ground loss due to movement of the tunnel face appears to be negligible. The well-known empirical approach to the calculation of settlements has in each case predicted a reasonable profile of surface settlements. However the effective ground loss seems to have been smaller than the full volume of the overbreak between excavation and pipe external diameter, and the calculated maximum settlements are in all cases greater

than measured. Additional sub-surface settlements have been very small, barely detectable using magnetic settlement plates; however electro-levels were used successfully to obtain a longitudinal settlement profile immediately above the tunnel face on one site.

6. ACKNOWLEDGEMENTS

The sites and some of the costs of ground instrumentation were provided by Thames Water, North West Water and Northumbrian Water. Instrumentation arrays were installed by Soil Instruments Ltd, Georesearch and Exploration Associates; the electro-levels were supplied and installed by the Building Research Establishment; pressuremeter testing was performed by Cambridge Insitu.

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Site-based research in pipe jacking—objectives, procedures and a case history

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This paper provides an introduction to the programme of field monitoring of pipe jacking undertaken at Oxford University, as part of a major research programme initiated and funded by industry in collaboration with the Engineering and Physical Sciences Research Council.

In the first half, the need for research is discussed, the objectives of the work introduced, and the reasons for the use of site monitoring to achieve these objectives presented. The instrumentation developed for this purpose, and the methods of planning and execution of the site work, are described, and brief details given of the five schemes monitored during the first stage of fieldwork.

The second half of the paper is a case history of the first instrumented scheme, involving a pipe jack at shallow depth in stiff glacial clay. Results presented include measurements of: total jacking force; local shear and normal stresses and pore water pressures at the pipe-soil interface; deviations in line and level of the pipe line, and consequent misalignment angles at the pipe joints; and the magnitude and distribution of joint pressures between pipes.

The instrumentation generally performed very well, so that only minor modifications were needed for later schemes. Pipe-soil interaction was found to be complex, affected by ground type, excavation technique and the presence of free water within the tunnel bore; however, two simple theoretical models were found to give calculated values for pipe jacking resistance in reasonable agreement with measured values. Pipe line alignment was shown to be very important in determining pipe barrel load paths and pipe joint pressure distributions. No evidence was found for any significant changes to alignment either due to continued movement of the pipes, or to pipe line straightening under load. The alignment was thus defined by the accuracy of steering of the tunnelling shield.

INTRODUCTION

A programme of research in pipe jacking has been in progress at Oxford University since 1984. It has involved laboratory testing, finite element computer analyses and site monitoring on live construction sites. It has been supported by both client bodies (water companies) and contractors from industry, and by government funding via the research councils. It provides an outstanding

example of co-operation between industry and academia.

This paper has two main parts. The first outlines the history, objectives, funding and management of the overall project, and then leads on to a review of the need for site measurements and the approach adopted to ensure the success of this work. The second part provides a full case history of the first site monitoring operation. Detailed case histories of the other instrumented schemes will be pre-

sented in future papers, whilst a summary of the main results and recommendations from the first five sites is given by Milligan and Norris.¹

point of view and allowing research to take place on active sites under their control.

PIPE JACKING RESEARCH

History

Pipe jacking has been used in the U.K. for some 30 years, and much practical experience and development has gone into the design of equipment and techniques. However, a basic scientific understanding of the complex interactions taking place between pipe and ground was lacking, and unexpected failures were not unusual, particularly as the technique has been applied to ever longer drives in more difficult ground conditions. The specialist contractors, in the form of the Pipe Jacking Association (PJA), realised that it was necessary to put the technique on a proper footing by increasing understanding through research, if the major clients such as the (then) Water Authorities were not to lose confidence in it.

Funding and management

The overall costs and funding arrangements for the four stages are also given in Table 1. In addition to the direct costs shown, substantial research assistant expenses in Stage 1 were carried by the PJA and CPA. In Stages 2 and 3 each site operation has incurred expenditure of between £10,000 and £20,000; each water company has borne this cost for one site in each stage.

The research programme is overseen by a management group which from Stage 2 onwards has consisted of two representatives from the PJA, one from Oxford and one from each of the water companies. The research team reports on technical and financial progress to this group at quarterly meetings, and in return receives much useful guidance, assistance and encouragement.

Objectives

The research has set out to provide a basic scientific understanding of the process of pipe jacking. This should allow improvements in design, specification, estimating, site operation and control, resolution of claims (or preferably avoidance of the situations that lead to them), and perhaps even the costs of insuring works. In the long term, this should lead to more economical solutions, safer construction techniques, and a superior end-product. Both clients and contractors should benefit, hence it is entirely appropriate that they have joined forces to support the research.

The general objectives for the research programme were the areas listed by Craig² as requiring investigation:

- (a) prediction of friction loads in different ground conditions;

Table 1. Stages of research work

Stage	Dates	Nature of work	Total cost (£)	Supported by	Res. assistant
1	1986-1989	Laboratory model testing	40,000	SERC ³	Kevin Ripley
2	1989-1992	Monitoring of five pipe jacks	220,000	SERC, PJA ² , Water Cos. ⁴	Paul Norris
3	1992-1995	Monitoring of five pipe jacks	310,000 ⁵	SERC EPSRC ³ , Water Cos., PJA	Mark Marshall
4	1993-1996	F.E. modelling of pipes	100,000 ⁵	SERC EPSRC	Fan-fang Zhou

¹Science and Engineering Research Council.

²Pipe Jacking Association.

³Northumbrian, North West, Severn Trent, Thames and Yorkshire.

⁴Engineering and Physical Sciences Research Council.

⁵For stages 3 and 4, costs included overhead charges on research council grants.

- (b) load deflection characteristics of the joints with different packing materials;
- (c) the effect of cyclic loading on the pipes at intermediate jacking stations;
- (d) the effect of lubricants in reducing friction along a pipe;
- (e) the development of a site investigation test suitable for the prediction of frictional forces.

The measurement and prediction of ground movements associated with pipe jacking has subsequently been added to this list of objectives.

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 project is therefore to investigate the load transfer through and between pipes, and the contact stresses between pipes and soil.

In more detail, these objectives require the determination of the areas of contact between pipe and ground in both granular and cohesive soils, the normal and shear stresses developed at these contacts, the effects of lubrication, the stresses and strains in individual pipes, the effects of angular deviations of joints, and the performance of the packing material in the joint. Some of these could be studied in the laboratory investiga-

tion, but needed confirmation at full scale, while other effects could only be achieved in the field as they depended on realistic ground conditions and construction procedures.

Instrumentation

Special instrumentation was developed to meet these objectives, taking full consideration of the difficulties and constraints involved in working in the often wet, dirty and confined environment of a pipe jacked tunnel. The instruments were designed to be:

- easy to install, with minimum delay to site work
- able to be read remotely
- cheap enough to allow sufficient numbers to be installed
- robust and reliable
- sufficiently accurate
- reasonably simple to calibrate
- having minimal effect on the behaviour of the pipe in which they are installed.

A full set of instruments is shown in Fig. 1. They were arranged in three groups, in the lead pipe, in the main instrumented pipe and at the jacking pit. For drives through cohesive soil, the lead pipe contained a ground convergence indicator which was designed to measure the rate at which the ground closed on to the pipe, particularly when jacking was halted overnight or at weekends. It consisted essentially of a hinged arm spring-loaded against the ground and connected to a rotary potentiometer.

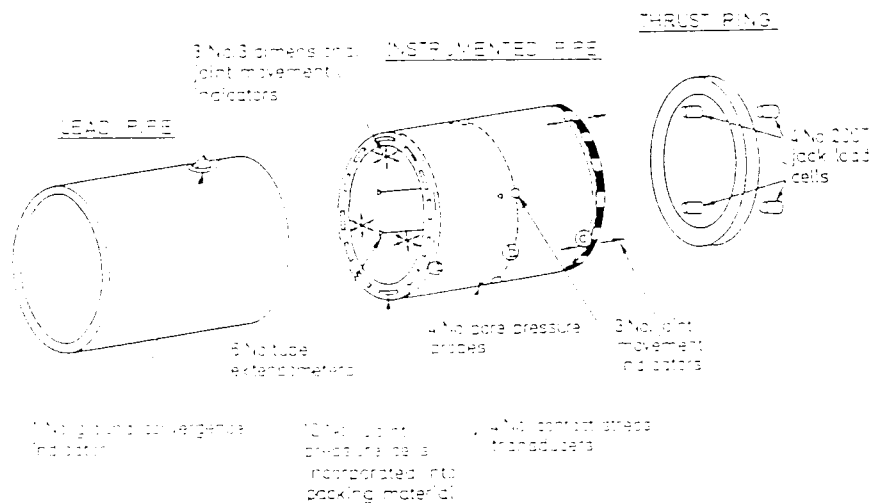


Fig. 1. Schematic of instrument arrangement.

The main instrumented pipe was located further back in the pipe string and contained the following instruments:

- (i) four contact stress cells, to measure both total radial and shear stresses on the surface of the pipe; their active face was flush with the pipe surface and provided with a similar surface to the pipe;
- (ii) four pore pressure cells adjacent to the contact stress cells, measuring the local pore water pressure and hence allowing determination of the effective radial stress;
- (iii) three joint movement indicators at each end of the pipe, to measure the three-dimensional angular misalignment of the joint gap;
- (iv) up to twelve pressure cells built into the packer in the joint at either end of the pipe, to measure the magnitude and distribution of the stresses transferred across the joints;
- (v) up to six extensometers fitted to the internal surface of the pipe and equally spaced around it to measure the compression of the pipe under load.

The main instrumented pipe also contained a modular data acquisition system and stable power supply.

In the jacking pit the total jacking load was recorded by two or four load cells positioned between the jack rams and the thrust ring, and the forward movement of the pipe string measured by a displacement transducer mounted above the tunnel entrance. A flow chart summarising the objectives of the instrumentation is shown in Fig. 2. Readings from the instruments were backed up by a detailed log of all site activities, and regular surveys of line and level of the full length of the tunnel. Additional observation, sampling and testing of the ground conditions were also undertaken to supplement site investigation data. Details of the design, construction and calibration of the instruments are summarised by Norris and Milligan³ and covered in detail by Norris.^{4,5}

Data acquisition

The data acquisition system is shown diagrammatically in Fig. 3. The system employed a family of

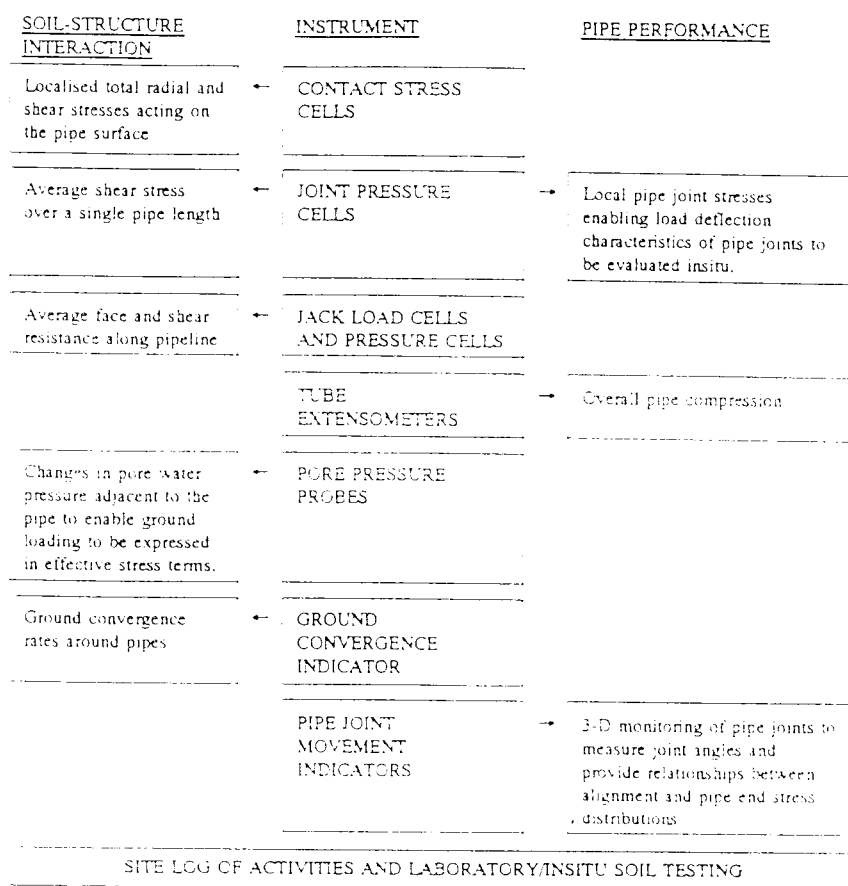


Fig. 2. Objectives of the instrumentation.

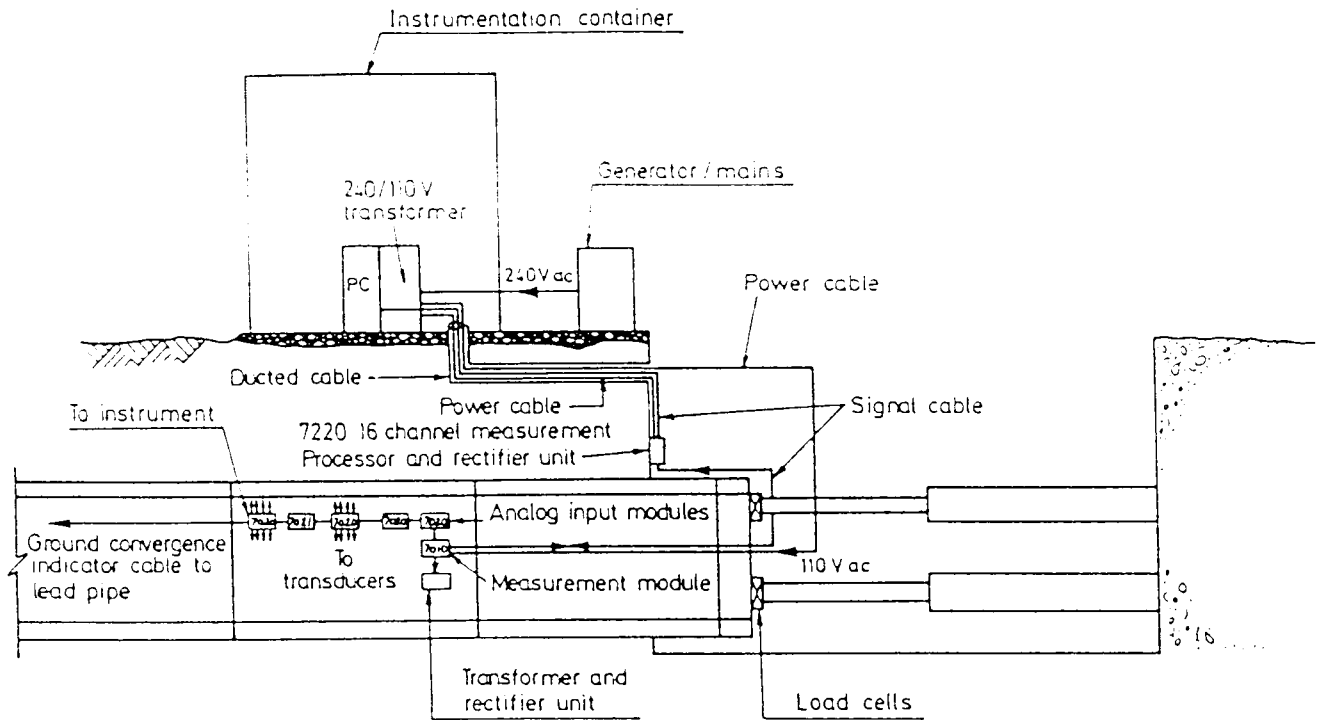


Fig. 3. Schematic of data acquisition system.

commercially available Datascan control and analogue input modules capable of accepting information from different types of transducer. They were housed in protective steel boxes within the instrumented pipe, and connected by short lengths of analogue signal cable to the various instruments. The signals were converted to digital form for communication between the modules and the 286-AT personal computer controlling the system from the surface, to minimise the risk of signal corruption. The system could handle up to 80 channels. Each control module contained a non-volatile memory which retained the set-up information during power loss, such as when the power and signal cables were disconnected when each new pipe was added to the string.

Readings were taken at 5 s intervals during pushes of the pipe line, and 1 min intervals at all other times. Data were regularly backed-up on to floppy disks, and later analysed using Lotus 123 spreadsheets.

Site selection

It was intended that five schemes would be monitored during Stage 2, with one being provided by each of the sponsoring water companies. It was expected that a number of sites would be offered, with sufficient lead time for the research activities to

be built into the contract documents, from which those most suitable for the research would be selected. In practice, due to the effects of privatisation of the water industry, few suitable sites became available and most had to be accepted at short notice; details of the five schemes monitored are given in Table 2. The second half of this paper covers the first of these in detail, while the other schemes will be the subject of future papers.

Planning and execution of the field work

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 work in such a way that 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 items to be added to the bill of quantities is given in Table 3. 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

Table 2. Details of schemes monitored

	1	2	3	4	5
Date	August 1990	January 1991	March 1991	July 1991	December 1991
Location	Bolton, Lanes	Gateshead, Tyneside	Honor Oak, S.E. London	Chorley, Lanes	Cheltenham, Glos
Utility	N.W. Water, Bolton M.B.	Northumbrian Water	Thames Water	N.W. Water, Chorley D.C.	Severn Trent Water, Cheltenham B.C.
Consultant	—	—	Binnie Taylor Woodrow	Halerow	—
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	85 m 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.

*Monitoring only for part of drive.

†MDF = Medium Density Fibreboard.

Table 3. Main research items for bill of quantities

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 100 x 12 mm 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

items were all activities which could have a major disruptive effect on the contractor's programme of work. Unforeseen delays caused by the research were covered by the rate for standing by.

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, and deliver the pipes to site with sufficient time to complete installation.

On site, the first priority was to install the jacking pit instruments. The jack load cells were coupled to the rams, and the Celesco displacement

unit mounted above the tunnel entrance. 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 preparation of the instrumented pipe was started. 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. The total time for assembly and system checking was 2-3 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.

Protection of the instruments and data acquisition system against mechanical damage was provided by a steel liner which fitted inside the instrumented pipe (Fig. 4). The liner was typically

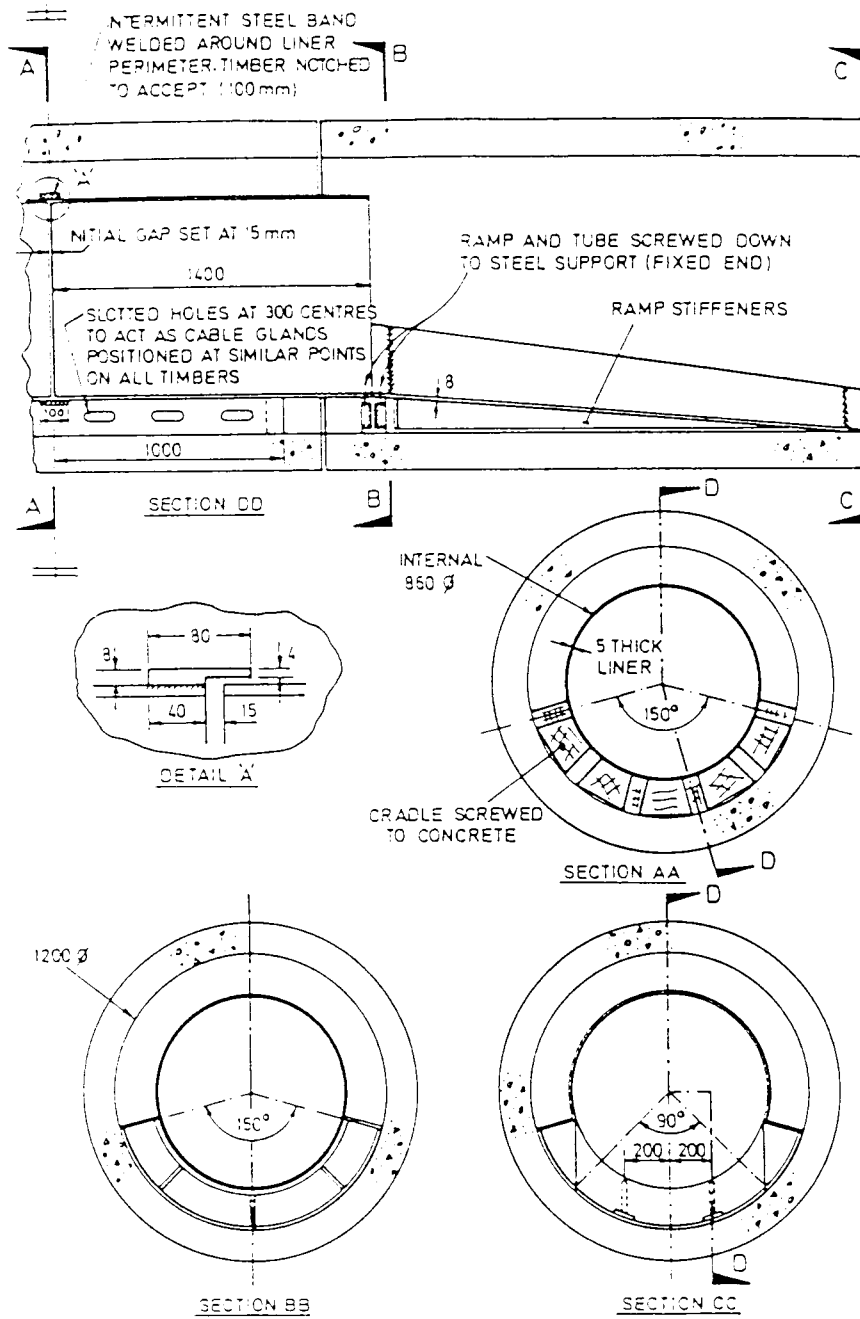


Fig. 4. Details of protective liner.

300 mm smaller in diameter than the pipeline, and was fabricated in two 1.4 m lengths, which was slightly longer than the 2.5 m instrumented pipe length. Each overhanging portion was bolted to a steel cradle fastened to the leading and trailing pipes. The liner was supported in the pipe by a timber cradle in the pipe invert. Articulation of the liner was provided by a central steel banded joint which allowed the two halves to slide ± 10 mm relative to each other; this also ensured that none of the axial load on the pipe was transmitted

through the liner. A set of ramps in the leading and trailing pipes allowed muck skips to travel through the liner. A delay of about 6 h occurred while the liner was manoeuvred into position in the tunnel. 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 the liner provided was well worth the slight reduction in productivity that it caused.

All the instruments were recovered at the end of

a drive for re-use. The jacking pit instruments were unbolted from their anchorage points, then the liner was dismantled and the surface mounted extensometers, 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. Removal of the pressure cells from the pipe joints required special anchorage brackets to be bolted to the pipes on either 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.

Once monitoring was in progress, good communications 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 by providing precise and continuous records of jacking loads which helped with decisions on whether or when to lubricate or use an interjack.

A CASE STUDY OF AN INSTRUMENTED DRIVE—SCHEME 1

Introduction

Scheme 1 was set up as a pilot test for the instrumentation and site procedures; the funding bodies were warned that failures were likely and that useful data might not be obtained. In fact it was completed successfully.

The client, Bolton Metropolitan Borough Council acting as agents for North West Water Ltd, had designed the Bury Road Resewerage Scheme to alleviate foul flooding which was affecting premises in the Brightmet 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 despite the shallow cover of 1.5 m; 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.

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 5 m behind the shield. This enabled the first few pipes to be pushed out of the tunnel into the reception pit and the main instrumented pipe to be jacked into the tunnel again for a distance of two pipe lengths, to investigate any differences in behaviour at the front and rear end of the pipe string.

Ground conditions

A longitudinal section (Fig. 5) constructed from the face log taken during the construction period, illustrates that the drive was predominantly through stiff glacial clay. Soil strength and classification details were obtained from *in situ*

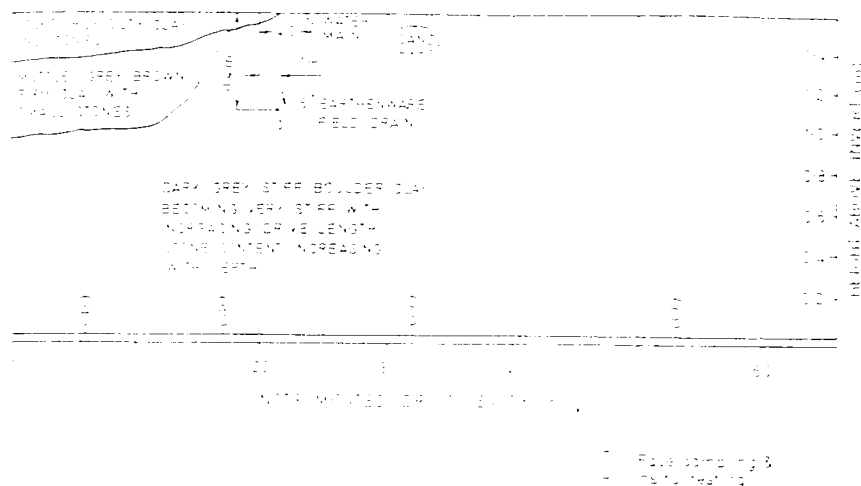


Fig. 5. Longitudinal section along tunnel centre line from face logs

Table 4. Soils test data from tunnel face

		CH 6 m 1	CH 17.5 m 2	CH 32 m 3	CH 53 m 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	w _p	40— 30— 20— 10—			
		— LL — PL	— LL		
Undrained shear strength	S _u	(3&5) 150 kPa	(2,4,5) 117 kPa [200]	(2,4,5) 8.5 kPa [245]	(2,4,5) 136 kPa [305]
	φ _u	0°	17° [0]	30° [0]	27° [0]
Consolidated undrained	c'	(2,4,5) 16 kPa	(1,3,5) 0 kPa	(1,3,5) 6 kPa	(1,3,5) 25 kPa
	φ'	30	39	30	34
Pocket penetrometer	Av (kPa)	72 180 169	82 196 210	132 169 220	240 259 258
Shear Vane	Av (kPa)	52 136 147	54 160 176	70 149 182	188 188 188

and laboratory testing of material at four chainages. A summary of the results appears in Table 4 which indicates that the undrained shear strength increased from 150 to 300 kPa 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.

Instrumentation

As this was a pilot test, a reduced set of instruments was used (see Table 5). In particular, only six joint pressure cells were fitted at each end of the instrumented pipe, and joint gap measurements were only made across one joint. The

pipeline displacement transducer was only introduced after this project.

The careful selection and design of the instruments led to few failures in service (Table 5). 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 suffered from problems due to ground water ingress, but this was overcome on later schemes by redesigning the primary seals. The pore pressure probes were found to be acceptable in cable damage during extraction, and the ground convergence indicator performed well in the field. Trouble was caused by the ingress of fine particles past the PTFE seals, causing jamming.

Table 5. Field reliability of instruments

Instrument type	Number	Failures cause
Contact stress cell	4	2 moisture ingress
Pore pressure cell	2	—
Pipe joint pressure cell	12	—
Joint movement indicator	3	—
Tube extensometers	3	—
Jack load cells	2	—
Temperature probes	4	—
Ground convergence	1	1 jammed by fine particles
Data acquisition system	42 channels	—

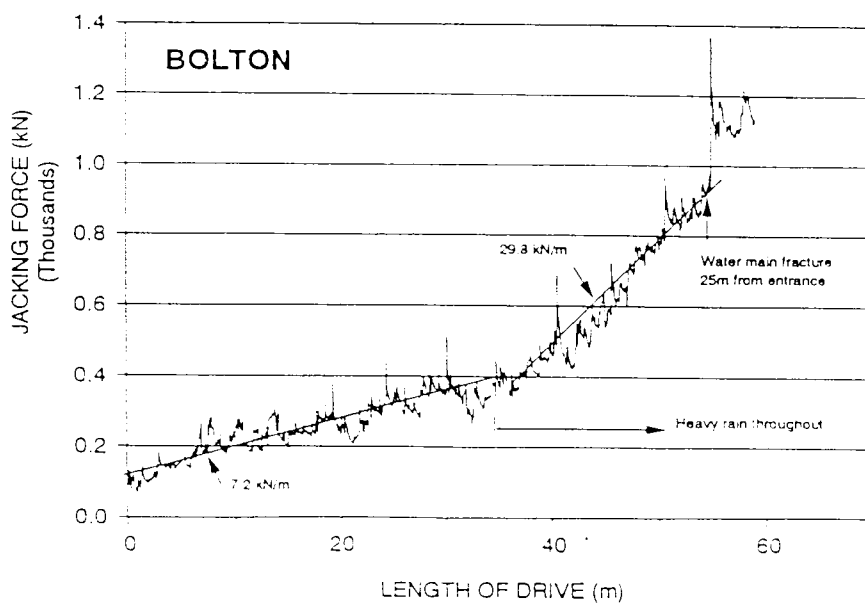
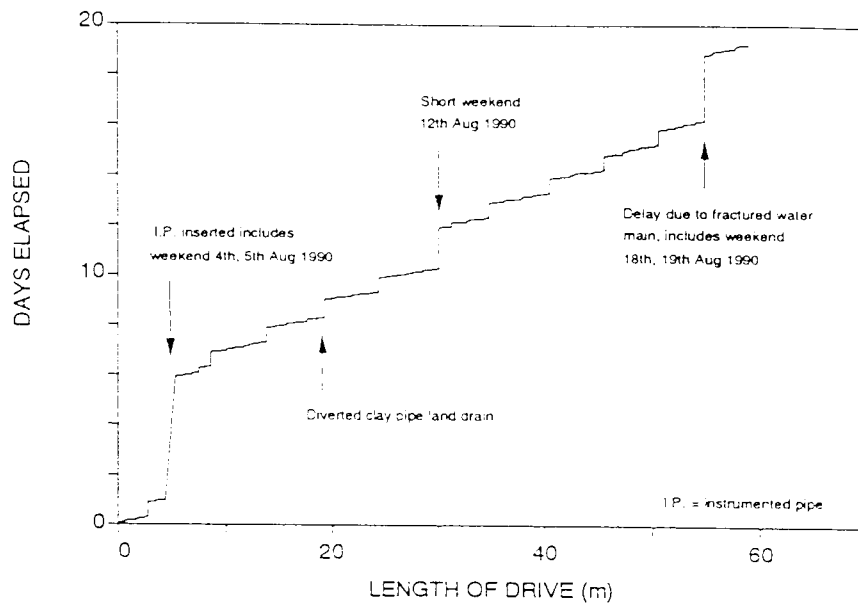


Fig. 6. Jacking records.

Constraints of programming or ground conditions have in any case prevented this instrument from being used after the first scheme.

Jacking records

The jacking records for the scheme are shown in Fig. 6. A prominent feature is the change in average jacking resistance at chainage 35m from a uniform rate of 7.2 to 29.8 kN m⁻¹. 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 6 m per shift was achieved. The jacking force increased from an initial face resistance of 130kN 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.

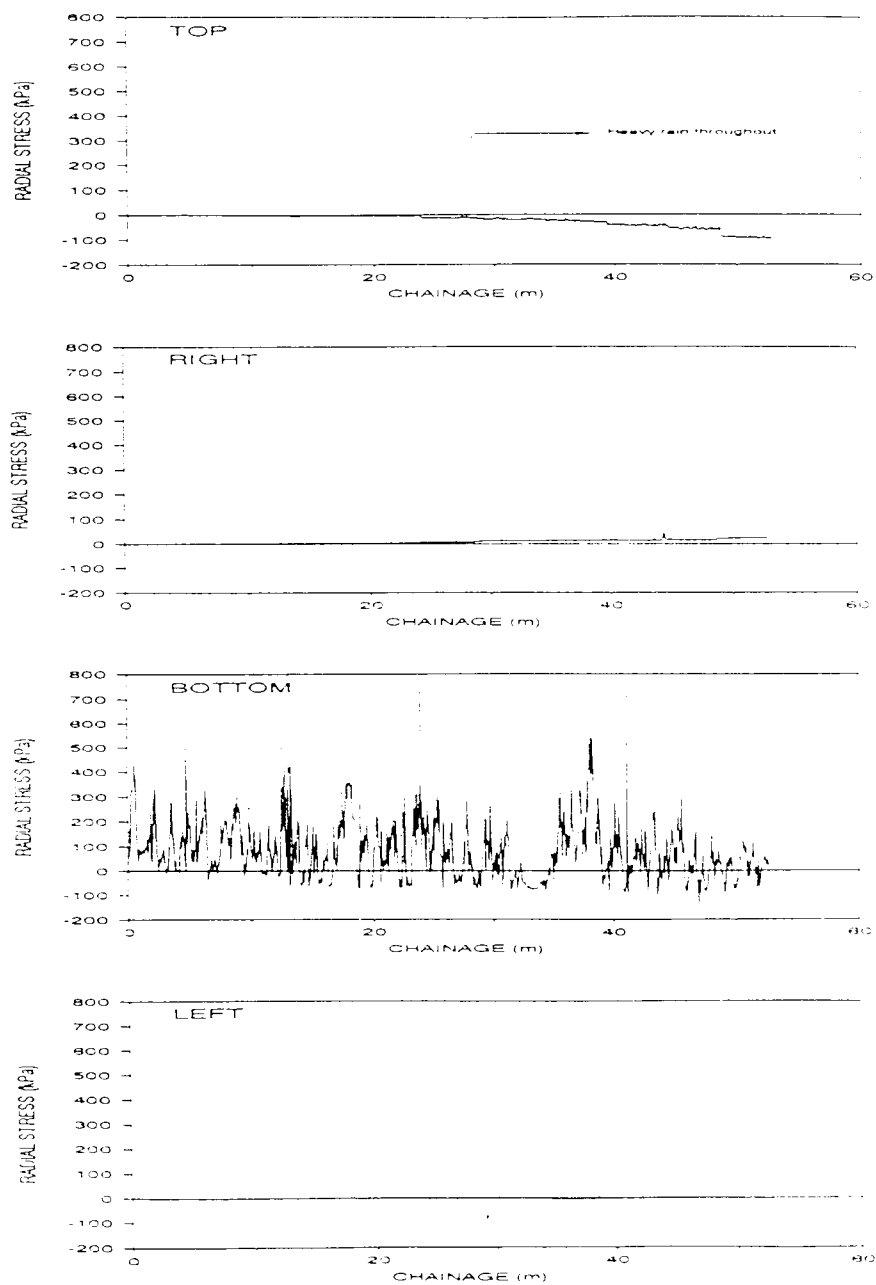


Fig. 7. Variation in total radial stress.

Jacking records are generally the only information on ground related loading that a contractor has at his disposal. They allow values of nominal face resistance and average pipeline friction to be determined in different ground conditions. To improve on 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 instrumented pipe wall allowed such measurements to be made.

Local interface stresses mobilised during jacking

The contact stresses measured by the stress cells are shown in Figs 7 and 8. The readings are affected by zero drift, due to moisture ingress. The instrumented pipe was positioned three pipes behind the shield resulting in the excavation being generally

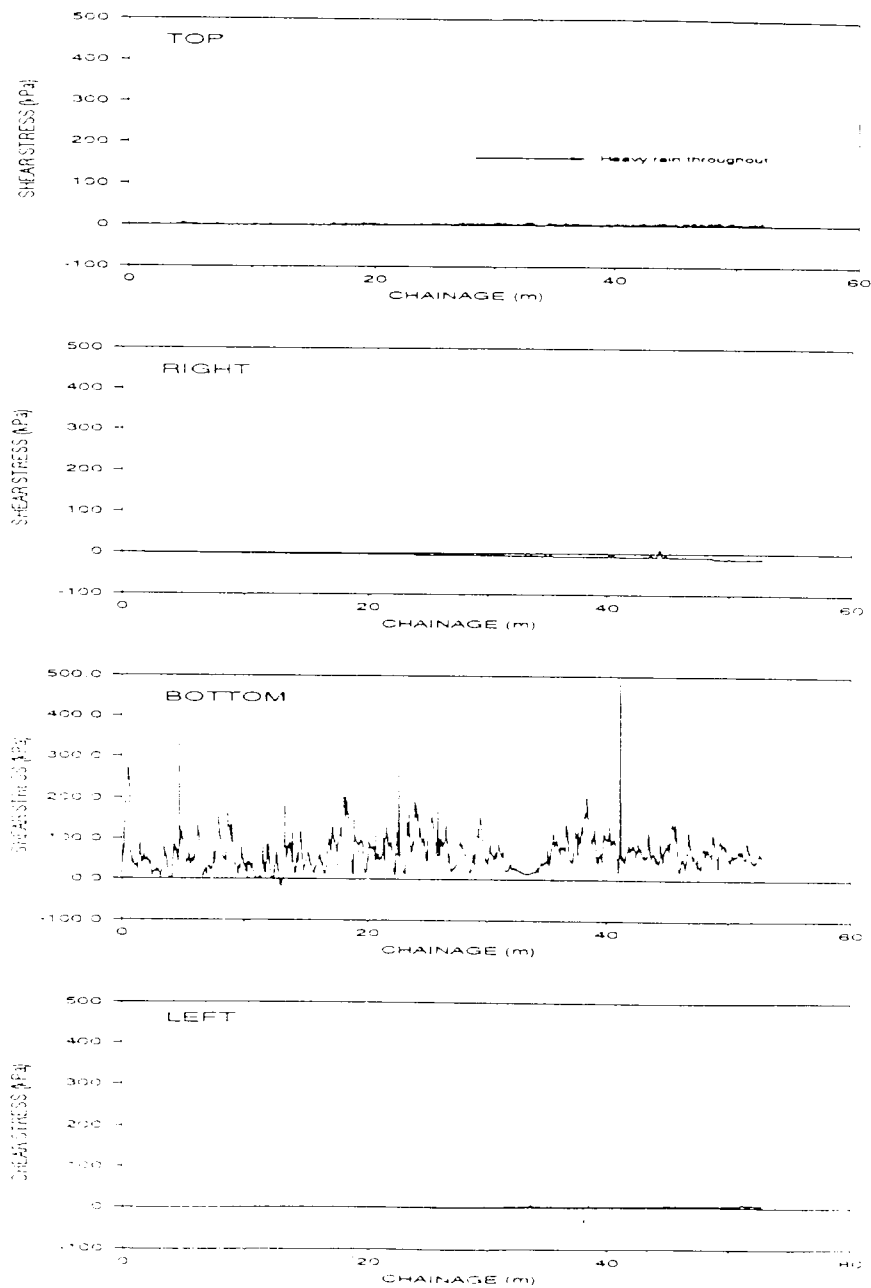


Fig. 8. Variation in interface shear stress.

1 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; this is consistent with the over-break in stiff cohesive soils at low cover depths remaining open, and the pipe sliding along the base of the open bore. The total radial stress varied between -120 and 750 kPa. These values were for extremely short periods with the majority of readings lying in the range 0 – 250 kPa. The larger values were probably the result of local cobbles pressing on to 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. Shear stresses varied between -40 and 480 kPa. The negative stress was an isolated value and was possibly due to recoil at the end of a push. The majority of shear stresses were between 0 and 150 kPa.

The pore water pressures (Fig. 9) during the dry weather conditions were generally smaller than the changes in total stress. This suggests that the ground at the interface may be unsaturated even though the original material was saturated, and/or the relatively high initial 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 at the pipe interface being greatly diminished by the excess water.

The compressed nature of the interface stress plots makes it difficult to evaluate the ground behaviour during a single push. Results from typical pushes are presented in Fig. 10. A number of observations may be made:

- There is excellent correspondence between the responses of radial, shear and pore water pressure profiles during pushes, providing increased confidence in the validity of the readings.
- Radial stresses vary rapidly over short distances of less than 300 mm and are probably a function of local variations in excavation profile.
- In the drier ground conditions the pore pressures are relatively small; after soaking of the ground, greater pore pressure variations occurred and significant suctions were measured. Pore pressures tended to decrease with increasing shear stress, but respond positively to increases in radial stress.
- The stress paths in the drier ground conditions suggested frictional behaviour under increasing stresses, but "adhesive" behaviour of the interface (constant shear stress) under reducing stress. Adhesive behaviour appeared to dominate in the wetter ground conditions.

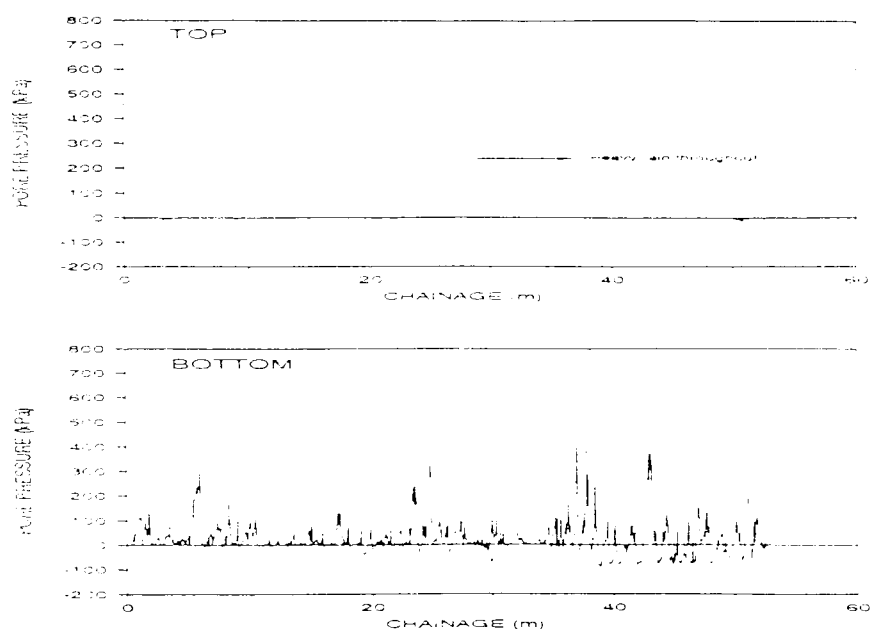
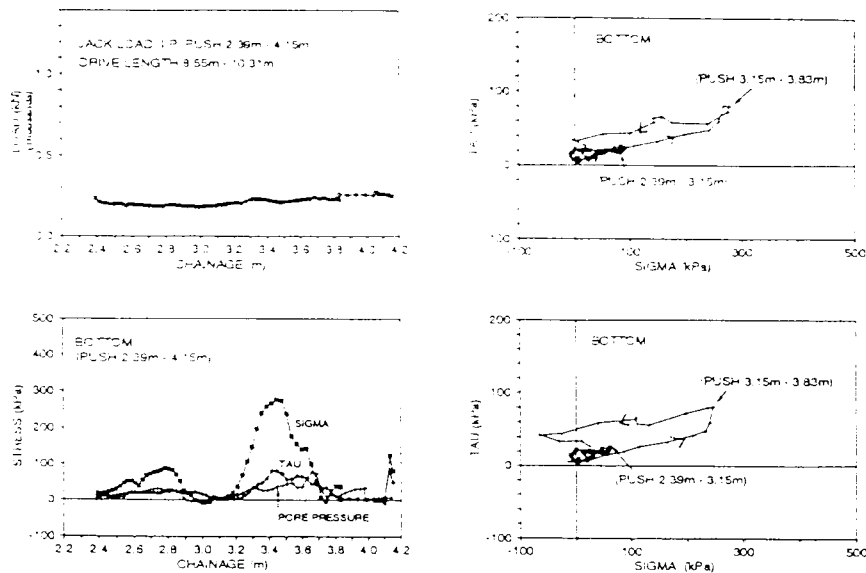
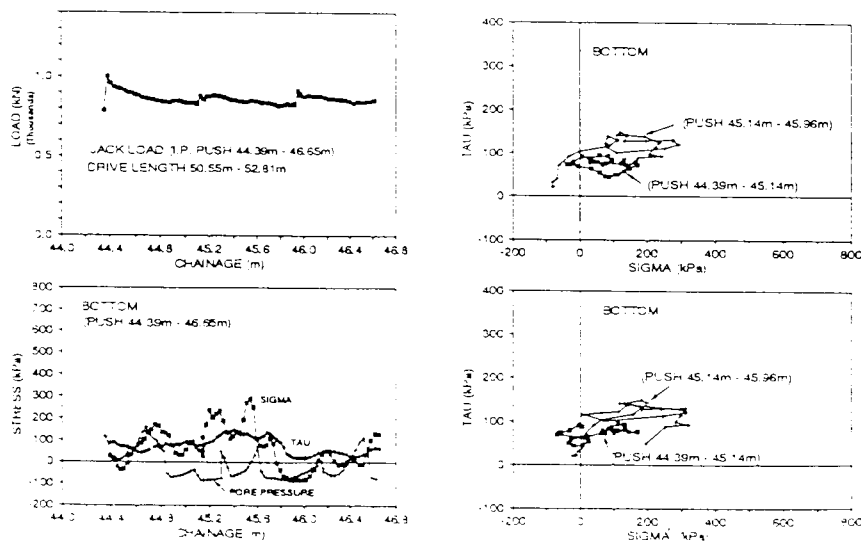


Fig. 9. Variation in pore water pressure.



Detailed response of pushes between chainages 2.39m and 4.15m



Detailed response of pushes between chainages 44.39m and 46.65m

Fig. 10. Detailed response of pushes.

Pipe-soil interface friction

Figure 11 shows plots of shear stress against total and effective radial stress for all pushes in scheme 1 from the stress cell in the invert 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,⁶ 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

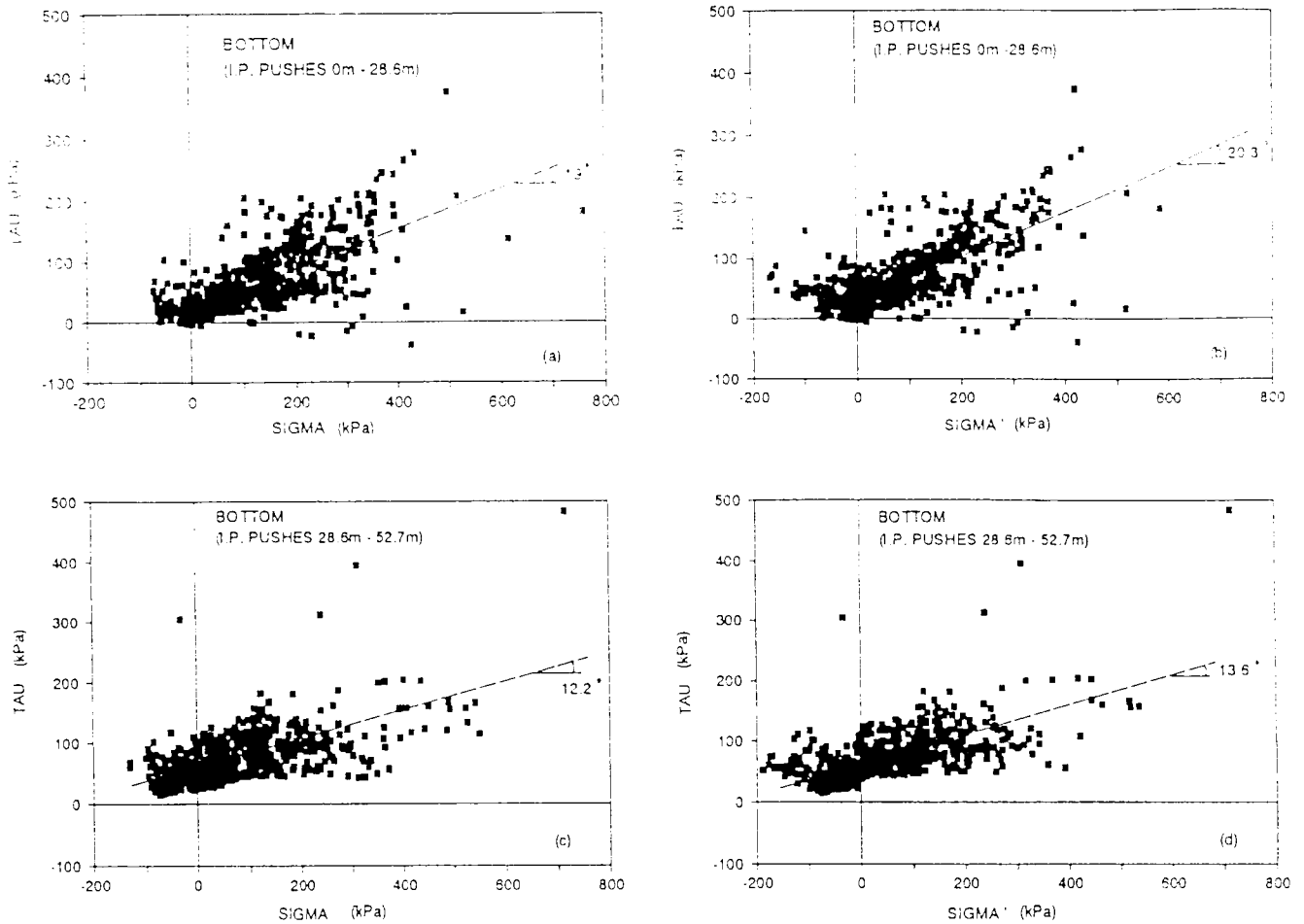


Fig. 11. Shear stress radial stress relationships: total and effective stress responses prior to heavy rain (a and b) and after wetting (c and d).

reduced angle of skin friction (δ') of 13.6° . In fact there is some suggestion that at higher radial stresses the interface shear stress becomes almost constant, suggesting behaviour akin to undrained adhesion.

It seems likely 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. A typical value of $\phi'_{residual}$ for low-plasticity glacial clay is given by Lupini *et al.*⁷ as 25.3° ; the measured interface value of δ' of 20.3° suggests an interface reduction factor of 0.8. Under "undrained" conditions at the end of the drive the ratio between the interface shear strength and the undrained strength of the clay is about 0.5-0.6, not dissimilar to factors used for design of piles.

Jacking force analysis and design

For a pipe resting within a stable tunnel bore a first approximation to the contact stresses may be

obtained from the elastic solution for a solid elastic cylinder resting in a cylindrical cavity.⁸ The contact width is given by

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

where Pu is the contact force per unit length, $kd = D_1 D_2 / (D_1 + D_2)$, D_1 is the internal diameter of the cavity, D_2 is the external diameter of the pipe, $Ce = (1 - n_1^2) E_1 + (1 - n_2^2) E_2$, E_1 is the elastic modulus of the soil, E_2 is the elastic modulus of the concrete pipe, n is the Poisson ratio as for E , and the contact normal stress distribution is given by

$$p = \frac{2Pu}{\pi d} \left(1 - \frac{x^2}{d^2}\right)^{1/2} \quad (2)$$

where $a = b/2$ and x is the distance to either side of the centre line of the area of contact.

In this case the pipes weighed 17.7 kN m^{-1} , and using

$$E_1 = 48 \text{ MPa}, \quad E_2 = 40 \text{ GPa}, \quad n_1 = n_2 = 0.2.$$

$$D_1 = 1.554 \text{ m}, \quad D_2 = 1.530 \text{ m}.$$

The initial interface stresses calculated to be 0.3 m, giving an average contact stress of 59 kPa and a maximum value of 78 kPa. These compare reasonably well with the average of the widely varying local contact stresses measured on the base of the pipe.

In this case the resistance to jacking will simply be due to the frictional resistance at the base of the pipes due to their self weight. Using the measured (total stress) value of δ of 19°, the calculated frictional resistance is 6.1 kN m^{-1} , about 15%

lower than the average measured value over the initial part of the drive of 7.2 kN m^{-1} . The discrepancy is probably due to the misalignments in the drive which cause some localised contacts between the pipe and the sides of the tunnel, adding to the total frictional resistance.

The much higher resistance over the later stages may be due to greatly increased contact between pipe and ground due to swelling of the clay and hence closure onto the pipeline. Certainly when the instrumented pipe was re-driven a short distance from the jacking pit after retrieval from the reception pit, significant contact between pipe and ground was observed (Fig. 12). An alternative suggestion is that of the model proposed by

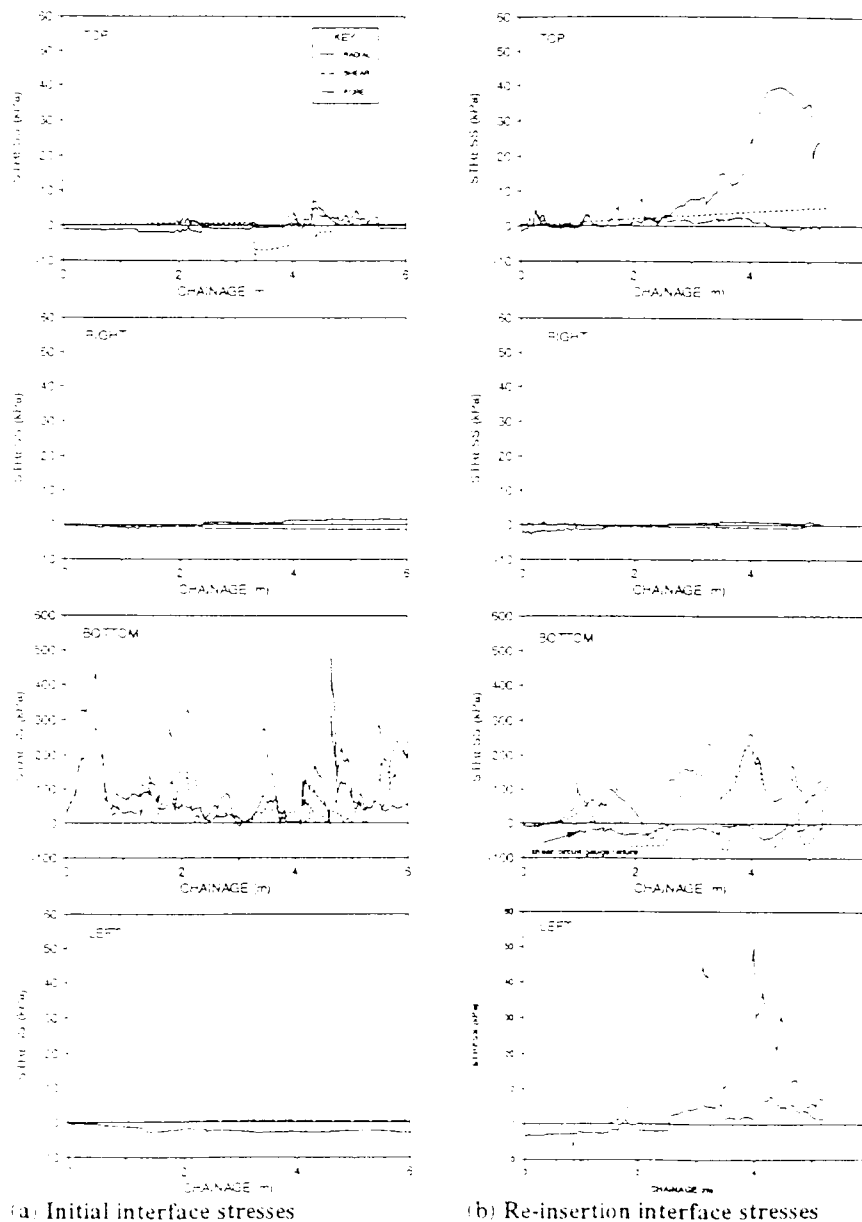


Fig. 12. Interface stresses during initial 6 m of drive and over corresponding length after re-insertion of the instrumented pipe.

Haslem,⁹ whereby undrained adhesion is assumed to act over a contact width given by the elastic solution in Eqn (1) above. It is difficult to know in this case exactly what undrained adhesion value to use. Back-calculation from the measured jacking resistance of 29.8 kN m^{-1} suggests an adhesion value of 100 kPa , which compares reasonably with the scatter of measured values for the later stages of the drive.

Two further aspects have to be included in calculations of overall jacking force. The face resistance in this case is low at about 120 kN , presumably because hand excavation of the clay to a slightly greater diameter than that of the shield was taking place. Use of the shield to trim the full annulus of the tunnel at the face could give much higher loads.

It should also be noted that jacking forces tended to be larger after a rest period, although it is known that this effect tends to be greater in high-plasticity clays. This effect appears to be infinitely repeatable, and is presumably related to dissipation during rest periods of pore pressures generated during pushes, but the exact mechanism needs further investigation. An attempt has been made to relate the proportional increase in jacking force during a stoppage to the length of the stoppage, on a logarithmic scale—see Fig. 13. There is considerable scatter but some trend is discernible. The maximum increase even for quite lengthy stoppages is about 0.3, and this would seem to be a reasonable factor to include in predictions of jacking force in such low-plasticity clays.

Pipeline alignment

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

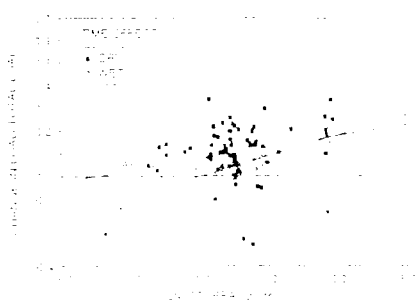


Fig. 13. Increases in jacking force during stoppages.

string to take a zig-zag course, known as "wriggle", which results in deflections at pipe joints. Throughout the research, changes in pipe-line 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 for this project are shown in Fig. 14. Within the limitations of measurement accuracy, the pipe line profiles did not change between successive surveys. Thus at least in these ground conditions, the full pipe line will continue to follow the path traced out by the shield during excavation.

Of more importance to the performance of the pipe joints than the line and level deviations are the resulting misalignments between successive pipes at the joint between them. The angle β (in three dimensions) between pipes may be calculated from line and level readings, or much more accurately measured by the joint gap instruments. The resulting values appear in the final two plots of Fig. 14. In this well-controlled pipe jack the angle is generally less than 0.3° , and never more than 0.5° .

It is sometimes suggested that pipe joints straighten significantly under load. Figure 15 shows a typical variation in β during a single push. Application of the jacking load has negligible effect on β , as does its reduction to zero at the end of the push, although β varies continuously during the push. In these ground conditions it appears that any attempt by the pipe line to straighten under load is restrained by ground reactions. Normal tunnel surveys, undertaken during breaks of work when the pipe string is unloaded, are then sufficient to establish pipe misalignment for site control.

Additional angular deviations can occur at joints due to a lack of pipe end squareness, and in theory these need to be offset against permissible misalignment angles at a joint. BS 5911 allows maximum angles of 0.13° – 0.15° (on each pipe end) for pipes in the range of internal diameter of 1200 – 1800 mm . However, an audit of a number of sample pipes from three different British manufacturers found typical end-squareness angles much less than this; it seems unlikely that joint angles of greater than 0.1° will often occur due to lack of end squareness when using pipes of British manufacture.

Pipe load paths

The joint gap instrumentation also allows the location to be determined of the point of

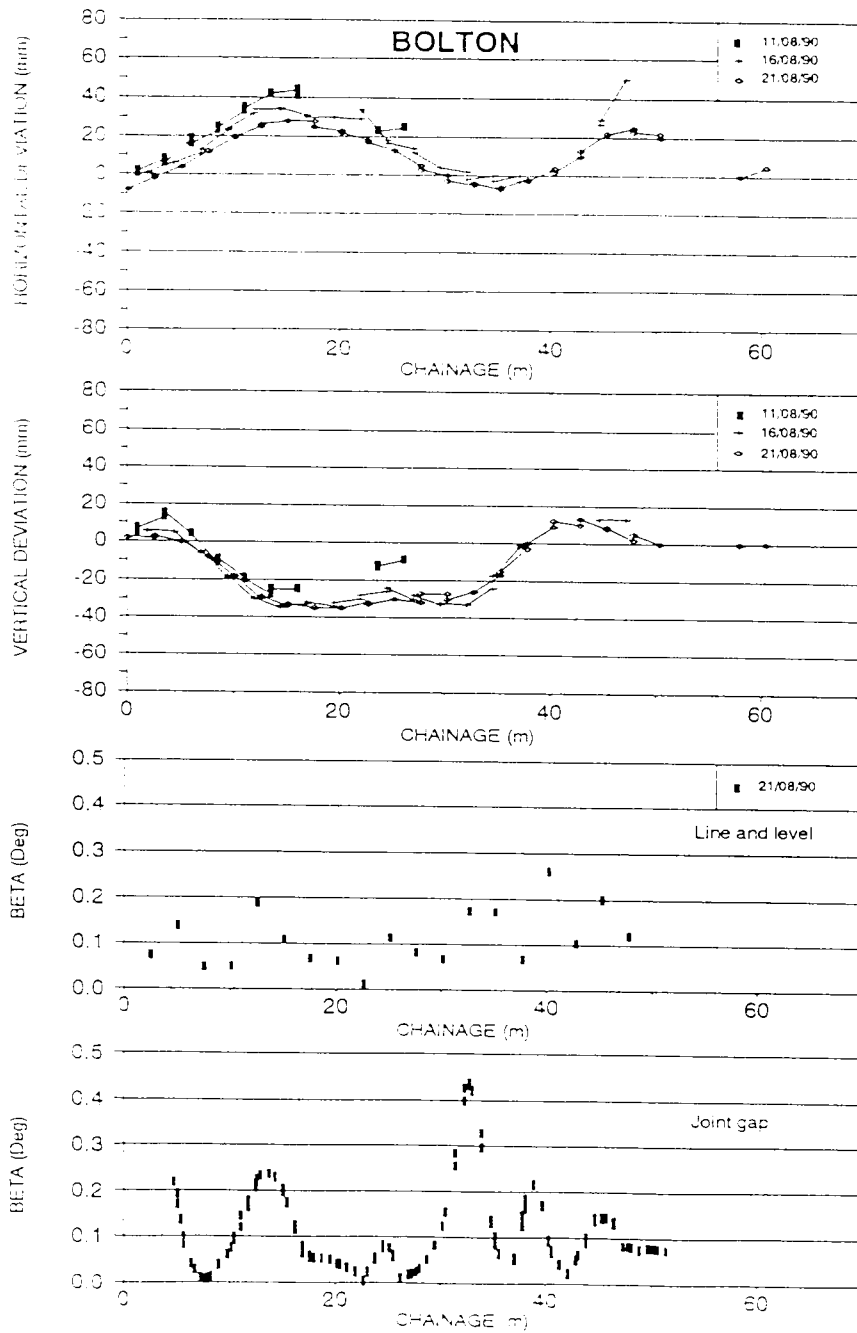


Fig. 14. Tunnel alignment surveys.

maximum compression in the joint, that is the point at which the two pipe ends are closest together and maximum stress is transmitted through the packer. The variation of this point, defined by the angle α , for the rear joint of the instrumented pipe throughout the drive is presented in Fig. 16. The point started near the invert, moved rapidly to the crown in a clockwise direction, then all the way round the pipe in an anticlockwise direction.

Assuming that the front joint behaved similarly,

but with a chainage lead of 2.5 m, typical "load paths" through the pipe can be determined. During rapid changes in α , the maximum stress points at the two ends of the pipe could be 90° or more out of phase, leading to near-diagonal loading of the pipe. However, at points of maximum β the value of α is almost stationary and loading is essentially down one edge of the pipe. These alternative loading scenarios are relevant to the design of the pipe barrel for jacking loads. Because of the relatively low jacking forces in this case the pipe

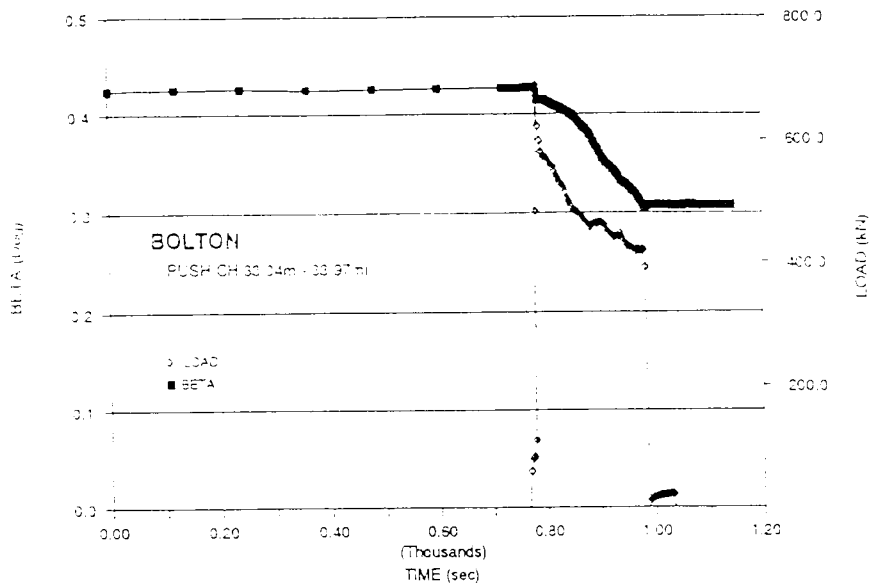


Fig. 15. Changes in pipe joint angle during a typical push.

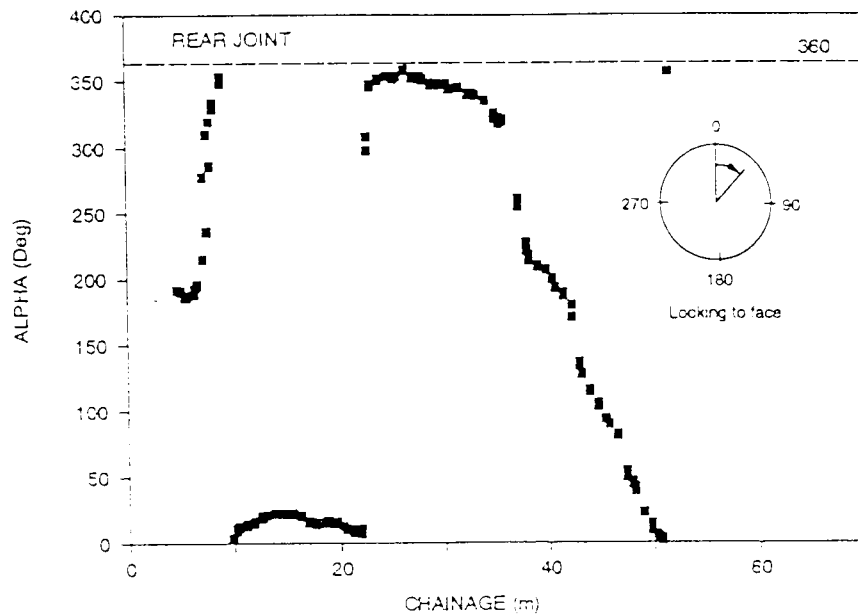


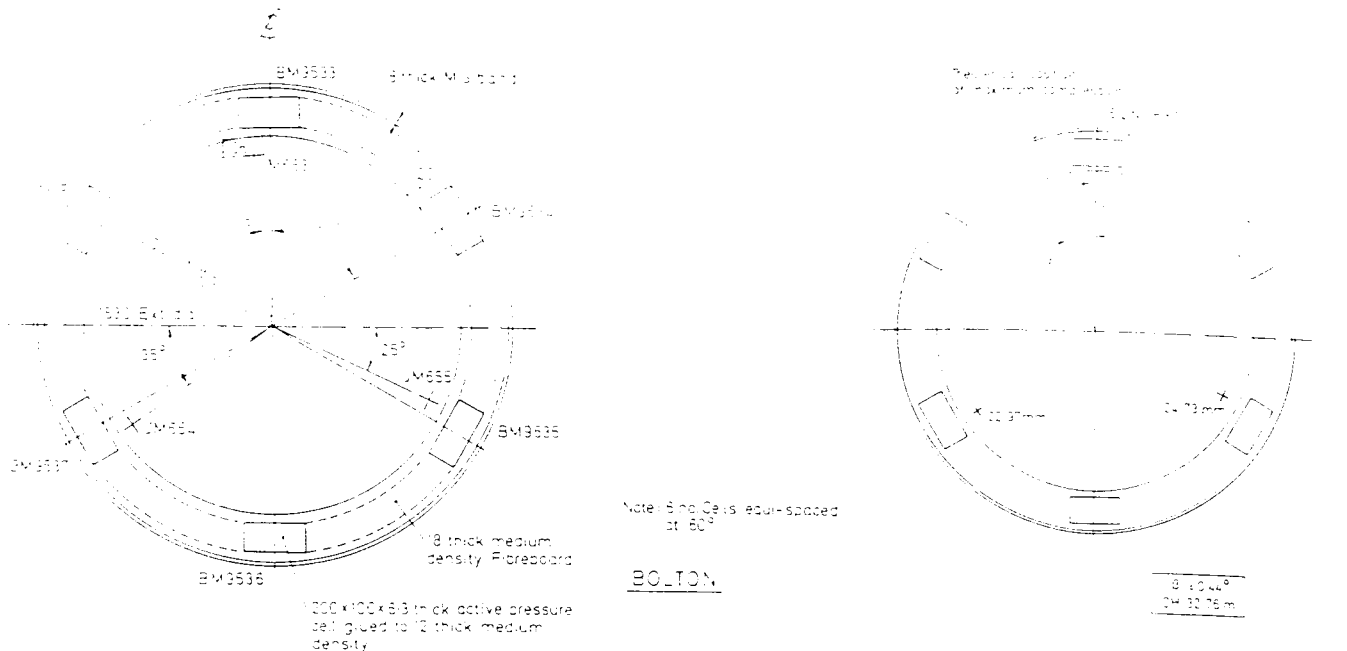
Fig. 16. Variation in the point of maximum compression across the rear joint of the instrumented pipe

strains were too small to allow accurate correlation with loads.

Pipe joint stresses

The pipe joint stress cells allow direct measurement of the stress distribution around a pipe joint, for correlation with joint loads and misalignment angles. In this scheme, because the instrumented pipe was close to the front of the pipe string, the load transmitted through the instrumented joint was very low at about 150 kN (15 tonnes). Also,

since only six joint cells were used they were quite widely spaced around the pipe end and could only give a crude indication of stress distribution around the joint (Fig. 17a). However, these were sufficient to demonstrate the important effect of stress localisation in misaligned joints: Fig. 17b shows the total jacking force being transmitted through a single pressure cell. Even allowing for the locally higher stiffness of the pressure cell compared with the packing material to either side, it is clear that only about an eighth of the pipe circumference is loaded. This effect is of consider-



(a) Location of pressure cells and joint movement indicators in instrumented joint

(b) Results from joint instruments at maximum misalignment angle

Fig. 17. (a) Location of pressure cells and joint movement indicators in instrumented joint. (b) Results from joint instruments at maximum misalignment angle.

able importance to joint design for higher jacking loads and larger misalignment angles, and will be considered in detail elsewhere.

CONCLUSIONS

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 and jack load cells fell 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 into 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. These 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.

Pipe-soil interaction

Data have been collected on the overall jacking resistance, including face resistance, and on the local interface stresses between pipe and ground. The interaction mechanisms in this case reflected 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. This happened through most of scheme 1.

Detailed examination of individual pushes has highlighted a partially drained frictional material response. The data suggest that under typical

radial stress levels, slippage occurred 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 suggested that the soil at the interface was in an unsaturated state (even in originally saturated ground). The time between excavation and the arrival of the instrumented pipe allowed the surface to dry out, with additional drainage taking place during the shearing process as a result of the relatively high permeability and absorption of the concrete.

For pipes sliding in a stable bore, the jacking load can be assessed from simple pipe selfweight sliding resistance. In cohesive soil a residual friction angle from shear box tests should provide suitable conservative values for the interface friction angle, though allowance should be made for the effects of misalignment (an increase of about 15%) and for increases during stoppages (+30% in this case) unless continuous working is used to prevent the latter. Face load needs to be added to pipeline resistance, but is highly dependent on excavation procedure. In softer clay a calculation based on undrained adhesion may be more appropriate.

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.

Typical misalignment angles on a nominally straight drive, 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 at the joints (even when a packer of compressible material such as fibreboard was used) and these correlated well with the joint misalignments. 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.

Pipe barrel loading

As the pipe string "wiggles" through the ground, the angular orientation of the position of maximum compression moves around the joint. 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°, the pipe will be loaded across a diagonal. In this scheme, with the instrumented pipe close behind the shield, the maximum phase difference appeared to be about 90°, but was close to zero in situations of large misalignment angle.

ACKNOWLEDGEMENTS

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A Case Study of an Instrumented Microtunnel in Fine Sand

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SYNOPSIS: A man-entry size microtunnel in fine sand was monitored in 1994 as part of a programme of research into pipe jacking at Oxford University. Measurements of jacking forces, radial and shear stresses at the pipe-soil interface and ground movements were made. Selected data from the field measurements are presented and discussed and conclusions drawn.

1. INTRODUCTION

1.1 *General*

The use of microtunnelling and pipe jacking for the installation of new sewer pipelines continues to be employed U.K. To gain a better understanding of the technique, a research project was established at Oxford University in 1986. The programme of research is supported by the Pipe Jacking Association, the Engineering and Physical Sciences Research Council, and five U.K. water companies (North West, Northumbrian, Severn Trent, Thames and Yorkshire). Milligan and Norris (1993) describe the programme of research; results from the first two stages are summarised by Milligan and Norris (1994). The current stage of research started in November 1992 and involves monitoring actual pipe jacks under construction by inserting a specially-instrumented pipe into the pipe string. To date four schemes have been monitored and this paper presents a case history of the second of these, describing the monitoring work and presenting some of the results.

1.2 *Units*

The S.I. system of units has been used throughout this paper. The basic units of measurement are:

length m (metre),
and
force N (Newton).

The derived units for stress or pressure are kPa ($1 \text{ kN m}^{-2} = 1 \text{ kPa}$).

Conversion to Imperial units:

1 m = 3.281 ft.

1 kN = 0.1004 tonf.

1 kPa = 0.145 lbf in².

2. SITE DETAILS

2.1 *Background*

The reported microtunnelling took place in Southport on the north-west coast of England. The overall scheme was for the construction of 5.3km of interceptor sewer, using segmental tunnelling, for North West Water. It involved intercepting three existing outfalls and also the construction of two new spurs using microtunnelling techniques. One of these spurs underneath a very busy urban road (Nevill Street) was instrumented and monitored during construction. The fully lubricated drive, below the water table, was 160m long in dense silty sand, average depth to axis was about 7.4m and the size was of 1.0m internal diameter, 1.2m outer diameter. Buoyancy theory gives a negative self weight for the jacking pipe equivalent to -3kN/m in water.

A long-section, exaggerated vertically, illustrates the microtunnel in Figure 1. Miller-Markham carried out the microtunnelling using a full-face slurry tunnel boring machine.

2.2 *Ground conditions*

The stratum below the road pavement to a depth of about 3.5m is aeolian (wind blown) sand, comprising medium dense silty fine sand. The tunnel passes through the stratum underlying the aeolian sand. This material is alluvium, consisting principally of medium dense grey fine

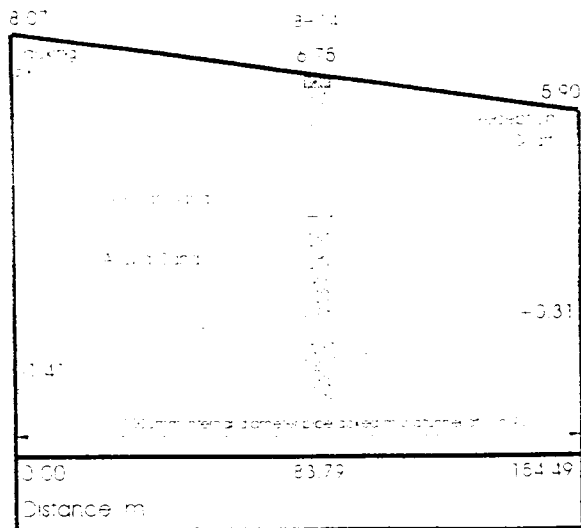


Figure 1 Long section of the Nevill Street microtunnel

sand. Bedrock, Mercia mudstone, typically lies at about 24m below ground level.

As part of the Client's site investigation, a borehole (No. 14) was sunk close to the centre line of the proposed microtunnel - details are presented in Table 1. The particle size distribution for the alluvial sand is shown in Figure 2.

Table 1 Borehole details

Depth: m	Description
0.00 - 0.40	Road pavement - tarmac over concrete.
0.40 - 3.60	Medium dense fine to medium SAND with occasional angular shell fragments up to fine gravel size. Rare angular fine to medium gravel to 1.50m.
3.60 - 12.00 (end of borehole)	Medium dense silty fine SAND with occasional angular shell fragments up to 3mm. Becoming less silty with depth, medium sand content increasing. Organic in parts. Average permeability 2×10^{-7} m/s Shear strength parameters $c' = 0 \text{ kN/m}^2$ $\phi' = 43$

Since undisturbed samples of granular material are difficult to obtain, emphasis was placed on Standard Penetration tests (SPTs) - results shown in Figure 3. The average N value in the alluvium

is 16.

Groundwater in Nevill Street was about 2.5m below ground level. During the site investigations variation in groundwater levels in any one observation well was minimal, typically less than 0.2m. Since readings were taken at roughly the same time each day, it was concluded that there is no significant tidal variation in groundwater levels.

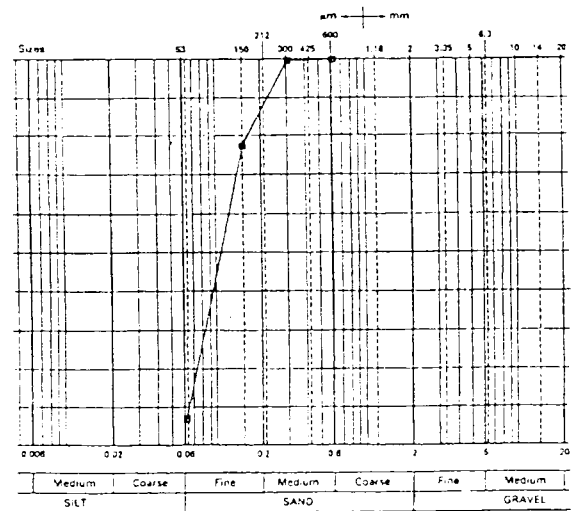


Figure 2 Particle size distribution: Borehole No. 14, depth 5.5 - 6.0m

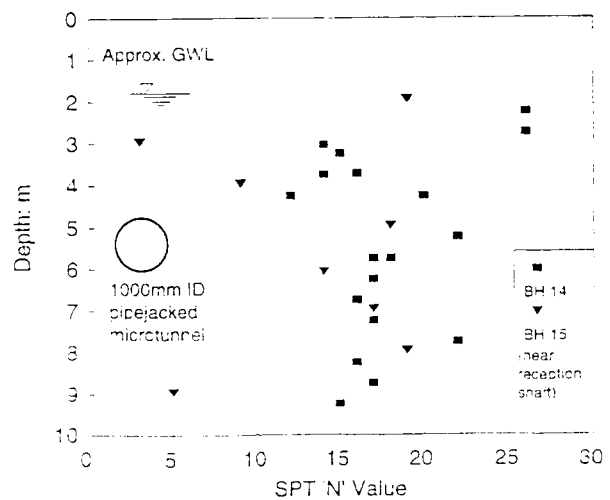


Figure 3 Plot of SPT 'N' values against depth: Nevill Street

2.3 Equipment

The tunnelling machine used was the 'Super Mighty' - supplied to Miller-Markham by Okumara Heavy Industries - which is illustrated in Figure 4. A feature of the machine which has been adapted for the European and U.K. markets is the crusher cutterhead, a universal head suitable for excavation in a wide range of ground

conditions. The overbreak at the cutterhead was 15mm on diameter relative to the shield which in turn was 10mm larger in diameter than the pipes.

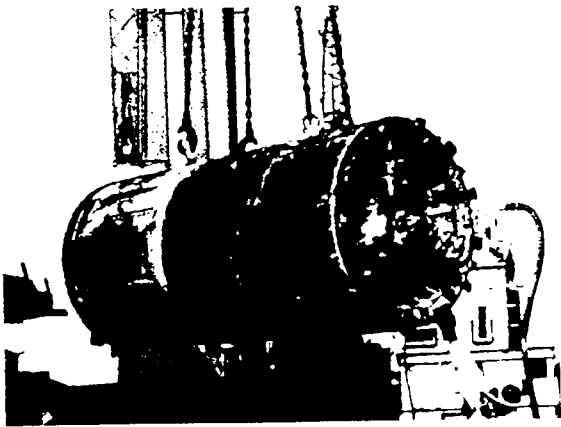


Figure 4 Miller-Markham 'Super-Mighty' TBM

In the jacking pit the variable speed thrust jack unit is fitted with two hydraulic rams mounted either side of the jacking pipe. Two strokes of the thrust rams are required to complete the jacking of one pipe, achieved by inserting large pins into the assembly to move the reaction point from the front flange of the ram to the rear. Jacking forces were monitored independently for the research project. Where possible, they are measured by attaching heavy duty load cells to the ram ends. With this particular jacking unit however, it was not possible to fix the load cells, so transducers were fitted to the hydraulic feed circuit to monitor hydraulic pressures. Output from the transducers was later converted to jacking forces by calibrating the hydraulic pressures against the load cells normally used.

3. INSTRUMENTATION

3.1 Pipe jacking instruments

The research instrumentation scheme used on this particular site is depicted in Figure 5. Instruments in the thrust pit record jacking forces. The special instrumented pipe - inserted at pipe number two - contained the following instruments:

- a) Twelve contact stress cells (CSCs) to record localised normal and shear stresses and pore water pressures at the soil-pipe interface. The active face of these instruments have frictional similitude with the concrete of the jacking pipes.
- b) Three joint movement indicators

positioned equidistantly to measure movements across the joint gaps.

- c) The data acquisition hardware with power supply.

To assist in interpretation of the huge amount of data, a comprehensive site diary was kept detailing shift progress, times of lubrication and line and level surveys.

3.2 Ground instruments

Surface settlements above the tunnel were measured using precise levelling techniques on a series of studs driven into the road pavement. Sub-surface movements were monitored by installing two inclinometer access tubes with magnetic settlement plates at the locations shown in Figure 6.

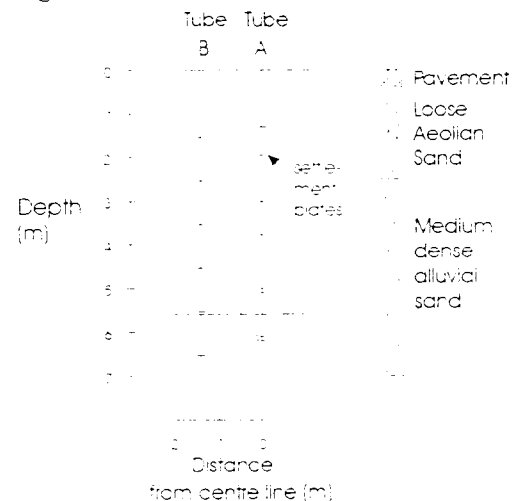


Figure 6 Inclinometer tubes

Change in tube profile was monitored frequently during and after construction using a uni-axial inclinometer torpedo with an automatic logger. Vertical movements around the access tubes were monitored using a reed probe triggered by the magnetic plates. Depths to the plates were recorded relative to the top of the tubes so levels were taken on the tube tops at about the same time.

4. PIPE-SOIL INTERACTION

4.1 Introduction

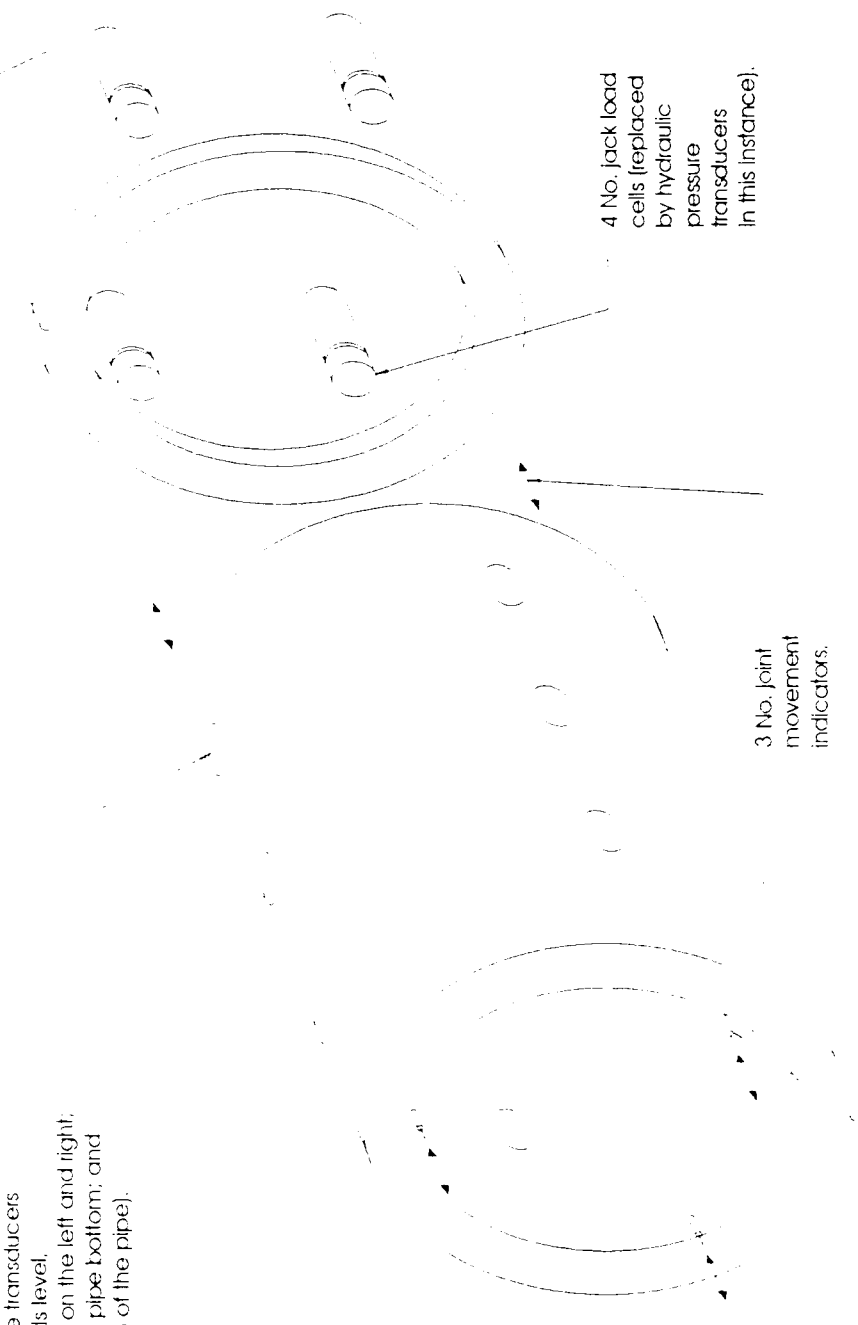
The jacking load is a combination of face resistance at the shield and line friction load. Face resistance is due to the amount of penetration resistance of the cutting edge and any measures taken to stabilise the face, i.e. slurry pressure, as is used with the Super Mighty microtunnelling system. Frictional resistance

Jacking Pit

Instrumented Pipe

- 12 No. Contact stress cells, incorporating pore pressure transducers (6 at axis level, 3 each on the left and right; 3 in the pipe bottom; and 3 in top of the pipe).

- 1 No. linear displacement unit.



- 4 No. jack load cells (replaced by hydraulic pressure transducers in this instance).

- 3 No. joint movement indicators.

- 3 No. joint movement indicators.

Figure 5 Research instrumentation scheme

around the instrumented pipe was measured during the pushing of the pipe string allowing an estimate of line friction load to be made. An examination into pipe-soil interaction during the reported scheme follows. Construction related factors including pipeline misalignment, the use of pressurised lubricant, and the effect of stoppages will each be investigated.

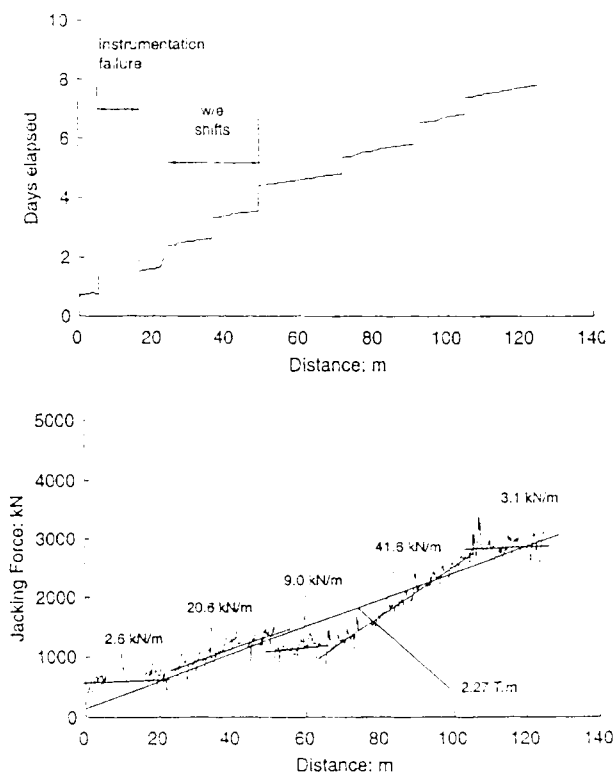


Figure 7 Jacking force record

4.2 Jacking forces

The measured jacking loads together with the progress record are depicted in Figure 7. The record can be divided into five different areas which appear to correspond to the pumping of lubricant. Average frictional resistance (jacking force divided by length of pipeline) is shown on the plot and Table 2. The increase in force to 41.6kN m between 65m and 106m can probably be attributed to a break in lubricating caused by a pump failure. Once lubrication recommenced, after two or three pushes when higher jacking forces were recorded, the average increase fell to 3.1kN m, more than a tenfold reduction in jacking load.

The overall average increase in line friction is 22.7kN m, equivalent to about 6kPa. Craig (1983) gives typical friction limits of between 10kPa to 15kPa for unlubricated wet sand.

Table 2 Jacking loads

Distance m	Lubrication	Measure average friction	
		kN/m	kPa
0 - 22	frequent pumping	2.6	0.69
24 - 52	frequent pumping	20.6	5.46
52 - 65	infrequent pumping	9.0	2.39
65 - 106	pump failure	41.6	11.03
106 - 126	frequent pumping	3.1	0.82
0 - 126 (all)		22.7	6.02

4.3 Face resistance

From the machine operator's driving records, the average face pressure applied was 0.64kg/cm², equivalent to a face resistance of 76.6kN.

A method for predicting shield resistance from "Standard Pipe Jacking Construction System" (author unknown), a manual of Iseki Poly-Tech. Inc., which takes into account the N value as determined by SPTs is given by:

$$P_s = 1.32 \cdot \pi \cdot D_s \cdot N \quad (1)$$

where P_s = shield resistance (kN), and D_s = external diameter of shield (m).

For this scheme:

$N = 16$; and

$D_s = 1.22m$.

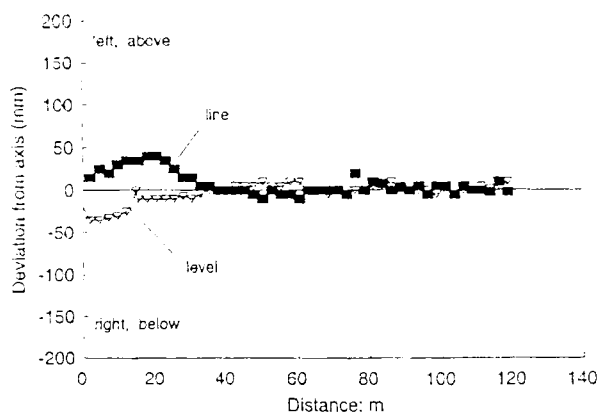
giving $P_s = 80.9kN$. Measured face resistance plus P_s gives a resistance of 157kN.

The constant force required to push the shield into the excavation can also be measured off the vertical axis of the jacking force record. Since the jacking load is non-linear, the intercept of forces measured over the first 22m is taken, the magnitude of which is 592kN. The difference, of over 400kN, is most likely due to an underestimate of shield resistance from eqn. (1).

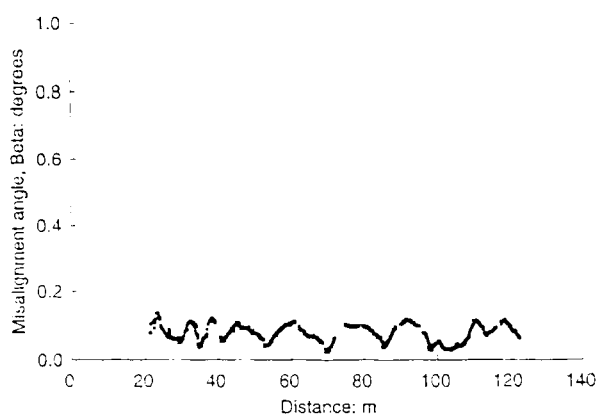
4.3 Line and level control

A laser was mounted in the pit bottom for line and level control with the beam being picked up on a target positioned within the machine mounted survey device. Steering to line and level

is achieved through four steering jacks connecting front and rear sections of the shield. Control of the jacks is from the operator's control panel through the selective articulation of the relevant steering jacks. Results from a line and level survey carried out along the pipeline by the researcher are illustrated in Figure 8a.



a Pipeline alignment survey



b Variation of misalignment angle, Beta

Figure 8 Pipeline misalignment

The plot shows that directional control was poorest at the start of the drive, but remained within allowable tolerances. After about 30m however, steering adjustment brought both line and level alignment onto axis with very little deviation after that. Deflections in the instrumented pipe joint were monitored using the joint movement indicators. The resulting misalignment angle data are shown in Figure 8b. The effect of misalignment on overall jacking loads and increase in radial stresses will be minimal here because line and level control was so good.

4.4 Interface stresses

The local interface stresses recorded around the instrumented pipe during pushing are shown in

Figures 9, 10, 11, and 12 for the bottom, left, top and right locations respectively. The plots show variation in total radial stress, shear stress and porewater pressure against distance travelled by the recording instrument. Effective radial stresses, calculated by deducting recorded pore pressures from total radial stresses, are shown in Figure 13. The effective stresses shown for the centre-right location are calculated using hydrostatic pressures due to failure of the pore pressure transducer in that particular contact stress cell. Average values from the presented data sets are shown in Table 3.

Table 3 Average stress values (kPa)

		Bottom	Left	Top	Right
Front	σ	54	49		57
	σ'	0	7		13
	τ	1	10		9
	u	54	43		45
Centre	σ	56	90	54	53
	σ'	6	45	16	12*
	τ	3	29	8	9
	u	50	46	38	
Rear	σ	52	91	82	46
	σ'	4	49	44	2
	τ	5	40	31	2
	u	48	41	38	44

* calculated using hydrostatic pressure

4.4.1 Porewater Pressure

Plots showing variation in porewater pressure, or fluid pressure, indicate that hydrostatic conditions mostly exist during jacking. Small increases above hydrostatic pressures during the pushing of the first 31 pipes (about distance 75m on the horizontal axes) and corresponding increases in total radial stresses result from pumping pressures during lubrication.

4.4.2 Bottom

Effective radial stresses and shear stresses along the pipe bottom are close to zero (suggesting no contact between pipe and soil) for most of the drive - the pipe is floating due to groundwater and lubricant pressure. The only bottom contact is shown to occur as the instrumented pipe

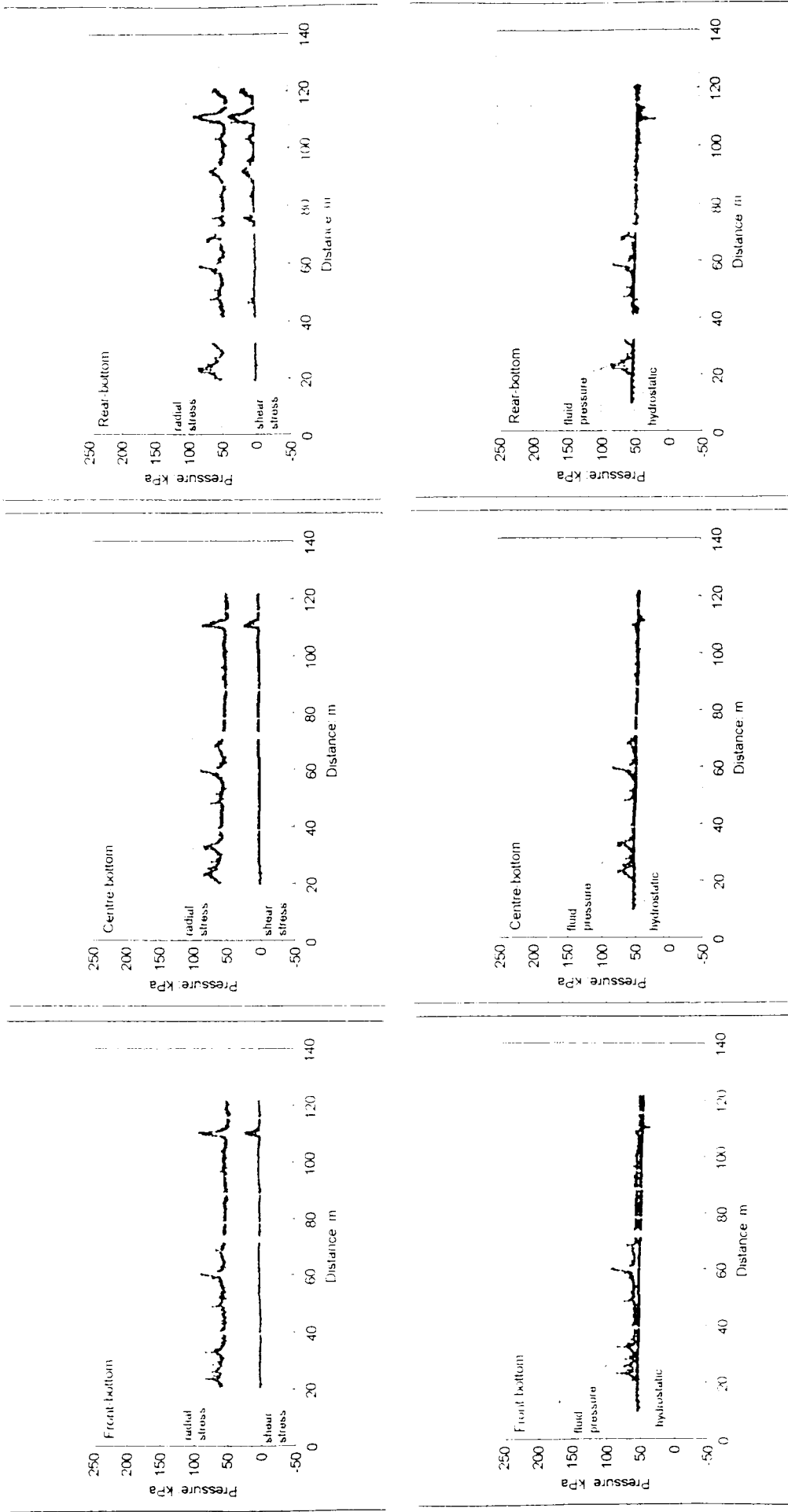


Figure 9 Variation of bottom interface stresses with distance

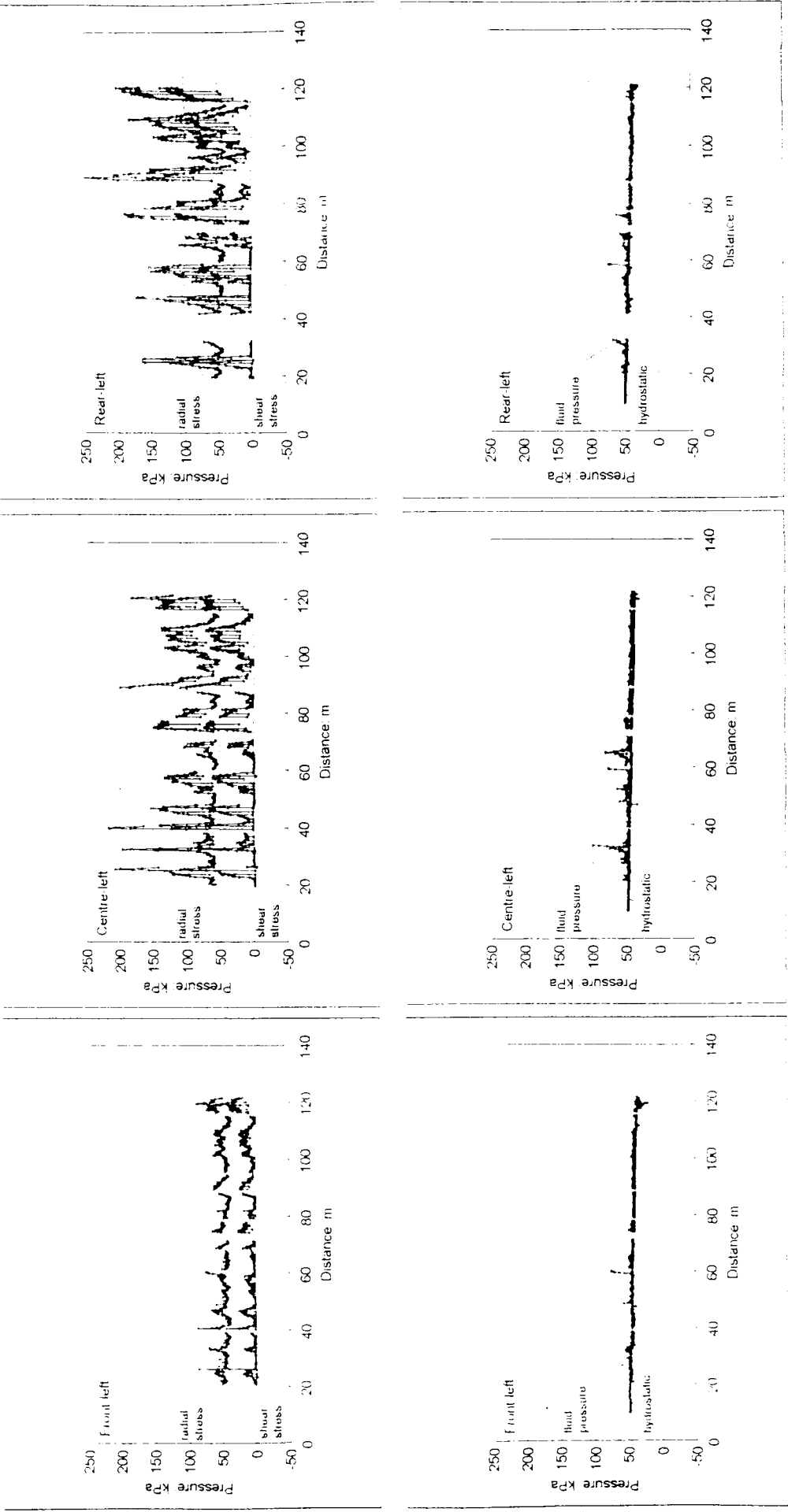


Figure 10 Variation of left interface stresses with distance

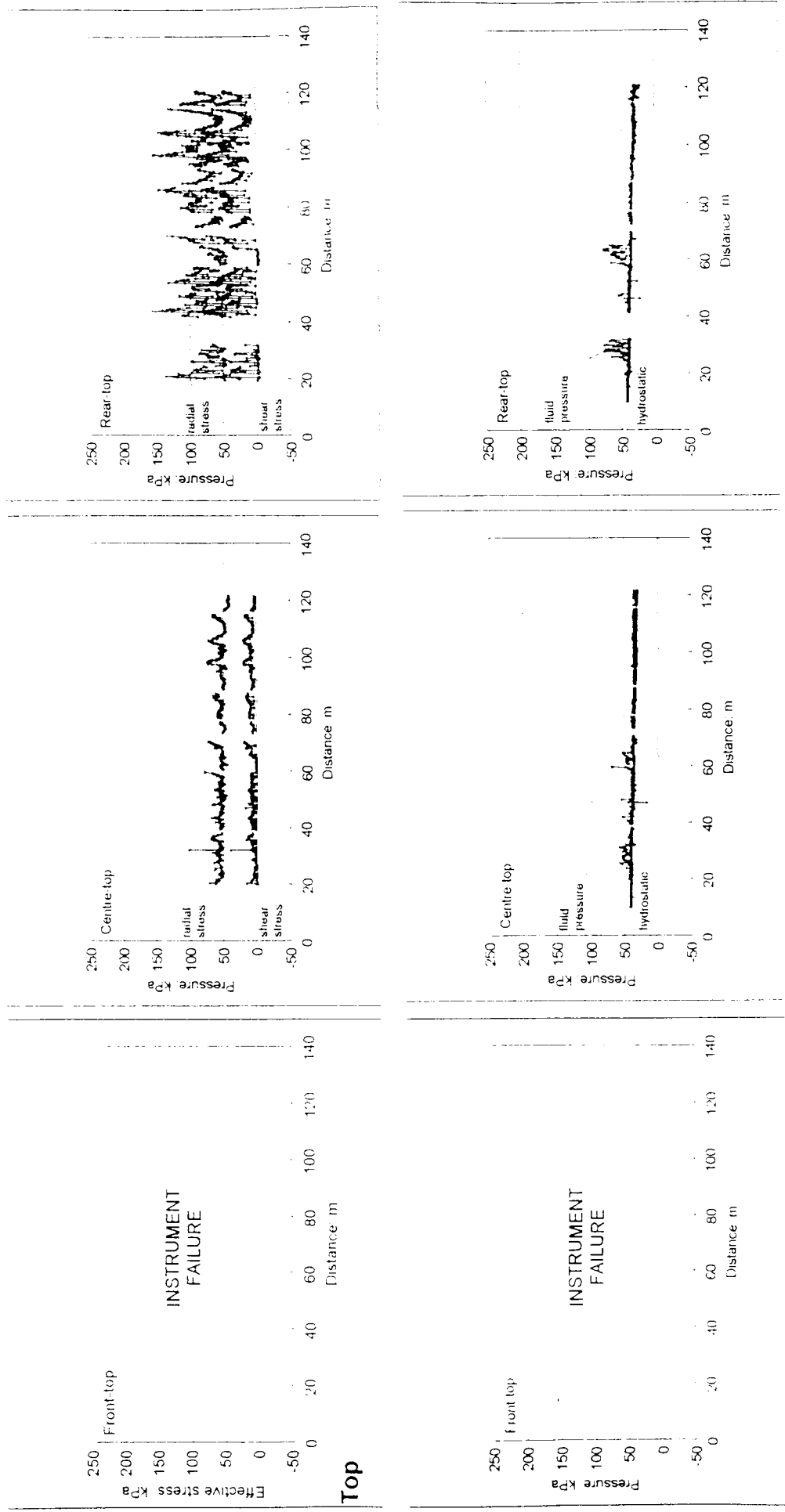


Figure 11 Variation of top interface stresses with distance

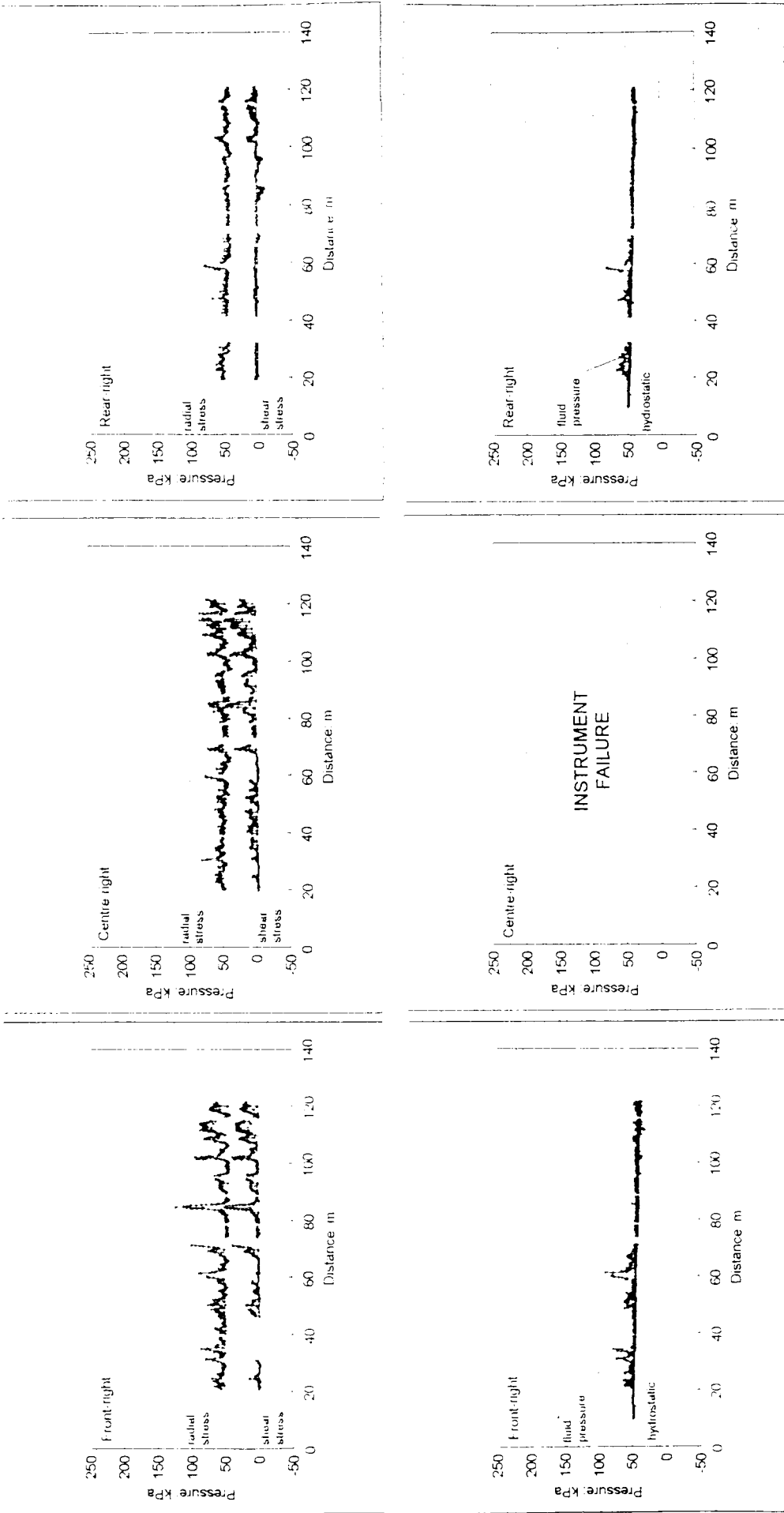
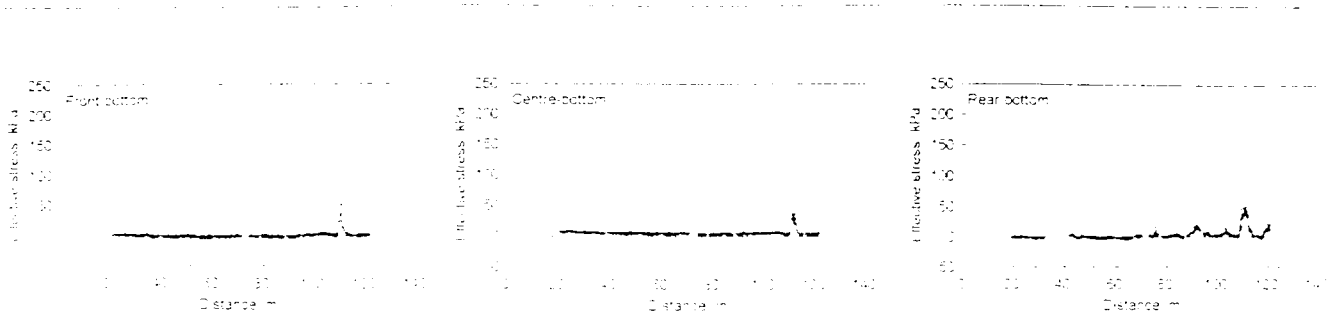
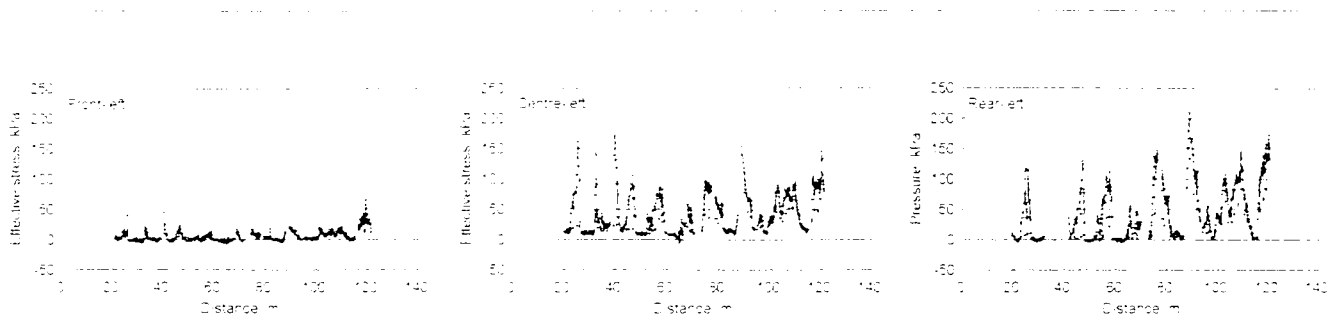


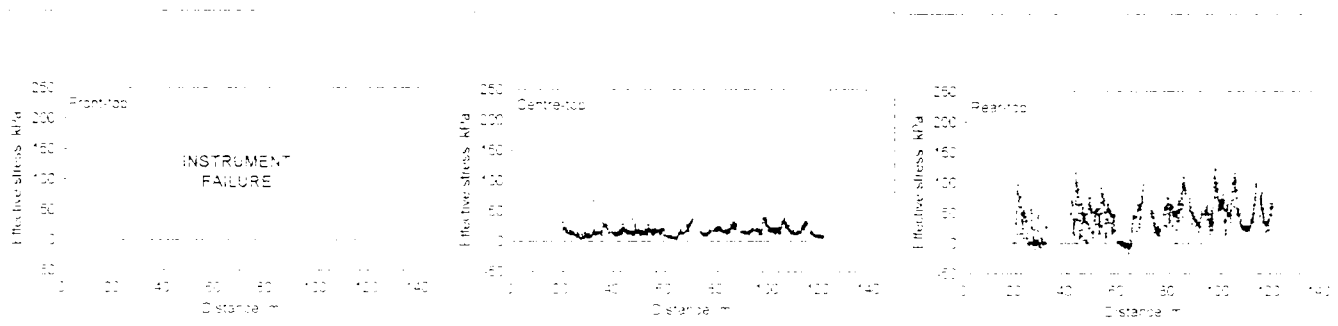
Figure 12 Variation of right interface stresses with distance



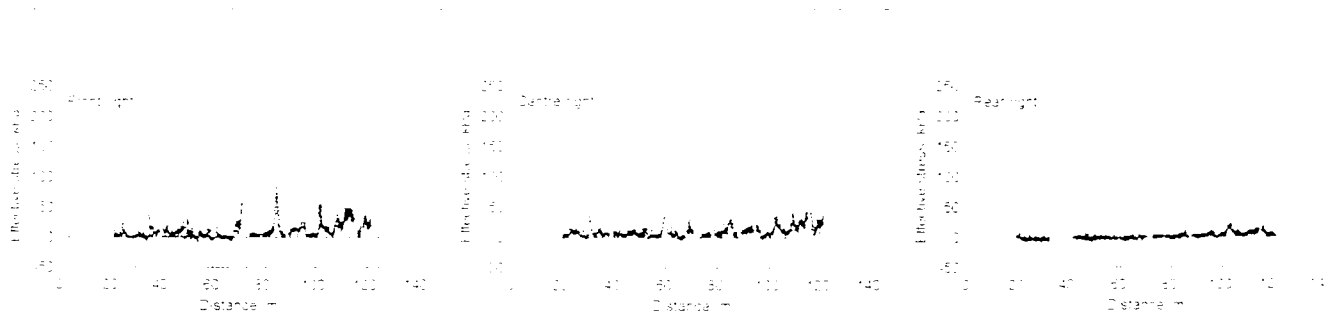
Bottom



Left

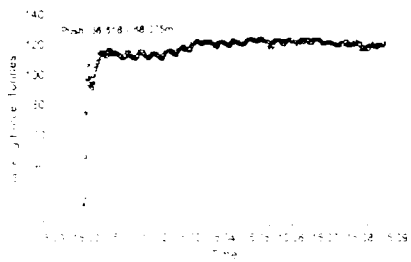


Top

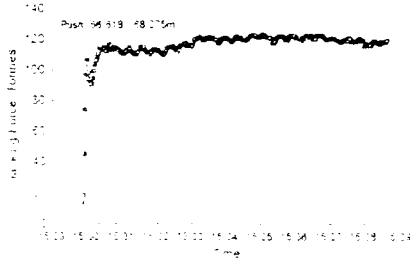


Right

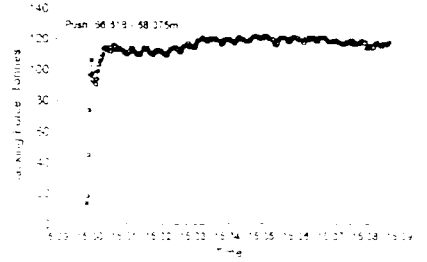
Figure 13 Variation of effective interface stresses with distance travelled



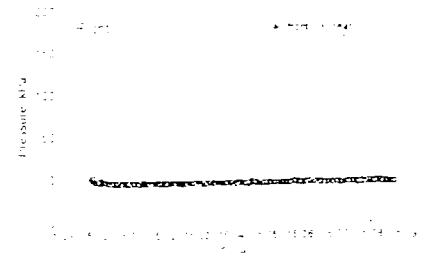
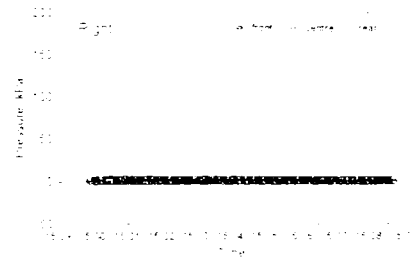
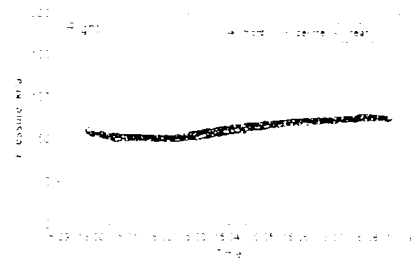
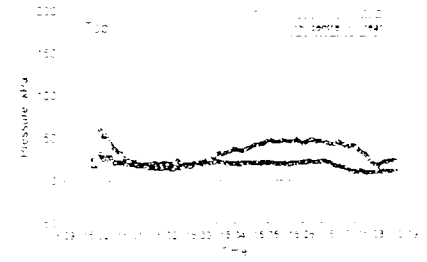
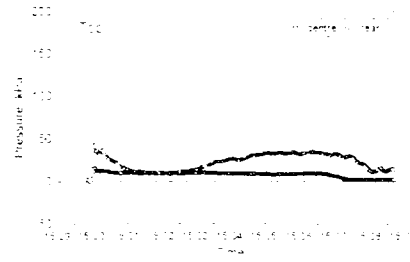
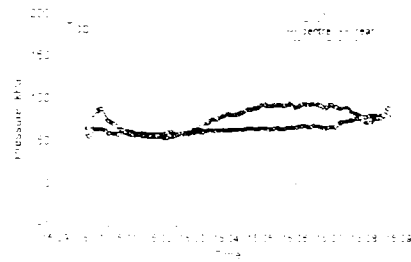
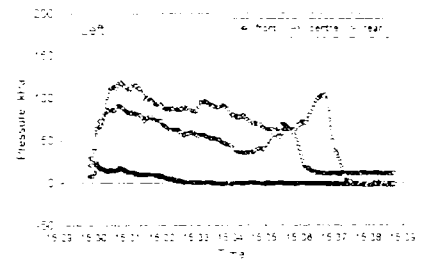
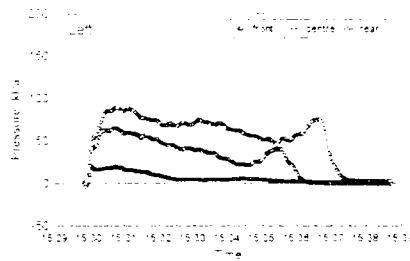
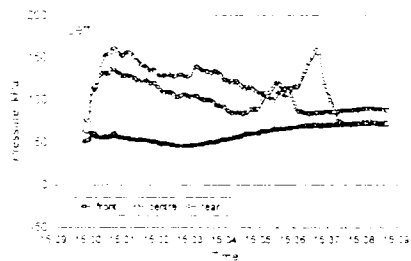
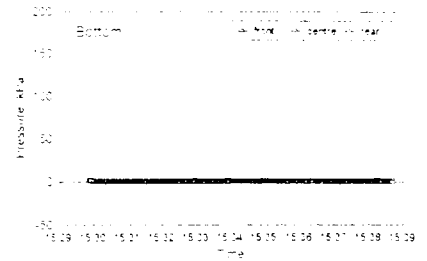
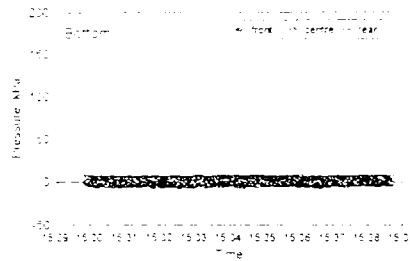
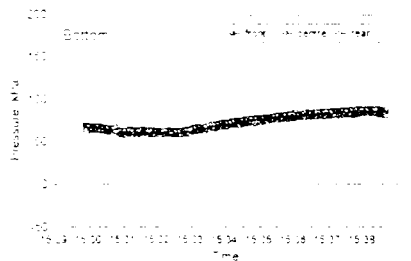
Jacking Force



Jacking Force



Jacking Force



Total Radial Stress

Shear Stress

Effective Radial Stress

Figure 14 Variation of interface stresses with time (push 61.968 to 63.425m)

passes through the tunnel at a distance of about 110m.

4.4.3 Left

Contact stresses at axis level along the left of the pipe are low at the front, but significant on the centre and rear - higher stresses occur at the rear.

4.4.4 Top

Unfortunately, contact stresses on the top-front position were not recorded due to instrument malfunction. Stresses measured at the pipe rear are much higher than those recorded at the centre.

4.4.5 Right

Data show contact during some pushes along the right axis of the pipe. The magnitude of stresses reduce from front to rear which is the reverse of that occurring along the left side.

4.4.6 General

Contact between soil and instrumented pipe only consistently occurs along the left (from between the front and centre, to the pipe rear) and along the top towards the rear of the pipe - all other plots probably show interface stresses mostly between lubricant and pipe. The mixture formed by lubricant and soil produces a stable gel which creates a low friction shear zone. The data suggest a non-uniform gel around the pipe both circumferentially and longitudinally.

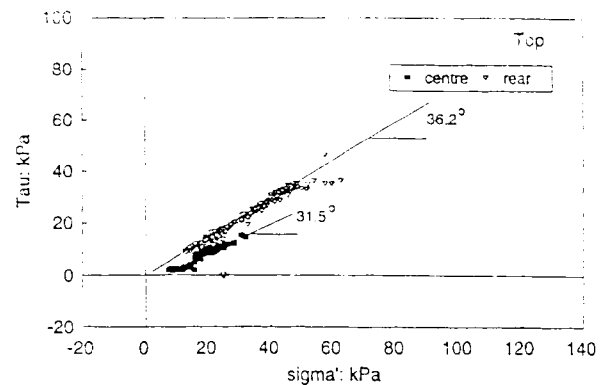
4.5 Pipe-soil friction

4.5.1 Push 61.968 to 63.425m

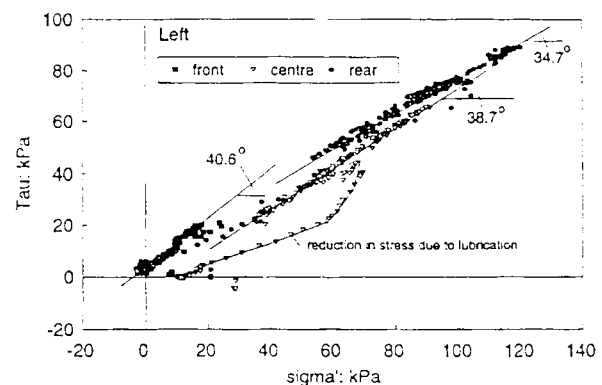
The compressed nature of the contact stress records shown in Figures 9 to 13 makes it difficult to evaluate pipe-soil friction behaviour during pushing. A single push has been chosen to illustrate the detailed response around the instrumented pipe. Figure 14 shows the change in contact stresses against time during the push from 61.968 to 63.425m. The three columns of plots show the jacking force and variation in total radial stress, shear stress and effective radial stress - for clarity, fluid pressures have been omitted.

Along the right axis and pipe bottom shear stresses and effective radial stresses are effectively zero - mean values of shear stress are 2kPa and 4kPa for the bottom and right of the pipe respectively. These values are consistent with the typical shear strength of a bentonite lubricating slurry. The increase in total radial stresses at these locations - beginning at about 15:32 - is due to the pumping of lubricant (pumped between 15:31:54 and 15:38:20).

There is clearly contact between pipe and soil along the left axis from around the centre to the pipe rear. The mean value of shear stress at the front is 6kPa (mean effective stress is 3kPa), but at the centre and rear the values are 28kPa and



a Top



b Left

Figure 15 Effective radial stress shear stress relationship during the push 61.968 - 63.425m.

54kPa respectively, with mean effective stresses of 45kPa and 69kPa. Contact stresses recorded along the pipe top during the push indicate contact between pipe and soil or soil lubricant gel - mean shear stress values are 8kPa at the centre and 23kPa on the rear with respective effective stresses of 19kPa and 32kPa. The relationship between shear and effective stresses on the pipe top and left are illustrated in Figure 15. The data show a purely frictional response as expected for the type of soil, but give differing friction angles. Taking data from the top-rear, left-centre and left-rear, where the range of stresses are greater, friction angles vary from 34.7° to 38.7°. The laboratory determined angle of internal friction of the soil, ϕ' , is 43° which gives a skin friction coefficient range from 0.81 to 0.90 - Potyondy (1961) gives a skin friction coefficient of 0.87 between sand and concrete.

4.5.2 Lubrication effect

The nearest lubricant injection points to the instrumented pipe, inserted at pipe 2, were placed at the following positions:

- 1) Shield - right.
- 2) Pipe 4 - right, and
- 3) Pipe 8 - right.

This would explain the low contact stresses along the right and the relatively high stresses measured along the left. A lubrication gel as illustrated in Figure 16 probably formed around the pipe.

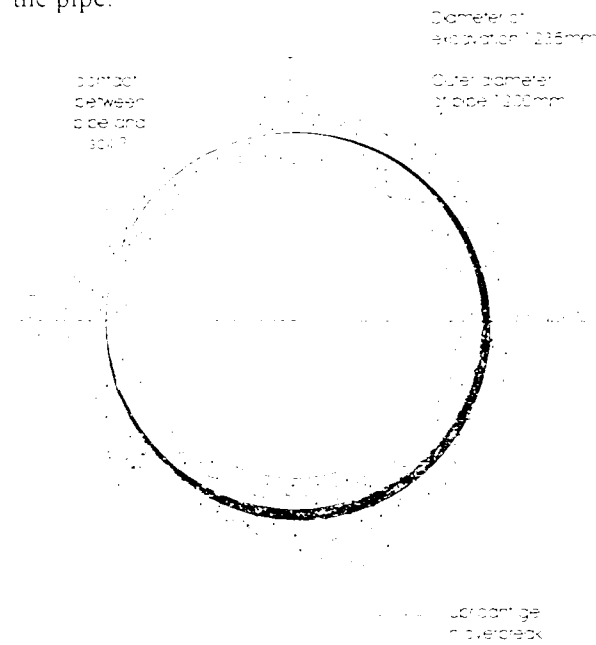


Figure 16 Lubrication gel around the centre and rear of the instrumented pipe.

Considering the change in stresses with time on the pipe left at the rear: after the initial increase in shear and effective stresses at the start of the push there is an almost linear decrease until about 15:34:30 when stresses begin to increase but then fall rapidly to zero from about 15:36:30. At the centre of the pipe the stresses follow a similar pattern but the increase and sudden reduction occur about a minute earlier. The reason for this can be attributed to the pumping of lubrication slurry. Similarly, the change in contact stresses on the pipe top during this push is mainly due to lubrication.

The change in recorded fluid pressures due to lubricant pumping typically results in an maximum increase of about 30kPa (0.3 bar) above hydrostatic pressure. The time over which the pressure change occurs has been found to vary around the pipe. An example of this is illustrated in Figure 17 where the increase in

measured fluid pressures along the left of the pipe are shown.

At the front the pressure increase above hydrostatic is almost immediate and rises gradually to a steady state of about 70kPa; at the centre, there is a very small linear increase after commencement of pumping until about 15:35:30 when a sudden increase takes place; fluid pressure at the rear follows a similar curve to the centre but with a time lag of about 1 minute.

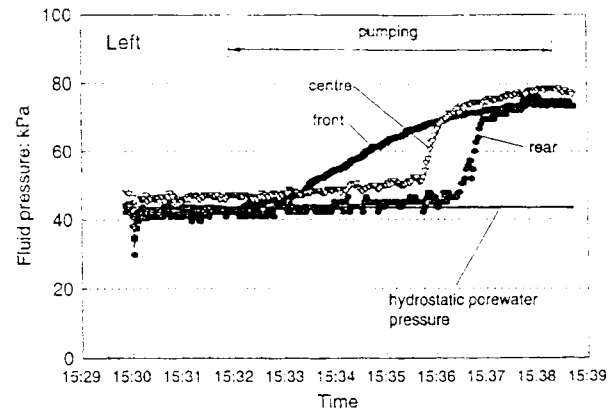


Figure 17 Change in pore pressures due to lubricant pumping - push 61.968 to 63.425m

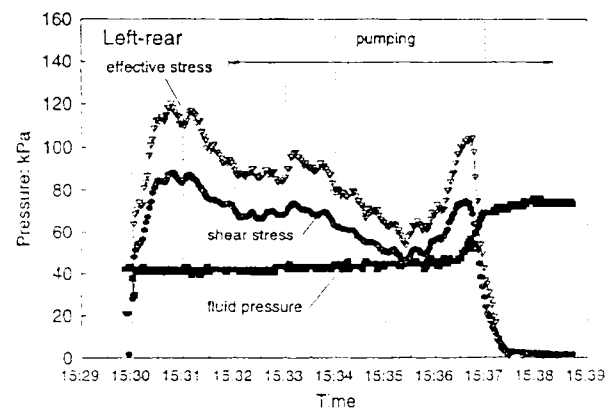


Figure 18 Effect of lubricant pumping at left-rear during the push 61.968 to 63.425m

During this push it was found that the increase in fluid pressure at the front-left location matched those recorded along the bottom and right of the pipe. Along the pipe top however, curve shapes similar to those shown for the centre-left and rear-left locations have been found but typically with a greater time lag. These observations are also true of other pushes before the original lubricant pump failed after jacking 31 pipes. This shows the introduction of fresh lubricating slurry, flowing from front to rear, had an easier flow path along the bottom, right and circumferentially around the front of the

instrumented pipe.

Figure 18 shows the reduction in shear and effective stress on the left-rear with the corresponding increase in fluid pressure. After the start of pumping the stresses decrease, somewhat erratically, until the increase from about 15:35:30 to 15:36:30. This is may be due to lubricant pressure from the right of the pipe forcing it against the soil on the left. As fluid pressures begin to rise sharply from 15:36:30, contact stresses fall dramatically to zero. As the change in fluid pressure reaches a steady state, indicating the formation of an effective lubricant zone, there is no further contact between pipe and soil.

Eleven pipes (a distance of 27.5m) were jacked without lubrication following the pump failure. After this break - from about distance 75m - contact stresses around the instrumented pipe increase and there is no significant reduction upon recommencement. The pore pressure records also show that pumping after the respite had no further effect which would suggest that fresh slurry was no longer able to reach the front of the pipeline. However, there was clearly still a low shear zone acting around some of the pipe as the stresses did not increase to give uniform contact around the right, top and left as one might expect if complete ground closure had occurred. The continued existence of a non-uniform low friction gel is not yet fully understood.

4.5.3 Predicting line friction

The self weight of the 1000mm pipe is equivalent to 8.1kN/m; the buoyancy uplift due to groundwater is 11.1kN/m, therefore the nett force acting on the crown of the tunnel wall is 3kN/m. The friction force due to pipe weight may be given by $W \cdot \tan \delta$. Using a measured friction angle of 38.7° , gives pipe friction of 2.4kN/m (0.6kPa). This agrees closely with measured friction from the jacking force record (see Table 2) when the line is fully pressurised with lubricant i.e. distance 0 - 22m and 106 - 126m.

A model for ground loading in cohesionless soil, detailed in Pipe Jacking Association (1995), gives the total frictional resistance, F , as:

$$F = \frac{\pi D}{2} (\sigma_v - \sigma_h) \tan \delta \quad (2)$$

where σ_v and σ_h are vertical and horizontal stresses respectively; D is the diameter of the

excavation and δ is the angle of friction. This gives frictional resistance, $F = 77\text{kN/m}$ (20kPa) for unlubricated conditions. There can be no direct comparison with measured friction from the jacking record as lubrication began at the start of the drive. However, during the break in lubrication, but with a soil-lubricant gel in the ground from earlier, the measured frictional resistance increased to about 42kN/m (11kPa).

4.5.4 General

Measured contact stresses vary greatly around the instrumented pipe as shown above. It is unclear as to whether recorded contact stresses are representative of stresses on pipes further back in the line. Taking a mean value of the all shear stresses in Table 3 gives a frictional resistance of 50kN/m (13kPa) which falls within the Craig limits for unlubricated wet sand (10 to 15kPa). The overall frictional resistance from the jacking force record is about 23kN/m (6kPa) which is less than half that given by contact stresses. It is assumed therefore that soil-pipe interaction at the front of this drive does not model the whole.

4.6 Face stability

A record of the slurry pressure used to maintain face stability is shown in Figure 19. This information has been extracted from the operator's driving records. Also depicted in the

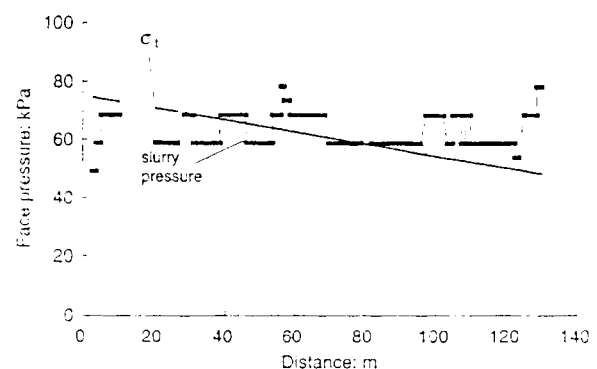


Figure 19 Variation of face pressure with distance

plot is the theoretical face support pressure, σ_t , for cohesionless soils. The method of analysis is described by Atkinson and Mair (1981) and gives:

$$\sigma_t = [\gamma \cdot (h - H_w) - \gamma_w \cdot H_w] \cdot D_s \cdot T_\gamma - \gamma_w \cdot H_w \quad (3)$$

where γ_w = unit weight of water (9.81 kN/m³),
 γ' = buoyant unit weight of soil,
 D_s = diameter of the bore,
 T_s = stability number (0.2 for this soil),
 h = height of soil above the crown, and
 H_w = height of water table above crown.

The calculated support pressures decrease linearly from 75kPa to 48kPa because cover depth decreases almost linearly. Hydrostatic conditions are assumed with a constant head, H_w . In practice however, dewatering was taking place around the reception shaft towards the end of the drive which would probably result in lower theoretical values than indicated. The average slurry pressure applied at the face was 63kPa, the average calculated value is 62kPa. Although there is good agreement between average values, the difference between the upper and lower bounds is probably due to assuming a constant head of water for the whole drive, whereas depth to groundwater level was monitored in the inclinometer access tubes only - located at a distance of about 134m. The slurry pressures may have exceeded the calculated values in this area by about 20kPa which is insufficient to cause outward movement or heave in this type of soil. Ground deformation was monitored as the machine approached and passed the inclinometer tubes, and no heave was detected.

4.7 Stoppages

After a stoppage, a weekend or overnight break for instance, the jacking forces required to advance the pipeline may be much larger. Norris (1992) and Marshall (1996), among others, have reported this phenomenon in cohesive soils.

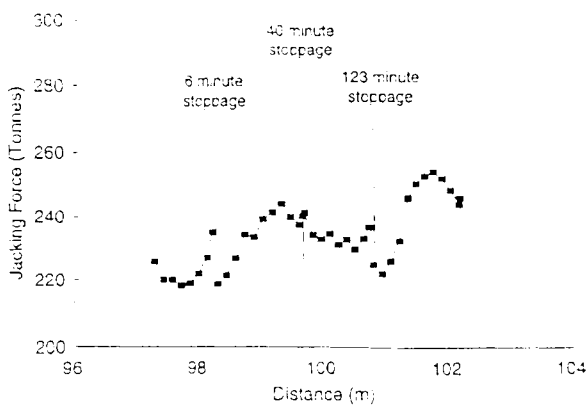


Figure 20 Effect of stoppages on jacking load

Data from this site show no significant increase in jacking forces needed to re-start pipeline

movement in the granular soil - see Figure 20. The plot shows typical jacking loads with delays for the introduction of new pipes.

5. GROUND MOVEMENTS

5.1 Introduction

The pipe jacking technique offers a good opportunity for limiting ground movements by installing a rigid lining immediately behind the shield. In urban areas where the technique is often used for the construction of new sewer pipes, as in this reported case, this is a particular advantage as ground movements may cause damage to buried services, road pavements and buildings. Provided there are no large displacements due to stress relief or face instability, pipe jacking should therefore offer a good approach to minimising ground movements - Milligan and Marshall (1995) report on measured ground movements on three monitored pipe jacks including this site.

5.2 Surface settlements

The most commonly used method for predicting a surface settlement profile is the Gaussian distribution curve (Peck 1969) - see Figure 21. Settlement, S_0 , is given by

$$S_0 = S_{0,max} \exp(-x^2 / 2i^2) \quad (4)$$

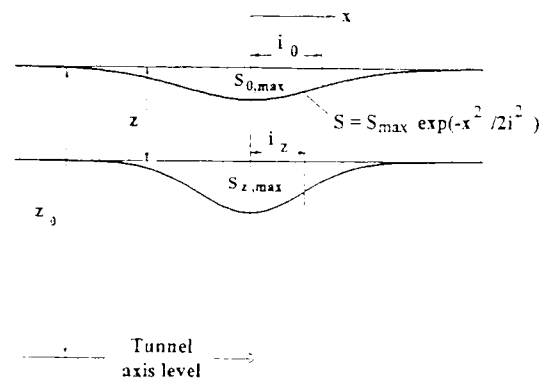


Figure 21 Calculation of settlements

where $S_{0,max}$ is the maximum settlement over the tunnel centre line and i is the distance from centre line to the point of inflexion. O'Reilly and New (1982) produced the following expression for granular soils:

$$i = 0.28z_0 - 0.12 \text{ (m)} \quad (5)$$

where z_0 is depth to tunnel axis.

The value of $S_{0,max}$ may be obtained by equating

the volume of the settlement trough V_s to the volume of estimated ground loss. It can be shown that

$$S_{max} = \frac{V_s}{i_s 2\pi} \quad (6)$$

In this paper, the calculations assume that ground movement at the tunnel face is negligible because the face was continually supported by slurry pressure. The only significant contribution to ground loss is due to the overbreak where the ground is excavated to a larger diameter, D_o , than the pipe diameter, D_p . It is therefore assumed here that

$$V_s = \frac{\pi}{4}(D_o^2 - D_p^2) \quad (7)$$

Equations (4) to (7) give a predicted profile as shown in Figure 22.

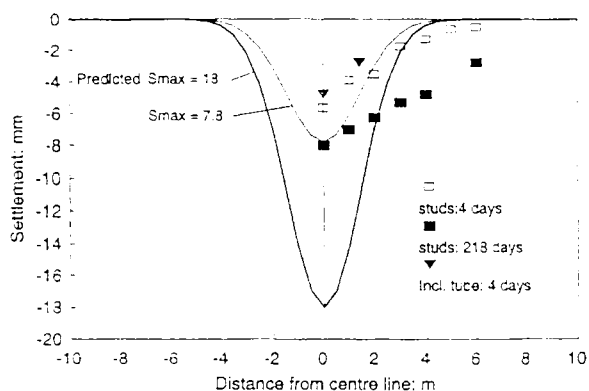


Figure 22 Surface settlements after completion

The maximum settlement of $S_{max} = 18\text{mm}$ assumes that the full overbreak of volume between cutter diameter and pipe outer diameter is converted into settlement. Observed movements four days after tunnel completion show a maximum settlement of only 6mm over the centre line but a distance to the point of inflexion close to the predicted value of 1.45m. There are two probable factors accounting for the discrepancy in magnitude of settlement:

i) Shearing of the soil during settlement will cause some dilation of the dense sand resulting in the volume of the settlement trough being less than the overbreak volume

ii) The bentonite slurry gel - used as a lubricant - prevents the ground completely collapsing onto the pipes.

An alternative calculation, giving $S_{max} = 7.8\text{mm}$, assumes that the difference in diameter between

cutter and shield gives the overbreak volume. Allowing for some dilation, this alternative predicted profile is in reasonable agreement with the observations made four days after completion. However, the calculation leaves some doubt over whether settlements continue as the bentonite/soil gel consolidate under pressure from the surrounding soil. In this case, observations made two hundred and eighteen days after completion show a further settlement over the centre line of about 2mm; the profile however, is much flatter than predicted probably due to the stiff road pavement. More recent measurements indicate no further increase in settlement.

5.3 Subsurface movements

Deformation of the inclinometer tubes was negligible - typically less than 1mm - and showed no consistent pattern. The magnetic settlement plates however, did indicate some downward movement around the tubes. The vectors shown in Figure 23 illustrate the plate movement.

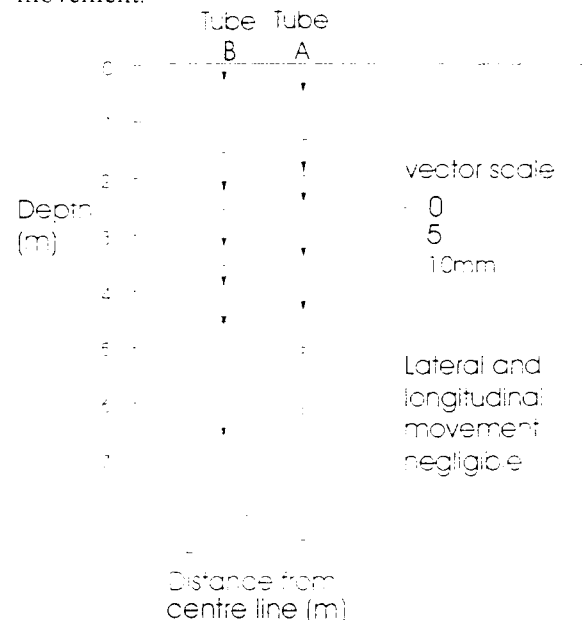


Figure 23 Ground movement vectors

6. CONCLUSIONS

1. The instrumented pipe, and probably the whole pipeline, was floating in the lubricant filling the tunnel bore.
2. Interface stresses around the instrumented pipe, are probably not typical of stresses along the whole pipeline, with exception to those recorded along the pipe bottom.
3. The use of lubrication and its

effectiveness in completely filling the overbreak void is the most important construction related factor in line friction loads and ultimately jacking forces in this monitored drive.

4. The position of lubricating points close to the instrumented pipe resulted in a lubricant gel around the pipe of variable effectiveness. Providing more injection points, alternating between right, top and left might have resulted in a more even distribution of gel around the pipe and consequently, lower contact stresses.
5. Measured friction angles are in close agreement with that given by Potyondy (1961).
6. The Terzaghi model for ground loading in cohesionless soil gives an upper bound for line friction loads.
7. A lower bound for the line friction load is about 3kN/m (0.8kPa) where line is fully lubricated and the pipes are floating.
8. Misalignment effects were negligible. A factor of safety of 1.2 should normally be used to allow for misalignment when calculating jacking forces.
9. The well-known empirical approach of calculating surface settlements predicts a reasonable profile but of a greater magnitude than measured.
10. Sub-surface settlements were very small indicating an accurately balanced face pressure as the machine passed the instrumentation array.

7. ACKNOWLEDGEMENTS

The site referred to in this paper and the cost of ground instrumentation were provided by North West Water, Markham, who manufacture tunnelling machinery, gave assistance and time free of charge when calibrating thrust rig hydraulic pressures against load cells. Inclinometer access tubes were installed by Georesearch.

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Movements and stress changes in London Clay due to the construction of a pipe jack

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ABSTRACT: A pipe jacked tunnel in London clay was monitored in 1993 as part of a programme of research into the technique at Oxford University. Measurements of jacking forces, radial and shear stresses at the pipe-soil interface, stresses in the ground and ground movements were made. Changes in ground stresses due to construction of the tunnel are presented and measured ground movements are compared with settlements predicted using empirical methods.

1. INTRODUCTION

1.1 Pipe jacking

Pipe jacking is a technique used to form small diameter tunnels by pushing a 'string' of pipes through the ground from a thrust pit to a receiving pit. Hydraulic rams in the thrust pit jack the pipes forward as the ground in front of the pipeline is mined. Methods of excavation range from hand excavation within a shield - as depicted in Figure 1 - to full-face tunnel boring machines. Slurry pressure machines or earth pressure balance machines would be used in poor ground to support the tunnel face.

1.2 The Oxford research project

A major research project into pipe jacking has been in progress at Oxford University for the past nine years. The project is supported by the Pipe Jacking

Association, the Engineering and Physical Sciences Research Council, and five U.K. water companies (North West, Northumbrian, Severn Trent, Thames and Yorkshire). Milligan and Norris (1993) describe the programme of research; results from the first two stages of work are summarised by Milligan and Norris (1994). The current Stage 3 of the research and the previous Stage 2 have monitored actual pipe jacks under construction by incorporating a specially-instrumented pipe into the pipe string. Ground movements during and after construction have also been recorded. Up to the present, four sites have been monitored during Stage 3; this paper will present results from ground movements and stresses within the London Clay in the first of these sites.

2. SITE DETAILS

2.1 Jacking method

The first of the sites on which Stage 3 monitoring work was carried out was in Leyton, East London during November and December 1993. It was on the Fillebrook Surface Water Sewer Relief Scheme for Thames Water; contractor for the tunnelling work was M. J. Clancy. The pipe jack monitored was along Wailwood Road, Leyton. It was a 75m long drive wholly within London clay, depth to axis was approximately 8.5m and the pipe size was 1.5m internal diameter, 1.8m outer diameter. Excavation was carried out by hand using pneumatic tools within a simple steerable shield.

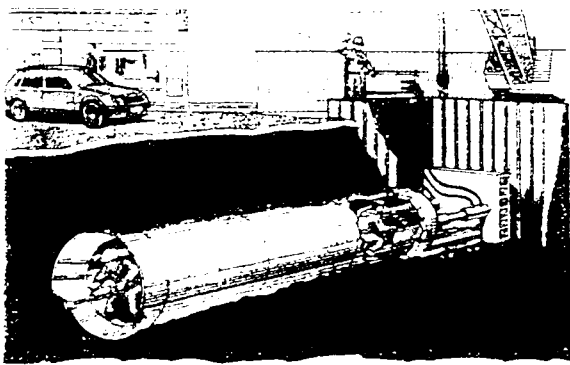


Figure 1. Elements of a hand excavated pipe jack (courtesy of the Pipe Jacking Association)

2.2 Ground conditions

2.2.1 Client's site investigation

A site investigation for the client was carried out around the area in November 1992. This comprised a series of five boreholes to depths of between 5.0 - 12.0 m along the centre line of proposed tunnels. Representative disturbed and undisturbed samples were taken at regular intervals or at changes of stratum. Relatively unweathered highly plastic London clay, composed of stiff grey fissured silty clay with sandy laminations was encountered at depth. The undrained shear strength of the material was classified as essentially firm to stiff, becoming stiff at depth (see Table 1 and Figure 2).

Table 1 Results of laboratory tests

z m	Description	Triaxial Compression			
		total stress kPa	bulk stress kPa/m	total strain %	shear strain kPa
5	Soft orange to brown light sandy silty CLAY				
2.5	Firm to stiff orange-brown silty CLAY with pockets of sandy silt re traces of medium gravel	3	2.17	1.0	27
1.5	WATER				
1.5	Stiff brown blue-grey cement fissured silty CLAY with occasional pockets of sandy silt and shales	4	1.98	2.72	101
	Stiff to very hard grey cement fissured silty CLAY with occasional pockets of sandy silt	2	1.77	2.73	111
7.5	Stiff to very hard silty CLAY with occasional pockets of soft fine sand	14	1.34	2.17	213

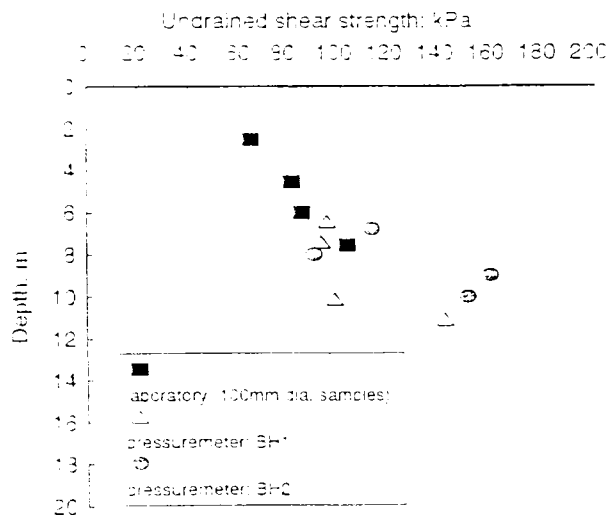


Figure 2 Variation of undrained shear strength with depth

2.2.2 Pressuremeter testing

Cambridge Insitu were employed by the Oxford research project to carry out a series of self-boring pressuremeter tests prior to any pipe jacking. Eight undrained expansion tests were carried out in two boreholes close to the proposed tunnel axis to a maximum depth of 11m. Results from the pressuremeter tests are summarised in Table 2. The undrained shear strengths determined from the tests are also shown in Figure 2. At tunnel axis level - about 8.5m - the total horizontal stress measured was about 330 kPa and the initial shear modulus, G_0 , averaged about 50 Mpa. A more reliable method for obtaining an estimate of the shear modulus is to take the slope of the chord of the unload/reload loops. Average values from the unloading/reloading tests are $G_{ur1} = 23$ MPa at an amplitude of 0.38% cavity strain, and $G_{ur2} = 20$ MPa with an amplitude of 0.46% cavity strain.

Table 2 Results of pressuremeter tests

Test	z (m)	P_0 kPa	s_u kPa	G_0 MPa	G_{ur1} MPa	G_{ur2} MPa	P_L kPa
B101	6.4	287	44	4	18	14	307
B102	7.4	327	42	55	27	20	193
B103	10.1	320	37	50	26	22	1112
B104	11.1	340	34	51	28	23	1121
B201	6.8	328	112	35	20	18	130
B202	8	310	80	45	17	18	1111
B203		28	30	4	23	2	1134
B204		212	3	45	24	23	1103

P_0 = total horizontal stress G_0 = initial shear modulus
 G_{ur1} = shear modulus from unload/reload loop (number "1")
 G_{ur2} = shear modulus from unload/reload loop (number "2")
 s_u = undrained shear strength P_L = limit pressure

3 INSTRUMENTATION

Ground deformation was monitored using an array of inclinometer access tubes with magnetic settlement plates - depicted in Figure 3. Surface settlements were measured using precise levelling techniques on a series of studs driven into the road pavement.

Two push-in pressure cells with pneumatic transducers were installed at the locations indicated in Figure 4. The thin blades of the spade cells were positioned to record in-situ total stresses and pore water pressures perpendicular to the line of the tunnel as the pipe jack approached and passed.

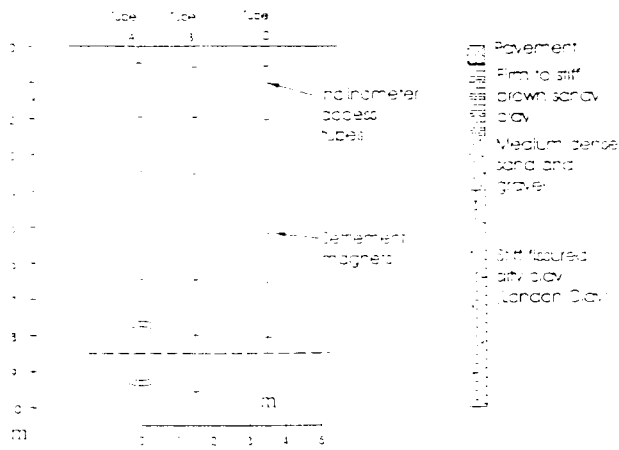


Figure 3 Inclinometer access tubes

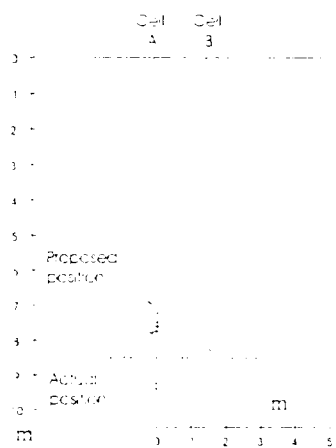


Figure 4 Push-in pressure cells

4 GROUND MOVEMENTS

4.1 Introduction

Pipe jacking has the advantage of lining the tunnel immediately behind the shield. Because jacking pipes have sufficient strength and stiffness to withstand jacking forces, their deformation due to ground loading will be negligible. The technique therefore should minimise ground movements, provided no large displacements occur due to instability at the tunnel face or stress relief. Milligan and Marshall (1995) report on ground movements measured on this site as well as two further sites.

The overcut on this tunnel was up to 50mm in places and the void was not usually completely filled with bentonite slurry to support the ground. The subsequent closure of this void will result in some settlement. This section will present measured ground movements and compare them to predictive methods.

4.2 Surface settlements

The most commonly used method for predicting a surface settlement profile is the Gaussian distribution curve (Peck 1969) - see Figure 5

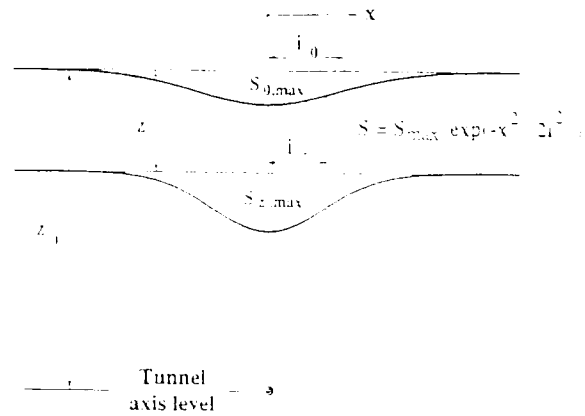


Figure 5 Calculation of settlements

Settlement, S_0 , is given by

$$S_0 = S_{0,max} \exp(-x^2 / 2i^2) \quad (1)$$

where $S_{0,max}$ is the maximum settlement over the tunnel centre line and i is the distance from centre line to the point of inflexion. O'Reilly and New (1982) produced the following expression for i in cohesive soils:

$$i = 0.43z_0 + 1.1 \quad (m) \quad (2)$$

where z_0 is depth to tunnel axis. Mair *et al* (1993) show that equation (2) can be reasonably approximated to $i = 0.5z_0$ for subsurface movements they present data showing the ratio $(i/z_0 - z_0)$ increasing with depth.

The value of $S_{0,max}$ may be obtained by equating the volume of the settlement trough, V_s , to the volume of estimated ground loss. It can be shown that

$$S_{0,max} = \frac{V_g}{L_s 2\pi} \quad (3)$$

In this paper ground loss due to movement at the face is assumed to be negligible as the open face in the stiff clay was highly stable with only very small movements due to elastic unloading occurring. The only significant contribution to ground loss is due to the overcut - the ground being excavated to a larger diameter, D_o , than the pipe diameter, D_p . It is therefore assumed here that

$$V_g = \frac{\pi}{4} (D_o^2 - D_p^2) \quad (4)$$

The overcut was provided by the miner excavating the clay with a clearance around the shield - varying from about 10mm at the crown to a maximum of about 50mm at the shoulder and zero at the invert. The

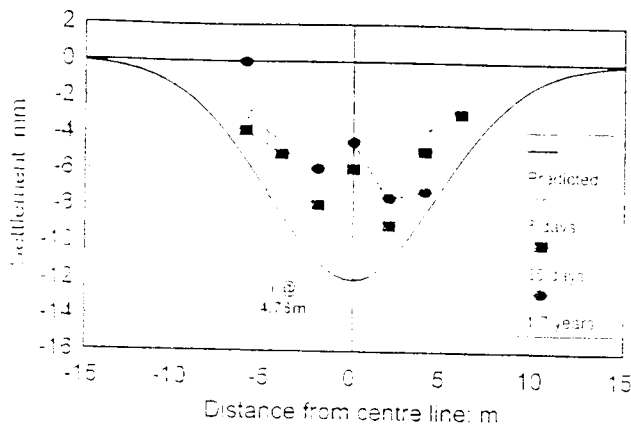


Figure 6 Surface settlements after completion

average overcut value is taken as being about 25mm. Using equations (1) to (4) gives a predicted profile as shown in Figure 6 with a maximum surface settlement of 12mm over the centre line and a value of 4.76m for x .

The measured values taken soon after completion of the tunnel, indicate a similar settlement profile except that the centre line value is rather less than those either side, though all lie within the calculated settlement profile. The final set of measurements on the road studs - taken almost two years after completion - show a consistent profile but with levels slightly higher than expected. This can probably be attributed to local movements in the road pavement, either by traffic damage or temperature and seasonal movement.

4.3. Subsurface movements

Subsurface movements were monitored using a uniaxial inclinometer torpedo and a reed switch probe triggered by the magnetic settlement plates. Depths to the settlement plates were taken relative to the top of the access tubes with levels taken on the tube tops at about the same time. It is assumed that the access tubes are fixed at the top - grouted into the road pavement - because financial constraints prevented the tubes from being sunk to a depth for bottom fixity. Access tube profiles perpendicular to the tunnel axis are plotted in Figure 7, showing horizontal movements of 5mm at about the level of the tunnel.

The final readings show similar profiles but with slightly greater or lesser movements. This could be attributed to small movements in the assumed fixed datum of the tube tops. Data from the inclinometer and settlement plates have been combined for measurements taken about one month after completion to give the ground movement vectors shown in Figure 8. The vertical movements for tubes B and C appear to

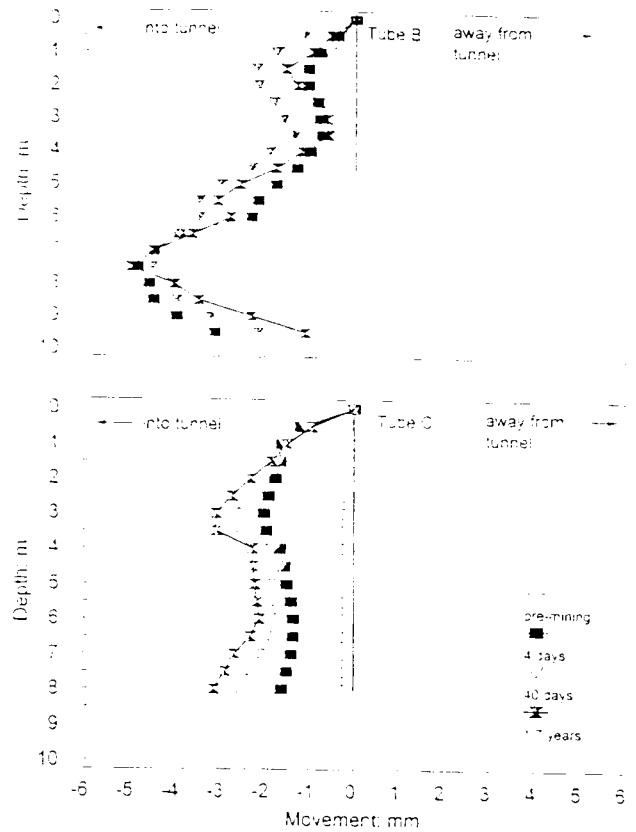


Figure 7 Change in inclinometer tube profile after completion

be inconsistent, possibly due to initial levelling errors to the top of the tubes.

Space limitations for this paper prevent the presentation of movements in the direction of the drive. Inclinometer data - taken forty days after completion - indicate maximum movements into the excavation of about 2mm in tubes A and B (over the centre line and offset by 1.5m respectively), and about 1mm for tube C (offset by 3.5m).

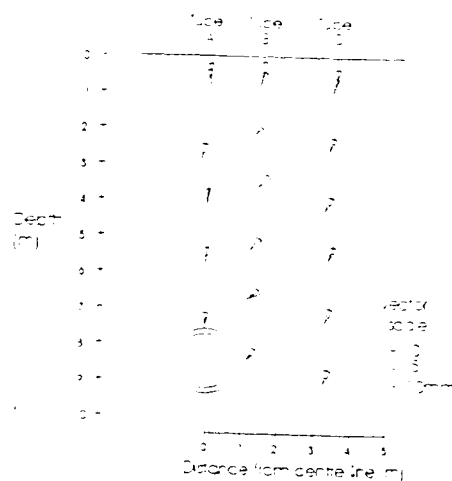


Figure 8 Movement vectors after completion

4.4 Comparison with deeper tunnels

Measurements of subsurface ground movements during construction of five London Underground tunnels of approximately 4m in diameter, at depths of 20-29m, in London Clay were presented in dimensionless form by Mair and Taylor (1993), as shown in Figure 9. The vertical ground movements were measured at different distances above the tunnel centre-line and the horizontal movements were measured at various positions at tunnel axis level. The data are reasonably consistent and are in general agreement with linear plots. Mair and Taylor (1993) show that the gradient of the lines in Figure 9, which are almost parallel, is consistent with a G_s/γ_s ratio of about 100; this assumes idealised linear elastic-perfectly plastic soil behaviour.

Figure 10 shows observed subsurface ground movements around the pipe jack plotted in the same dimensionless form as in Figure 9. It is interesting to note that the gradient of the lines through the pipe jack data are similar to those for the deeper and larger tunnels. Assuming an undrained shear strength of about 100kPa at the 8.5m depth of the pipe jack (see Figure 2), the stability ratio γ_s/s_u is 1.7, compared with an average value of about 2.5 for the deeper tunnels. Taking this into account, and assuming a complete unloading of the cylindrical cavity in the case of the pipe jack, a G_s/γ_s ratio of about 80 is implied by the data in Figure 10. This compares favourably with the G_s/γ_s ratio of about 100 deduced from the movements around larger and deeper tunnels in London Clay.

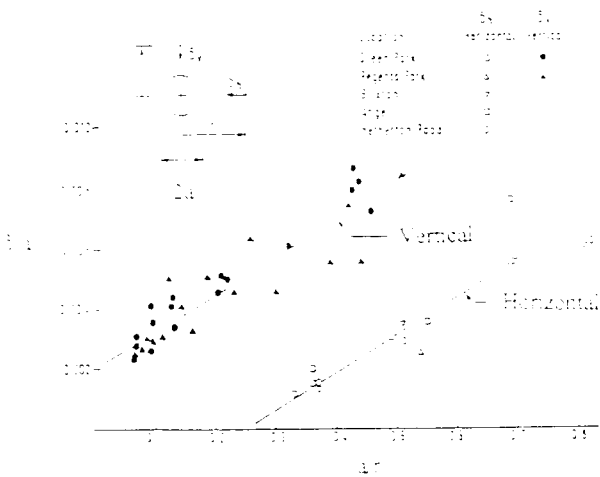


Figure 9 Vertical and horizontal sub-surface movements in the vicinity of 4m diameter tunnels in London Clay at depths of 20-29m (after Mair and Taylor, 1993)

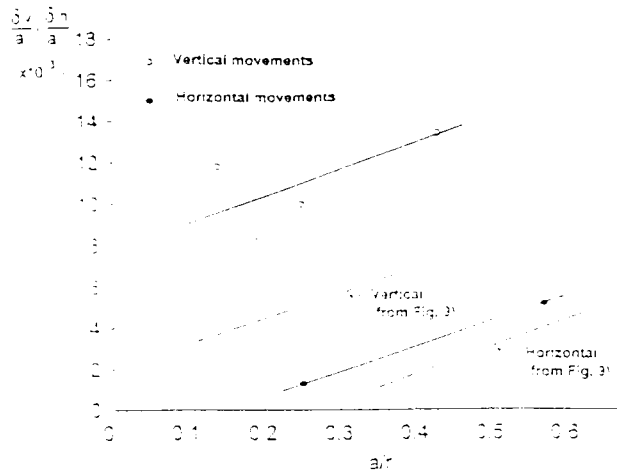


Figure 10 Comparison of vertical and horizontal sub-surface movements around the pipe jack with deeper 4m diameter tunnels

5 GROUND STRESSES

The readings from the push-in pressure cells are shown in Figure 11. After installation, the readings stabilise and show horizontal stresses of about 400kPa - a factor of about 1.2 greater than the total horizontal stresses given by the pressuremeter tests at the same depths. Tedd *et al* (1984) suggest that in this type of ground, the cells may over-read by approximately half the undrained strength of the clay. However, more recent work by Ryley and Carder (1995) recommend 0.8 s_u as a best-fit correction for stiff clays, which gives better agreement between the pressure cell readings and the pressuremeter tests.

As the shield approached, both cells show an increase

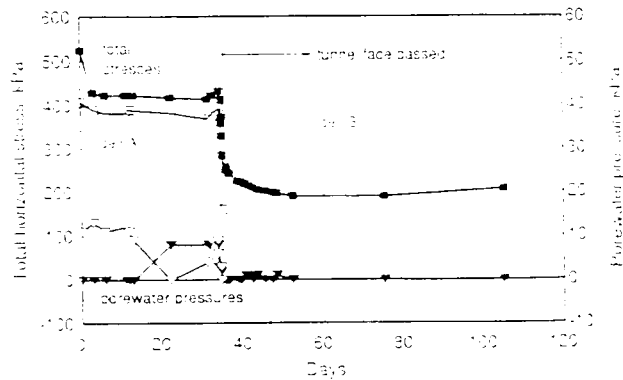


Figure 11 Variation of total stress and porewater pressure with time

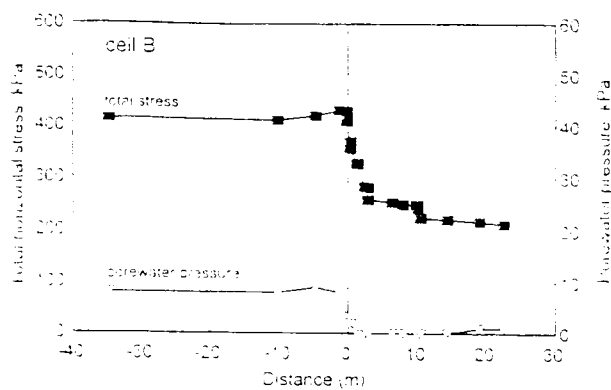


Figure 12 Variation of total stress and porewater pressure with distance to face in cell B

in stress of about 18kPa. This may be due to the soil arching in front of the tunnel excavation. Cell A, intended to be just above the crown of the tunnel, was exposed at the face having been installed at the incorrect level - it recorded a drop of about 240kPa to a final reading of 160kPa.

Readings from cell B, 1.5m from the tunnel axis, are shown in Figure 12 - the horizontal axis now showing the distance between the face and the cell. It shows a drop in stress of 60kPa from 430kPa to 370kPa as the tunnel face passed. The readings then decrease further with time to about 215kPa - a total drop of 215kPa. Elastic analysis of the stresses around an open bore - see Poulos and Davis (1991), with initial vertical and horizontal stresses of 190 and 330kPa respectively, predicts a drop in horizontal stress at the location of the cell of 167kPa. The greater drop recorded in practice may be the result of local plastic yielding around the opening.

6. CONCLUSIONS

The principal conclusions are:

- i) Surface settlements are consistent with expected distribution but magnitudes are somewhat smaller than predicted.
- ii) The subsurface ground movements were found to be in good agreement with the pattern of behaviour observed for deeper and larger tunnels in London Clay.
- iii) There is reasonable agreement between pressuremeter and spade cell readings after allowing for installation. The reduction in horizontal stress is rather greater than that predicted by elastic analysis.

7. ACKNOWLEDGEMENTS

The site referred to in this paper was provided by Thames Water. The tunnelling contractor was M J Clancy & Sons whose staff were most cooperative. Inclinometer access tubes and push-in pressure cells

were installed by Soil Instruments Ltd.; pressuremeter testing was done by Cambridge Insitu.

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The functions and effects of lubrication in pipe jacking

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ABSTRACT: The use of lubricating fluids around pipe jacks to help reduce the jacking loads can be extremely effective, but only if their function and effects are properly understood and they are properly used. In cohesionless soils, their first purpose is to prevent the ground from collapsing onto the pipes and hence maintain a small annulus of overcut around the pipes. The fluid must be able to form a filter cake in the surrounding soil, and must be sufficiently pressurised to support the ground. In a stable bore, the pipes will then become wholly or partially buoyant and the frictional resistance between pipe and ground will be reduced. In very soft clays, and in stable excavated bores in stiff cohesive soils, the introduction of lubrication may be delayed until the jacking force approaches its limit, and when introduced will still be effective over the full length of the pipe line.

1 INTRODUCTION

It is well established practice to use lubricants to reduce the jacking forces during construction of pipe jacked tunnels. These may be bentonite slurries or polymer materials, or a combination of the two. They are usually injected through ports in the pipes into the small overcut space formed by excavating, whether by hand or using a tunnelling machine, to a slightly greater diameter than the external diameter of the pipes. However the true functions of the lubricant are not always understood, and as a result it is not always used as effectively or economically as it might be. This paper considers the functions of lubricants and provides illustrations of their effects in different ground conditions drawn from field measurements made during a major research programme.

2 PIPE-SOIL INTERACTION

The interaction between pipes and soil during pipe jacking is complex. It is of considerable importance since it controls the build-up of jacking force as the increasing length of pipe line is jacked forward. The maximum allowable force may be controlled by pipe strength, jack capacity, or thrust wall resistance. If it is likely to be exceeded before a drive is completed, measures such as the introduction of interjack stations or the use of lubrication must be considered.

Pipe-soil interaction has been investigated as one aspect of a major research project based at Oxford University. Nine full-scale pipe jacks have been

monitored during construction; measurements have been made of the overall jacking force, using load cells on the jack rams, and of local radial and shear contact stresses between pipe and ground, using stress cells incorporated into the walls of an instrumented pipe. On several of the schemes lubrication was used for all or part of the monitored drive.

The effects of lubrication will be considered in terms of four simplified models of pipe-ground interaction, which have been shown by the research work to be applicable in appropriate ground conditions. The first two apply when the excavated tunnel is stable and the pipes are simply able to slide along the base of the bore, as shown in Figure 1. These

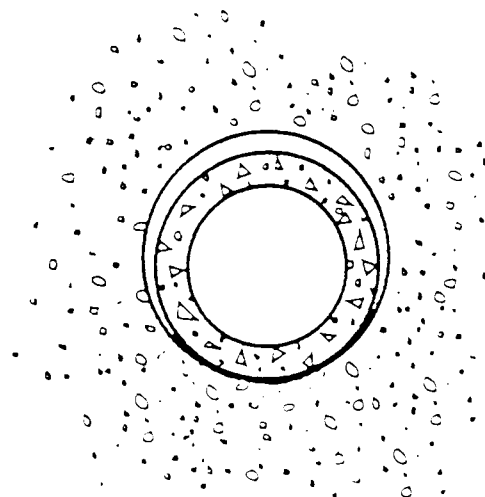


Figure 1 Pipes sliding in a stable bore

Table 1. Interface friction angles (δ) in various ground conditions

Soil type	δ ($^\circ$)
London Clay (high plasticity)	13
Stiff glacial clay (low-medium plasticity)	19
Weathered mudstone	17
Loose to medium dense sand and gravel	32
Dense silty sand	38

conditions apply either in cohesive soils of adequate undrained strength, or in fine sands above the water table when capillary suctions are sufficient to prevent the ground collapsing. The stability of the excavated face and bore may be checked using analyses developed for normal tunnelling methods (Atkinson and Mair 1981). These conditions were apparent in the field work when contact stresses between pipe and ground were only measured at the bottom of the pipe. The resistance to sliding will then usually be frictional in nature, and given by the weight of the pipes multiplied by a suitable interface friction coefficient between pipe and soil. Values of the friction angle measured for concrete pipes in various ground conditions are given in Table 1.

Alternatively, in softer clays, the interaction may be more cohesive in nature, the resistance over a contact width between pipe and ground being given by αs_u , where s_u is the undrained strength of the ground and α a reduction coefficient similar to that used in the design of piles. With both these models it has been found that the total jacking force in practice is typically 10 to 25% higher than calculated, depending on the straightness of the drive.

Similarly there are two models for pipes in unstable ground. In very soft clays the ground will close around the pipe and the resistance to jacking will then be given by the remoulded undrained strength of the soil acting over the full external surface of the pipe. In cohesionless soils the ground will also collapse onto the pipes. In this case the radial contact stresses can be calculated using a "pipe-in-trench" analysis based on the conditions originally proposed by Terzaghi (1943) as shown in Figure 2. Resistance to jacking is then obtained by multiplying these contact stresses by the appropriate value of $\tan \delta$ (Table 1). The resulting resistance can be very large, and only short lengths of pipe can be jacked without jacking forces becoming excessive. It is in this situation that the use of lubrication has the greatest benefit, but its primary function is to support the ground and prevent it collapsing onto the pipes.

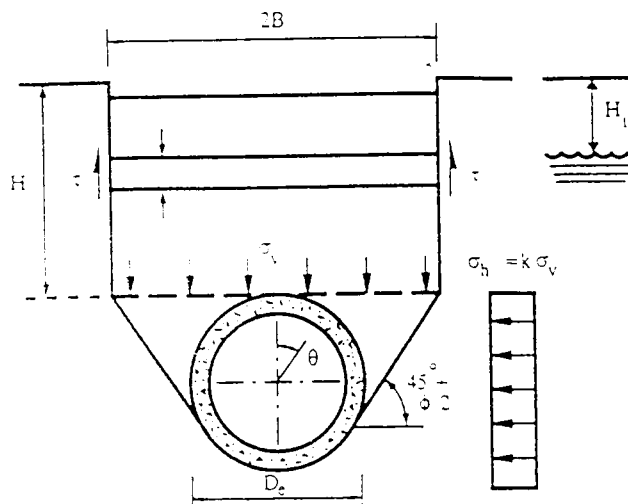


Figure 2 Model for ground loading in cohesionless soil after Terzaghi (1943)

3 STABILISATION OF COHESIONLESS SOILS

This paper is concerned with the use of stabilising slurry along the pipe line; however it should also be realised that the tunnel face must be kept stable (by slurry pressure, earth pressure balance, face boards etc) to prevent inflow of soil and excessive ground movements. Atkinson and Mair (1981) have provided theoretical solutions for the pressures needed to support a tunnel bore in cohesionless ground, for tunnels at normal depths below the surface

$$\sigma_r = \gamma D T_s \quad (1)$$

where σ_r is the support pressure, γ the unit weight of the ground, D the tunnel diameter, and T_s a stability coefficient given in Figure 3. Below the water table the submerged unit weight of the ground applies, and the water pressure must be added to obtain the total pressure.

The lubricating slurry must be able to form a filter cake in the ground, then be pressurised

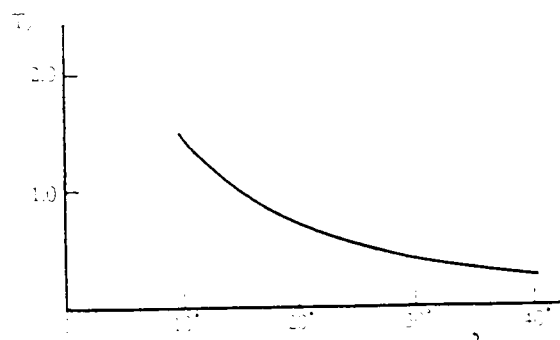


Figure 3 Stability coefficient for tunnel in cohesionless soil

sufficiently to provide the necessary support. The pressure required is not very great, except where the ground water pressures are large.

4 PIPE BUOYANCY

For a pipe in a stable bore, whether in naturally stable ground or ground made stable by fluid pressure, the effective weight of the pipe will be reduced by uplift forces due to buoyancy in any fluid in the overcut. With typical jacking pipes, particularly the larger sizes, in bentonite-based slurries a little denser than water, the uplift force will usually exceed the weight of the pipe if the overcut is completely filled. The pipe will then be lifted and forced against the roof of the tunnel. This has been observed on several sites, both from level surveys in the pipe line before and after the introduction of lubricant and from the contact stress cells. The cells at the top of the pipe show significant effective radial and shear stresses, while those at the bottom register only fluid pressure from the slurry, as shown in Figure 4

The resistance to jacking will now be greatly reduced. A perfectly lubricated pipe jack would be one in which the pipe was "weightless" due to buoyancy, and the overcut around the pipe was filled with lubricating fluid. The resistance would then only be the shear strength of the lubricant. In a fresh bentonite slurry, sheared at the rate typical of pipe jacking, the shear resistance is negligible. In practice the slurry is likely to become contaminated by soil as the pipes scrape against the sides of the tunnel bore and its shearing resistance increased. Shear stresses at the interface of up to about 2 kPa have been measured by the stress cells. On a pipe of say 1500 I.D. and 1800 O.D., with an external circumference of 5.65 m, the resistance to jacking would then be 11.3 kN per metre length of pipe line. For a typical jacking force of up to 5000 kN, a length of over 400 m could be jacked without use of an interjack.

For a pipe with positive buoyancy, a friction force between pipe and ground would be developed due to the nett upward force. For instance, the same pipe as above would have a weight of about 20 kN/m and displace slurry with a weight of about 27 kN/m. The nett contact force would be 7 kN/m; the effect of buoyancy would be to reduce the frictional resistance by about 65%.

A final effect of the lubricant may be to reduce the interface friction coefficient between pipe and soil. In coarse soils this effect appears to be small; presumably the contact is between the concrete surface and the surfaces of projecting particles of soil, with any lubricant scraped away. However in silty fine sand a reduced interface friction angle of 15° was measured on one site. In this case it is probable that a boundary layer of mixed silt, sand and lubricant was developed.

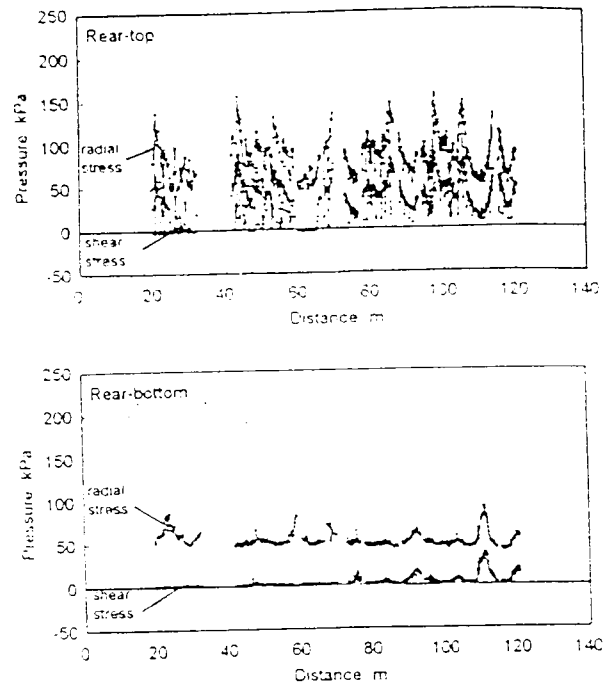


Figure 4 Contact stresses on buoyant pipe

5 SITE JACKING RECORDS

5.1 Introduction

Jacking records from some of the monitored projects will now be presented to demonstrate the effects of lubrication in different ground conditions. These records show the total jacking force at the thrust pit, measured by the load cells on the jack rams. They generally show an approximately linear increase in jacking force with the length of the drive, this being interpreted as the frictional resistance per metre length of pipe. In addition, there is normally an initial intercept or "face resistance", representing the forces acting on the shield or tunnelling machine. This may be negligible, as in a hand drive in which the miner is excavating the ground to a slightly greater diameter than that of the shield. Alternatively, it may be quite large, due to pressure on the face of a slurry- or EPB-type tunnelling machine, or if a shield is being driven into the ground to trim the excavated face.

5.2 Cohesionless soils

Figure 5 shows a classic case of a tunnel in cohesionless ground, sand and gravel below the water table, driven using a slurry-type tunnelling machine. No lubricant was used for the first 20 m of the drive, and the frictional resistance was about 100 kN/m. Thereafter full bentonite slurry support and lubrication was provided. The 1200 I.D. pipes became buoyant and were observed to lift by about 20mm, the size of the overcut, the nett force against the roof of the drive was then small, only 3 to 4 kN/m, and the total

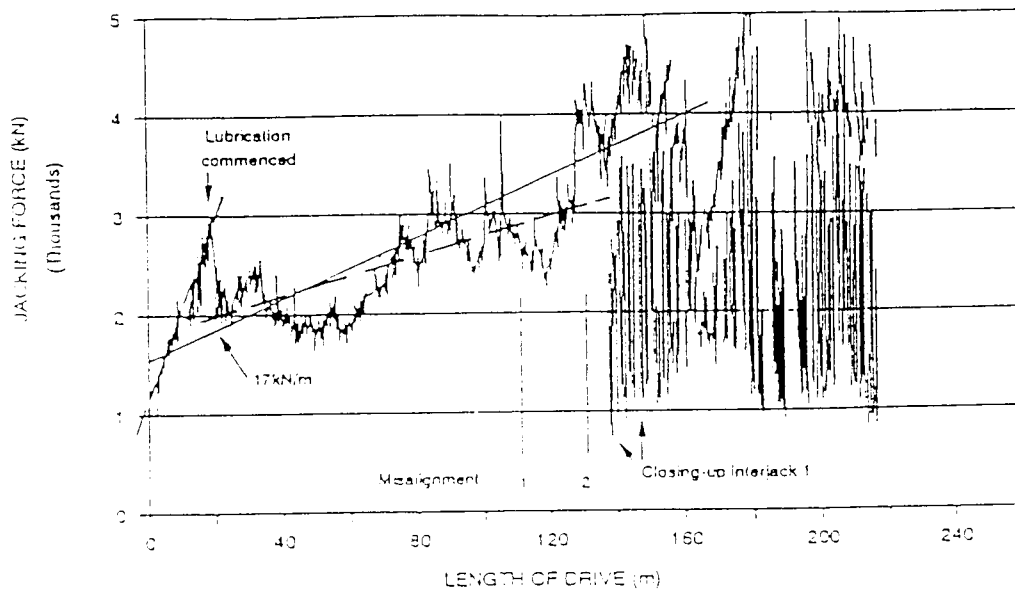


Figure 5 Jacking force record: sand and gravel

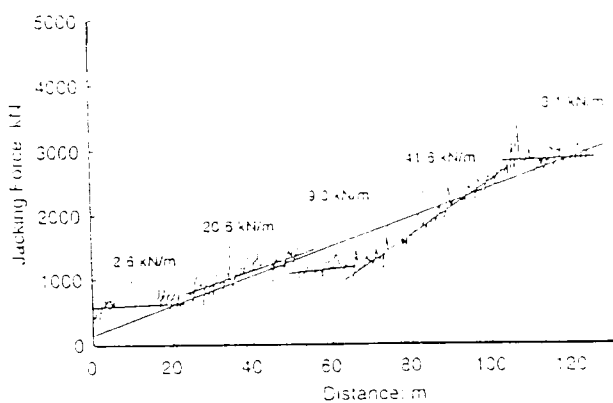


Figure 6 Jacking force record: fine sand

frictional resistance became about 10 kN m.

For the final 80 m of the 215 m long drive, an interjack was used, and it is interesting to note that when the interjack was being operated, the thrust was largely transferred back to the jacking pit and recorded by the load cells on the jacks. This showed that little resistance was being provided by the first 135 m of pipe line. It is also important to note that, even though the lubrication was introduced along the whole length of the pipe, the frictional resistance of the first 20 m was not reduced to the fully lubricated value. Once cohesionless soil has collapsed onto the pipe and closed the overcut annulus, it will not normally be possible to force the soil away from the pipe and establish a proper lubricating layer. It will therefore normally be beneficial to introduce lubrication as early in the drive as possible, though precautions will have to be taken to seal around the pipe at the jacking shaft to prevent loss of slurry there.

A second drive in cohesionless soil below the water table, but this time through fine sand, is

illustrated in Figure 6. This record is of interest because a lubricating system was in use from the start, but due to trouble with a pump was only fully operational at the beginning and end of the drive, and intermittently at other times. As in the previous case, there was a factor of ten or more in the frictional resistance between the properly-lubricated sections and those with little lubrication. The lower absolute values reflect the smaller diameter of the pipe in this case. Again it is clear that there was little if any reduction in previously-established friction when the lubrication system came back into operation.

5.3 Cohesive soils

Figures 7 and 8 show two examples of jacking records for pipe jacks through cohesive soils. The former relates to a drive in stiff glacial till (boulder clay). Progress on this drive was quite slow because of frequent delays while large boulders were dug out or broken up. Lubrication was introduced after the first 20 m. During the next 15 m, the jacking force steadily reduced, and then increased throughout the remainder of the drive at a lower rate. The reduction in resistance from unlubricated to average lubricated resistance in this case was nearly 70%. Since the tunnel bore was naturally stable and the overcut annulus had remained open, it was obviously possible for the lubrication to affect the full length of the pipe line. The fact that this did not happen instantaneously is probably a reflection of the time taken for the overcut to fill with slurry and the pipe become buoyant.

The very obvious peaks in the jacking record coincide with weekends during which no jacking took place. Surveys and stress cell readings indicate that some of the slurry pressure and pipe buoyancy was lost over these periods and the floating pipe tended to settle back

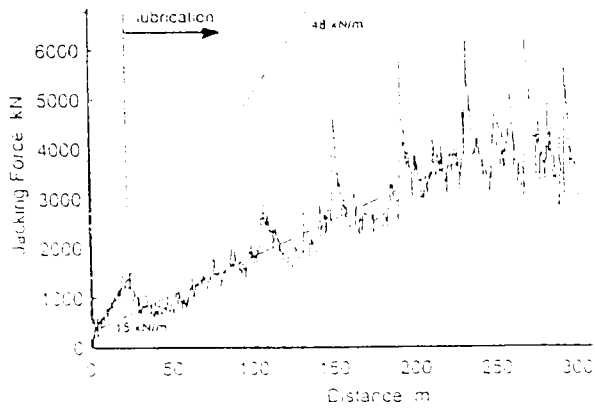


Figure 7 Jacking force record: stiff glacial clay

onto the base of the tunnel. In each case it took further jacking accompanied by injection of lubricant to reduce the jacking resistance to its lubricated value. Had the overcut been kept fully topped up with slurry at all times, a jacking resistance close to the lower bound of the measured resistances, of about 11.5 kN/m, would have been achieved. This represents an average shear resistance of about 1.7 kPa on the external surface of the 1800 mm I.D. pipes, and a drop of over 75% from the unlubricated value. At the end of the drive the jacking force became very erratic, but the average increase in resistance dropped to about 2 kPa; the reason for this is not at present clear.

Figure 8 shows the jacking record from a drive in very soft alluvial clay and peat. The interface stress cells showed that the ground had generally closed around the pipe. The initial resistance was about 25 kN/m, equivalent to 4.4 kPa over the full surface area of the pipe, consistent with the fully remoulded undrained shear strength of the clay. Lubrication was introduced when a large jacking force was needed to restart the pipes after a long weekend break. Within some 25 m of further driving the average resistance along the full length of the pipe line reduced to about 14 kN/m, a drop of some 44%; the average shear stress on the pipe surface was then 2.4 kPa. It is interesting that the lubricant was able to be equally effective over

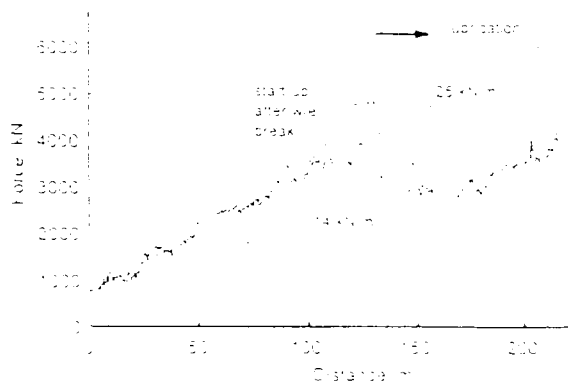


Figure 8 Jacking force record: soft clay and peat

both the originally unlubricated length and the newly excavated length; presumably the slurry was able to some extent to displace the soft clay in contact with the pipe.

Use of lubricants on pipe jacks through heavily overconsolidated, highly plastic soils needs further investigation. In these soils, plastic yielding and softening of the soil around the tunnel excavation will occur due to the relief of stresses in the ground. The ground may then close onto the pipe and develop high contact stresses, leading to high jacking resistance despite a low interface friction angle. Use of water or aqueous slurries to induce buoyancy or act as a lubricant may be counter-productive, since the presence of free water will accelerate the swelling process. The use of slurries containing chemicals which modify the clay chemistry and inhibit swelling, or non-aqueous polymer materials, appears to have considerable promise.

6 CONCLUSIONS

It has been shown that the use of a bentonite slurry or similar lubricating fluid may be very effective in reducing the resistance to jacking of a pipe line.

In unstable ground conditions, the primary purpose of the fluid is to support the excavated tunnel bore and prevent collapse of the ground on to the pipes.

Secondly, when the bore is stable, either due to the natural strength of the ground or to support from the fluid, the pipes will become partially or completely buoyant within the fluid; this in turn will greatly reduce the contact forces and hence frictional resistance between pipes and ground.

Finally, in appropriate ground conditions, the lubricant may reduce the friction coefficient (interface friction angle) between the pipes and the ground.

In unstable ground, particularly coarse granular soils below the water table, it is usually best to start full ground support and lubrication as soon as possible after the start of the drive, since it may not subsequently be possible to reduce the large jacking resistance developed where the ground has already collapsed. However if the tunnel bore is stable, or the drive is through very soft clay, introduction of lubrication may be delayed until the limiting jacking force is approached, since it will be effective over the full length of pipe line once brought into use.

The uses and effects of different types of lubricant for pipe jacks in high-plasticity clays need further investigation.

7 ACKNOWLEDGEMENTS

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Site control of pipe jack alignments

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ABSTRACT The final alignment of a pipe jacked tunnel may need to be kept within specified limits for a number of reasons. It has been normal practice to specify alignments in terms of maximum allowable deviations from line and level. However joint integrity and pipe damage are related to joint deviation angles, which are not directly related to absolute values of errors in line and level, but to their rate of change. Field measurements and theoretical models have demonstrated the need to control maximum joint deviation angles if serious joint damage is not to occur under high jacking forces. A simple graphical method is presented which allows joint deviation angles to be determined easily from line and level measurements, and rational decisions made about the steering adjustments needed to bring an errant pipe jack back on line without inducing excessive joint angles.

1 INTRODUCTION

In the specification of pipe jacks in the UK, tolerances have been generally set at $\pm 75\text{mm}$ for line (horizontal deviation) and $\pm 50\text{mm}$ for level (vertical deviation). These values have been accepted as a reasonable compromise between the requirement of the client for an accurately aligned pipeline and the need of the contractor to make steering corrections to cope with varying ground conditions etc. As far as the joints between pipes are concerned, BS5911 Part 120:1989 requires that they must be able to withstand an angular deviation of 1% for pipes of nominal size DN900 to 1200, and 0.5% for pipes of nominal size DN1350 to 1800, without loss of watertightness. However it can be shown theoretically, and has been confirmed by field measurements made as part of a research project at Oxford University, that different and in some cases more demanding restrictions are required to ensure that pipes do not fail during installation by overloading of the joints in compression. In particular, the angular deviations at pipe joints may have to be limited to values smaller than those required for watertightness, and rational specifications should cover this aspect. It then becomes important to be able to check and control these angular deviations during construction. This paper introduces a simple method of doing this which is suitable for use on site.

2 THE NEED FOR JOINT ANGLE CONTROL

In all pipe jacks the tunnelling machine or shield will

tend to stray off line, for a variety of reasons; small steering corrections will be made to maintain the right line and level, as a result small angular deviations between pipes will occur. These angular deviations have two major effects: they tend to increase the local contact stresses between pipes and the ground; and they cause serious stress concentrations at the joints between pipes. The former increases the overall jacking load, and the latter reduces the ability of the pipe joints to transmit the jacking force without damage to the pipes. A typical reduction in allowable load with joint angle β is shown in Figure 1, the reductions are even greater if plywood or solid timber is used as the packing material rather than chipboard or medium density fibreboard (MDF). These curves

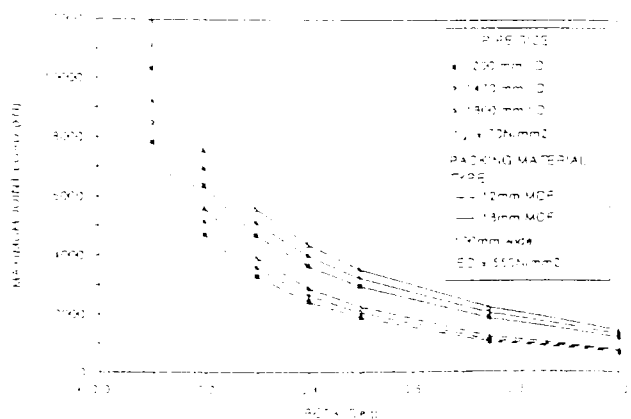


Figure 1 Permissible pipe end loading at various joint angular misalignments (β)

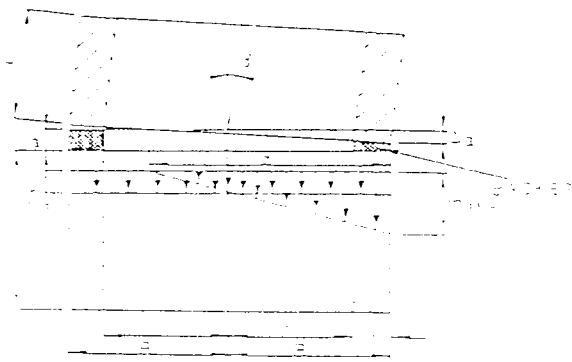


Figure 2 Linear stress analysis for pipe joint stresses

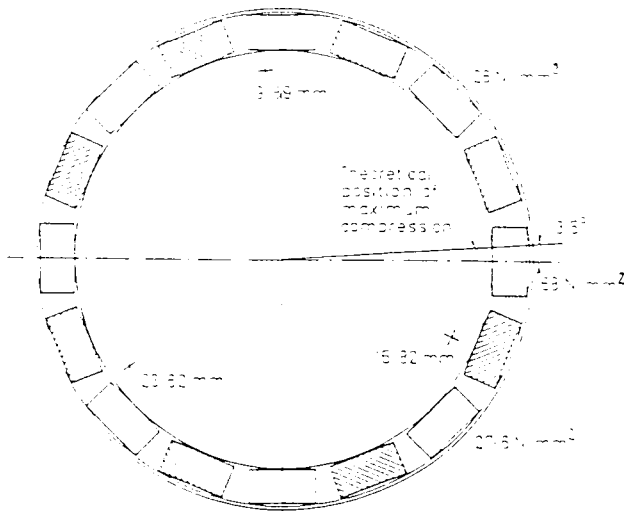


Figure 3 Compressive stresses measured by stress cells in pipe joint (1200mm I.D. pipe, $\beta = 0.47$)

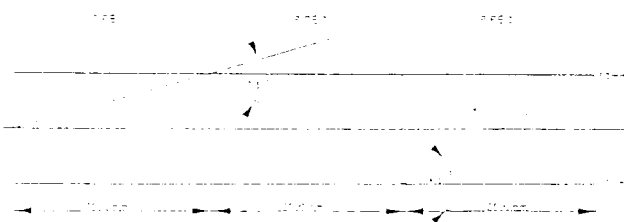


Figure 4 Pipes within line tolerance but with large angular deflections

have been calculated using the approach originally introduced by the Australian Concrete Pipe Association, using an approximate elastic unload-reload stiffness for the packing material (Figure 2). The validity of this calculation has been established by field measurements of joint stresses as part of a major research programme at Oxford University. Typically localised stresses at a joint angle of about 0.5° are shown in Figure 5.

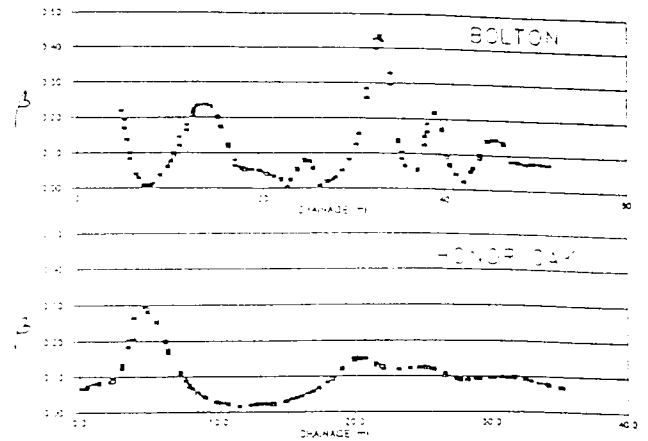


Figure 5 Actual joint misalignment angles on two instrumented pipe jacks

Traditional control of line and level is not sufficient as a means of controlling the angular deviations between successive pipes within acceptable limits of about 0.5° for transmission of large axial forces. An extreme example of pipes within specified tolerance for line but with large angular deflections is shown in Figure 4. In practice, the joint angles in well-controlled pipe jacks are typically in the range 0 to 0.5° ; for example, two sets of measurements from the field work are given in Figure 5. However, in difficult ground conditions, or if site control is poor, angles of 1.0° or greater may easily occur, often leading to pipe damage.

Measurements of the misalignment angles were made, along with tunnel surveys at a number of stages of each drive, on a total of nine projects with different ground conditions, cover depths and locations of the instrumented pipe within the pipe string. Two important observations in all cases were that:-

- the alignment of the pipe line did not change significantly as the pipe line was extended; thus local curvatures, once established by the steering of the tunnelling machine or shield, remain throughout the drive (Figure 6)
- any tendency of the pipe line to straighten under load was small (Figure 7)

These observations mean that conventional line and level measurements at the shield, usually in relation to a laser beam, are sufficient for practical purposes to determine joint angles at a particular location, with no significant change in alignment occurring due to the application of jacking loads or the passage of successive pipes. As the pipes are jacked forward, each joint between successive pipes will be subjected to the same angular distortion as it passes a particular point, but the joint stresses will become progressively more onerous as the jacking force increased. Control is often poorest at the very start of a drive, and it is at this location that the highest jacking forces will occur as the drive progresses.

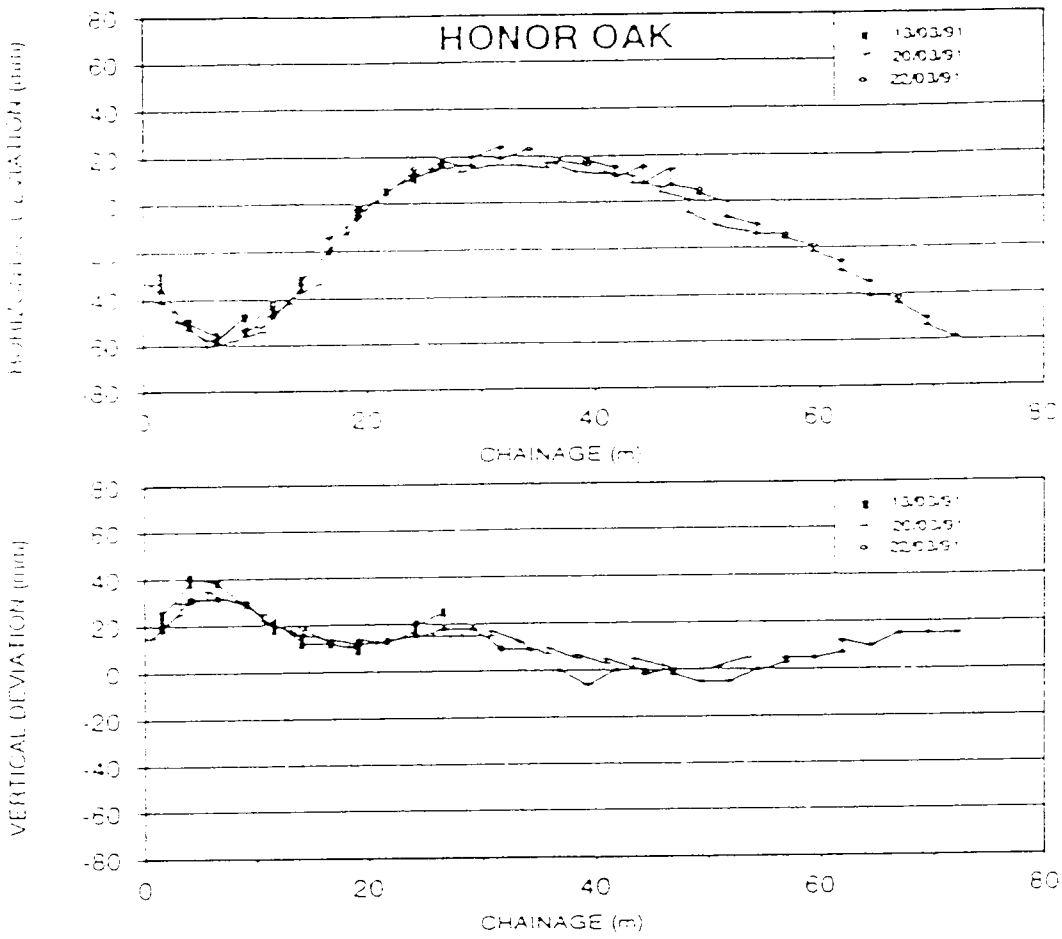


Figure 6 Tunnel alignment surveys at three dates

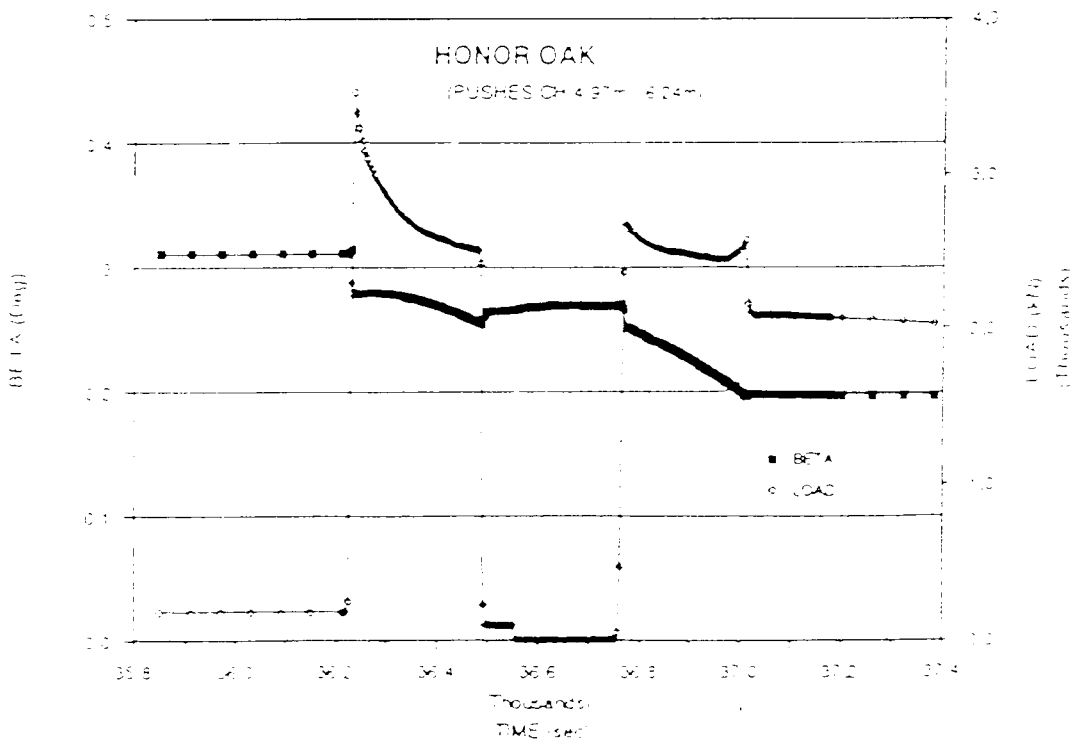


Figure 7 Variation in joint angle with load

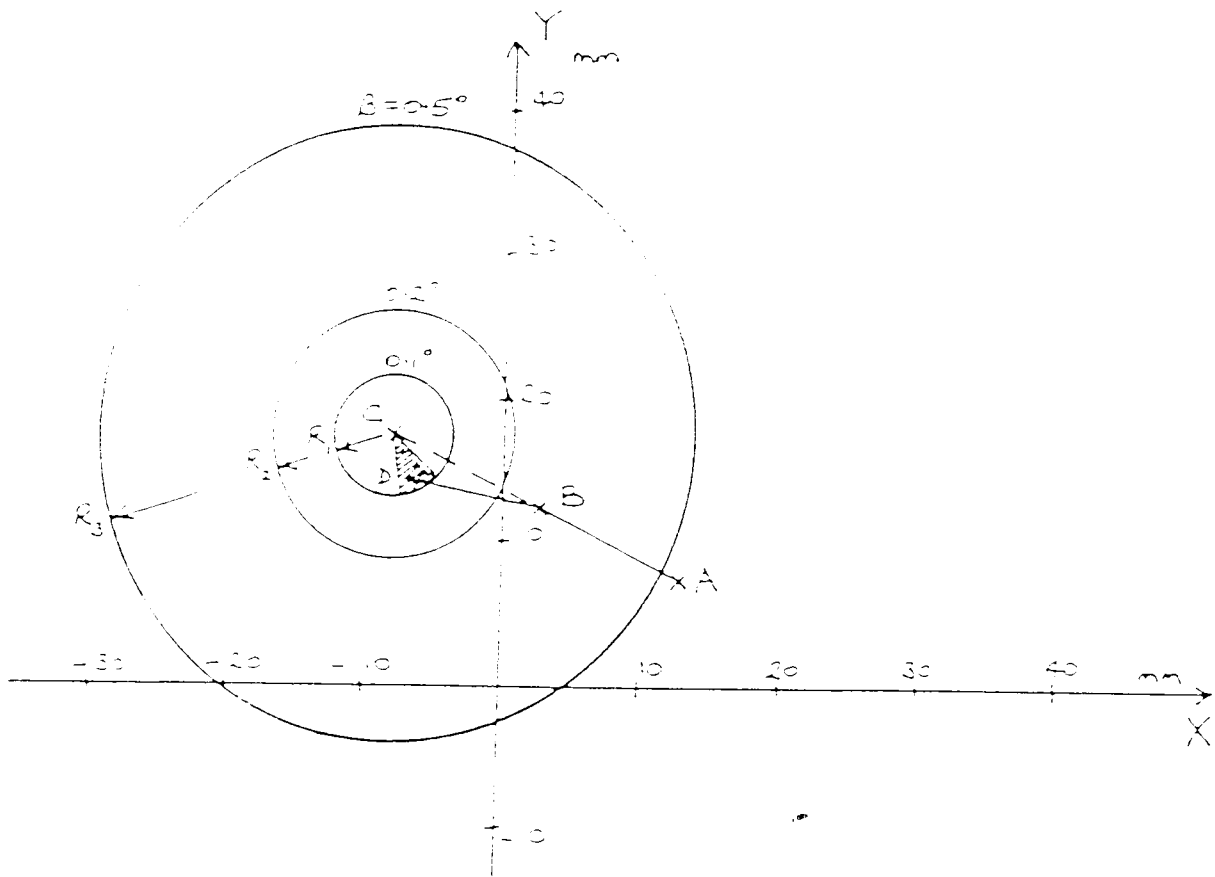


Figure 8 Control diagram for pipe jack alignment

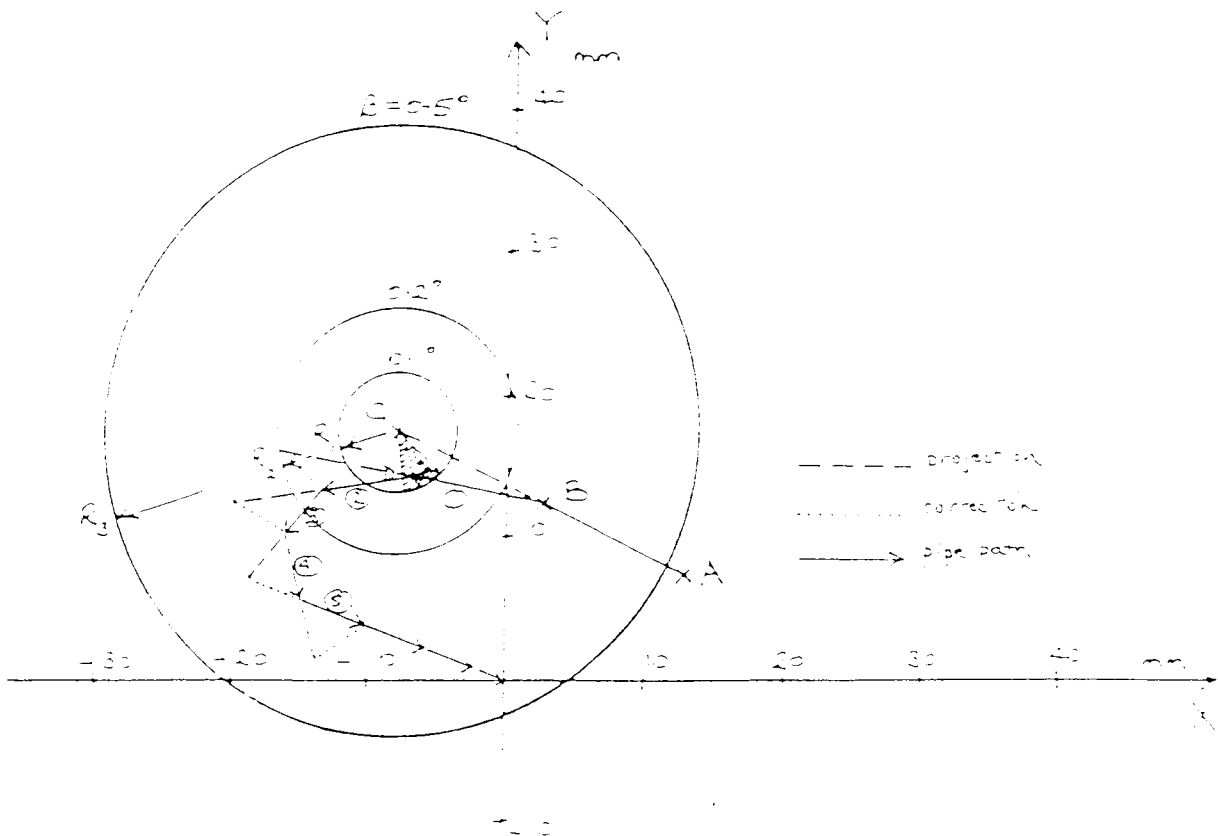


Figure 9 Successive steering corrections to pipe jack

Having demonstrated the importance of joint angles to the successful completion of a pipe jack without damage to the pipe, and the fact that deviations in line and level at the shield are sufficient to establish these joint angles for the duration of the drive, a simple graphical method will now be described which allows the joint angles to be determined from the line and level measurement. This then allows the angular misalignments that have occurred to be assessed on site, and decisions to be made about maximum allowable jacking loads or measures to reduce loads by the use of lubrication or insertion of interjacks. It also allows sensible decisions to be made about future steering corrections to keep angular deviations within acceptable limits.

3 CONTROL PROCEDURE

The control procedure is based on the use of a control diagram (Figure 8) on which successive line and level errors at the shield or lead pipe are plotted. For example, consider the lead pipe to have line and level errors X_1, Y_1 , plotted as point A on the diagram. The pipe line is now advanced by one pipe length, say 2.5 m, after which the line and level errors are X_2, Y_2 , point B on the diagram. If the tunnel is then to be advanced a further pipe length, pipe No. 2 will end up in the position previously occupied by pipe No. 1, with the offsets of its two ends given by points A and B. If pipe No. 1 were to continue along the same alignment, with no deviation at the joint with pipe No. 2, the new line and level offsets at its front end would be given by point C with co-ordinates X_3, Y_3 , such that $(X_3 - X_2) = (X_2 - X_1)$ and $(Y_3 - Y_2) = (Y_2 - Y_1)$. However if there is an angular deviation of the joint of $\beta = 0.1^\circ$ in any direction, the front end of pipe No. 1 could end up anywhere on the circle with centre C and radius R_1 . For $\beta = 0.2^\circ$ the corresponding circle would have radius R_2 , and for $\beta = 0.5^\circ$ radius R_3 . In each case the radius of the circle would be given by $R = L\beta$, where β is in radians and L is the pipe length; for $L = 2500$ mm and converting β to degrees, $R = 43.6\beta$ mm.

The controlling engineer, lead miner or computer (in an automated system) may then make a rational decision as to how line and level should be corrected over the next pipe length. The aim would be to head towards zero error ($X = 0, Y = 0$), but without exceeding an angular joint deviation of, say, 0.1° . In practice the actual deviations achieved might turn out to be somewhat greater, but still acceptable. The offset at the end of this advance of one pipe-length, point D, should be within an area such as shown hatched, within the circle with radius R . This would require the front end of the pipe to go from offset B to offset D, the necessary adjustments to the steering jacks being indicated by the change in direction from line BA to line BD. In the example shown this would

indicate an adjustment to the vertical alignment but very little to the horizontal alignment. Note that angular misalignments are controlled by rate of change of offset, not by absolute values. For the next pipe length the process is repeated, starting from what are now the last two data points, B and D. On site it would probably be most practical to have the control diagram at a large scale on a plastic sheet, so that points could be marked with a felt tip pen and easily removed once no longer needed, with the control circles at the same scale on a transparent overlay sheet.

The method easily allows flexibility of decision-making, for instance corrections can be deliberately biased towards controlling level at the expense of line should horizontal alignment be more important, for instance for hydraulic performance. It is also possible to plan "moves" ahead if it is necessary to get an off-line drive back accurately on line by a particular chainage, such as at an existing shaft, while ensuring that angular misalignments are acceptable. Figure 9 shows how the situation of Figure 8 could be corrected over a series of pipe lengths. The diagram can also be used to assess the angular deviations that have already occurred by plotting three successive offset positions at 2.5 m intervals of progress. In this case points A, B, and D in the diagram would be the actual measured offsets and the angle would be determined by projecting AB to C and finding the radial offset CD.

However possibly the greatest benefit on site is psychological, in emphasising to all concerned the importance of keeping angular deviations small, and providing a simple method of observing the actual deviations occurring as the tunnel progresses. Also, once clients are aware of the possibility of controlling joint angles, it makes sense for them to include in their specifications clauses to limit joint angles or, better, relate allowable jacking forces to joint angles.

4 CONCLUSIONS

The control of angular deviations between pipes during construction of tunnels by pipe jacking is important, in particular because the axial jacking force which can be transmitted through the joints without damage to the pipes is closely related to the joint deviation angle. The pipe line curvatures and hence local joint angles are essentially established by the path followed by the tunnelling machine or shield. Traditional tolerances specified for pipe line and level are not sufficient to control joint angles within acceptable limits.

A simple graphical method for use on site to determine joint angles from conventional line and level/offset measurements has been presented, this also allows decisions to be made about future steering

corrections on a rational basis while limiting joint angles to acceptable values.

Specifications for tolerances for pipe jack construction may then logically be written to address directly the requirements of the pipeline during construction and in use. Limits on line and level offsets may be relaxed to meet real restrictions, for instance in relation to other services in the ground, for hydraulic performance, or to ensure adequate clearance for later pipe or duct inserts. Construction tolerances, to make sure that drives are completed successfully without damage to the pipes, should concentrate on joint misalignment angles in relation to jacking force, pipe strength, packing material performance etc.

5 ACKNOWLEDGEMENTS

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The influence of lubrication on jacking loads from six monitored pipe jacks

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1. INTRODUCTION

Research related to pipe jacking and microtunnelling has been in progress at Oxford University for more than ten years. Several projects have been undertaken, including small and large scale laboratory tests, finite element modelling, and monitoring of full-scale projects during construction. The monitoring involved the incorporation into the pipe string of a heavily instrumented but otherwise standard jacking pipe (Figure 1), along with further instrumentation in the jacking pit. The instrumentation allowed measurement of overall jacking load, longitudinal strains in the pipes, articulation and compression of the pipe joints, pressure distributions in the pipe joints, and normal and shear stresses and pore pressures at the surface of the pipe due to contact with the ground. In many cases, measurements were also made of ground movements around the tunnel (Milligan and Marshall 1995), and all measurements were related to a detailed log of site activities and periodic surveys of tunnel line and level. Full details of the instrumentation are given by Norris (1992) and Marshall (1998).

A total of nine schemes were monitored. This paper is concerned with six of the schemes, for which lubrication of the pipe string was used for all or part of the drive; details of the projects are given in Table 1. The influence of the lubrication on the overall jacking resistance is considered first, in relation to the ground conditions. These included stiff clays, very soft clay, fine sand and gravelly sand; in four cases the pipe lines were below the water table. Measurements made at the pipe-soil interface are then used to illustrate some details of the mechanics of interaction between the pipes and the ground, and the ways in which these were affected by lubrication.

2. RECORDS OF JACKING FORCE

2.1 *Jacking force records*

Site records showing the total jacking forces measured at the drive shaft for the six schemes are presented in Figures 2 to 7. The jacking load is a combination of the penetration resistance of the shield, and the frictional resistance to forward movement of the pipes. The penetration resistance or face resistance depends very much on the method of excavation at the tunnel face. It is not generally affected by lubrication, and will not be considered further in this paper.

Frictional resistance shows up as the increase in total jacking force as the pipe string grows longer. It is influenced by many factors related to ground conditions and construction methods, and in the past has been estimated on the basis of experience, from which a range of apparent frictional stresses acting on the external surface area of the pipe have been derived (for example, see Craig (1983)). The word 'apparent' is used here because in most cases the pipes are only actually in contact with the ground over a limited portion of their surface area. Recent research has shown that in many cases the frictional resistance can be calculated more reliably on the basis of simple models of soil-pipe interaction (see Milligan and Norris (1998)).

Scheme	4	5	6	7	8	9
Location	Chorley Lancashire	Cheltenham Gloucester- shire	Leyton East London	Southport Lancashire	Seaham County Durham	Thurrock Essex
Date	July 1991	Dec. 1991	Nov. 1993	Feb. 1994	July 1994	Jan. 1995
Water Company	North West	Severn Trent	Thames	North West	North- umbrian	Anglian
Contractor	Barhale	Lilley	Clancy	Miller	Donelon	AMEC
Pipe supplier	Spun Concrete	Spun Concrete	ARC	Spun Concrete	Buchan	Buchan
Pipe ID ¹ (mm)	1500	1200	1500	1000	1800	1500
Ground type	Dense silty sand	Sand and gravel	London Clay	Dense silty sand	Stiff clay till	Very soft clay/peat
Below GWL ²	No	Yes	Yes	Yes	No	Yes
Cover (m)	7 - 10	4 - 7	6 - 8	5 - 8	6	5.5 - 6
Length (m)	158	384	75	160	310	260
Test pipe	No. 16	@ 85 m	No. 11	No. 2	No. 74	No.19
Excavation method	Hand	Slurry TBM	Hand	Slurry TBM	Open TBM	Slurry TBM
Lubrication	No/yes	Yes	Yes	Yes	No/yes	Yes
Settlement measured	No	Yes	Yes	Yes	Yes	Yes

Notes: 1. Internal diameter 2. Ground water level

TABLE 1. Details of schemes monitored

2.2 *Partial lubrication*

Figure 2 shows the jacking record for scheme 4, where the ground consisted of dense silty fine sand and sandy silt. The pipe level was above the ground water table, and there was sufficient capillary suction in the soil for the tunnel excavation to remain stable. As is usually the case, the tunnel was excavated to a slightly larger diameter than the external diameter of the pipes, so that the pipes were sliding along the invert of the tunnel and making only limited contact at the sides and crown. The frictional resistance was then calculated simply by multiplying the weight of the pipes by the coefficient of friction between pipes and ground as measured by the contact stress cells. Towards the end of the drive, bentonite slurry with polymer additives was introduced: the lubrication was only partial, in that the slurry did not completely fill the gap between pipe and ground, but produced a significant reduction in frictional resistance.

Partial lubrication was also used in scheme 6, but throughout the drive, so no comparison

was obtained between lubricated and unlubricated behaviour. The tunnel bore was again stable, in stiff London Clay, and the sliding resistance to jacking was low as the pipes moved forward on a layer of clay and lubricant in the invert of the tunnel. However whenever the pipeline was stationary the jacking force needed to restart sliding was considerably higher, giving the very peaky jacking record in Figure 3. This behaviour is typical for drives through stiff clays, with the increases much more marked in high plasticity than in low-plasticity clays.

2.3 *Full lubrication, cohesionless soils*

Schemes 5 and 7 were both pipe jacks through granular soils below the water table. In such situations, the ground will collapse immediately onto the pipes and develop large frictional resistance. The contact stresses between pipe and ground may then be estimated using some variant of Terzaghi's 'silo' analysis (see Stein et al.(1989), Pellet and Kastner (1998) and Milligan and Norris (1998)), and frictional resistance obtained by multiplying these stresses by the interface friction coefficient. However, the tunnel bore can be stabilised by introducing a bentonite slurry to fill the 'overcut' between pipes and ground and pressurising this sufficiently to overcome the groundwater pressure, form a filter cake in the surrounding ground, and then provide support to the ground. The pipes will become buoyant in the fluid, and in most cases the uplift force will exceed the weight of the pipes; the pipes will lift and press against the crown of the tunnel bore, and the frictional force will be reduced to that obtained by multiplying the (negative) buoyant weight by the interface friction coefficient. In scheme 5 this resulted in a reduction in jacking resistance of an order of magnitude. Scheme 7 was more complex in that periods of full lubrication were interspersed with periods when the lubrication was less than perfect, due to problems with the slurry pump. However the differences between the fully lubricated and partially lubricated resistances were again very substantial (Figures 4 and 5).

2.4 *Full lubrication, cohesive soils*

Schemes 8 and 9 (Figures 6 and 7) were both pipe jacks through cohesive soils, but whereas the ground for scheme 8 was a stiff glacial till, that for scheme 9 was very soft organic clay and peat. In the former, the tunnel bore was stable, and the pipes initially slid along the base of the bore. However the jacking resistance was quite high, much greater than calculated from the self weight of the pipes and the interface friction coefficient. After introduction of full lubrication, the pipes became buoyant and contact with the ground was primarily at the crown of the pipe, as shown by the contact stress cells. The increases in jacking loads after short stoppages, evident in the London Clay, were not apparent in this low-plasticity clay. However relatively large jacking forces were needed to restart movement after each weekend. Pressure cell readings indicated that the pipes had settled back onto the base of the bore after each weekend, due presumably to dissipation of the lubricant into the more permeable sand layers present in the till. Restarting driving, accompanied by pumping of fresh lubricant, brought the jacking force down to the lubricated value.

In the soft clay, progress was almost continuous, with a slurry tunnelling machine and double-shift working. No lubrication was used initially, but driving resistance was quite low due to the low strength of the ground. After a weekend break, the jacking force needed to restart the pipeline had risen substantially, and the contractor opted to use lubrication thereafter. The jacking resistance gradually dropped to a lower value.

2.5 *Jacking resistance*

The measured jacking resistances are summarised in Table 2: substantial reductions in

jacking resistance were obtained in all ground conditions, but most particularly in unstable coarse-grained soils. The lubricated values are also very much smaller than the typical values for different soil conditions given by Craig (1983), which are also included in the table. One important factor evident from the jacking records is the extent to which jacking resistance can be reduced for pipe lengths which have already been installed without lubrication. When the tunnel bore is self-stable, and the overcut remains open, lubrication can be introduced at any stage and will be effective over the full length of the pipe line. This was the case in scheme 8. When the ground is unstable, and has been allowed to collapse onto the pipes, effective lubrication cannot subsequently be expected to operate over these lengths. Only in 'new' pipe lengths installed with pressurised slurry to stabilise the ground first and then act as a lubricant is the benefit of lubrication obtained. This can be observed in the records for schemes 5 and 7. Interestingly, in the soft clay of scheme 9 it proved possible to establish lubricated behaviour over the full pipe length even though the ground was in contact with the outside of the pipe before introduction of the lubrication.

No.	Pipe diam. (mm)	Soil type	Cover (m)	Lubrication	Measured frictional resistance		Craig (1983) (kPa)	Reduction due to lubrication (%)
					(kN/m)	(kPa)		
4	1200	Dense silty sand	7 - 10	Unlubricated	23.1	4.2	5-20	59
				Partial	9.4	1.7		
5	1200	Sand and gravel *	4 - 7	Unlubricated	100	22.0	10-15	90
				Lubricated	10	2.2		
6	1500	London Clay	6 - 8	Partial	8.5 min.	1.2	5-20	
7	1000	Dense silty sand *	5 - 8	Imperfect	41.6	11.0	10-15	94
				Full	2.6	0.7		
8	1800	Stiff clay till	6	Unlubricated	48	7.1	5-18	69
				Lubricated	15	2.2		
9	1500	Very soft clay and peat	5.5 - 6	Unlubricated	25	4.4		44
				Lubricated	14	2.4		

* Below water table

TABLE 2 Jacking frictional resistance for lubricated and unlubricated conditions

3. LOCAL INTERFACE BEHAVIOUR

3.1 Contact stress cells

The local interaction between pipes and ground, which gave rise to the overall jacking forces discussed above, could be clarified from the readings from the contact stress cells built into the side walls of the instrumented pipe. These contained 'Cambridge-type' transducers which allowed the independent measurement of both the radial (normal) contact stress on the cell and the shear stress due to forward motion of the pipe. Pore pressures were also measured, and subtracted from the total stresses to give effective stresses. In schemes 4 and 5 the pore pressure instrument was separate from the stress cell, although close to it. This was found to complicate interpretation of rapidly-fluctuating

measurements, so for schemes 6 to 9 the pore pressure transducers were incorporated into the face of the stress cell

3.2 Buoyancy

Pipes sliding in a stable bore only make contact at the tunnel invert; a typical set of readings of shear stress, from scheme 6, are shown in Figure 8. The only contact with the side of the pipe occurred on the left hand side when the pipe string was negotiating a change in line due to a steering correction to the left at a chainage of about 20m. Occasional contact at the crown may be a result of irregular hand excavation above the shield. On the other hand, in scheme 8 the pipes were generally buoyant due to the use of complete lubrication filling the tunnel bore. Contact shear stress measurements are shown in Figure 9. In Figure 10, a complete set of stress readings is given for one cell location at the base of the pipe; the total normal stresses are essentially the same as the pressures measured by the pore pressure transducer, giving confidence that both are operating correctly in measuring the slurry pressure in the overcut. The effective normal stresses and shear stresses are mostly quite small, indicating negligible contact between pipe and ground at the tunnel invert.

3.3 Pipe-soil interface behaviour

An enormous amount of data has been collected on interface behaviour from the contact stress cells. Each forward push of the pipes produces in effect a large-displacement interface shear test at each cell in contact with the ground. There is only space in this paper to present summary plots from typical situations with cohesionless soils.

The first of these, shown in Figure 11, relates to the partial lubrication of scheme 4. The pipes were sliding along the base of a stable bore and substantial contact was made only at the base of the pipes. Line A on the figure is from measurements made under conditions of no lubrication, with quite high maximum contact stresses; the behaviour is clearly frictional in nature, with an interface friction angle between the concrete pipe and the dense silty sand of 39° . Line C relates to conditions immediately after injection of lubricant, in this case bentonite slurry with a polymer additive. The overcut was not completely filled, but the pipe has become partially buoyant, with reduced contact stresses. In this case the interface friction angle also seems to have been greatly reduced, to a value of about 15° . Line B relates to intermediate conditions after some dissipation of the lubricant, with contact stresses and interface friction angle both increasing towards the unlubricated values.

Figure 12 shows similar data from scheme 7: this was also through fine sand, but below the water table. When the lubrication system was fully functioning, sufficient slurry pressure was applied to counteract the hydrostatic pressure, form a filter cake and support the ground. The pipes were buoyant and pressing against the crown of the tunnel. However because the injection ports were all along one side of the pipe line, the pipes tended also to be forced against the left hand side of the tunnel. The effective contact stresses here were quite high, and gave a range of interface friction angles of about $37.5 \pm 3^\circ$. At the crown of the pipe the contact stresses were rather lower, and the interface friction angles also slightly lower; this may reflect some loosening of the soil at the crown of the tunnel.

At some points the contact between pipe and ground at the left side was lost and slurry was able to flow into the gap, with an immediate and dramatic reduction in normal and shear stresses to practically zero (Figure 13). At the same time the recorded pore pressure rose from the hydrostatic pressure of about 40 kPa to the slurry pumping pressure of 75 kPa.

Results from cohesive soils have proved much harder to interpret. At times the interface

behaviour may be represented by 'undrained' behaviour, with almost constant adhesion values equal to the remoulded shear strength of the soil. At other times the behaviour surprisingly appears to be frictional even in terms of total stress; possible explanations for this are explored by Milligan and Norris (1998).

4. CONCLUSIONS

Lubrication has been shown to affect the jacking resistance of pipe jacks and microtunnels in a number of ways. In unstable cohesionless soils the most important use of slurry is to support the ground and prevent it from collapsing on to the pipes. The next most important effect is to reduce the apparent weight of the pipes by buoyancy in the lubricant, and thereby reduce the effective contact stresses between pipe and ground. Pipe jacking pipes usually become fully buoyant and press against the crown of the tunnel. Microtunnelling pipes with smaller diameter and relatively thicker walls may not become fully buoyant, but may approach a 'weightless' condition. Finally the lubricant may reduce the pipe-soil interface friction at the points of contact, and form a layer of low shear resistance between pipe and ground elsewhere around the pipe. The overall jacking resistances may be substantially reduced in all ground conditions by the use of lubrication, but most particularly in unstable cohesionless ground.

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The financial and technical support of the UK Pipe Jacking Association, Thames, Severn Trent and North West Water Companies, and of the Engineering and Physical Sciences Research Council are gratefully acknowledged, as are the co-operation and support of contractors and pipe suppliers involved with the work on site. The fieldwork was undertaken by the second author and by Dr Paul Norris, now of Mott MacDonald, with the assistance of technicians of the Civil Engineering Research Group at Oxford University

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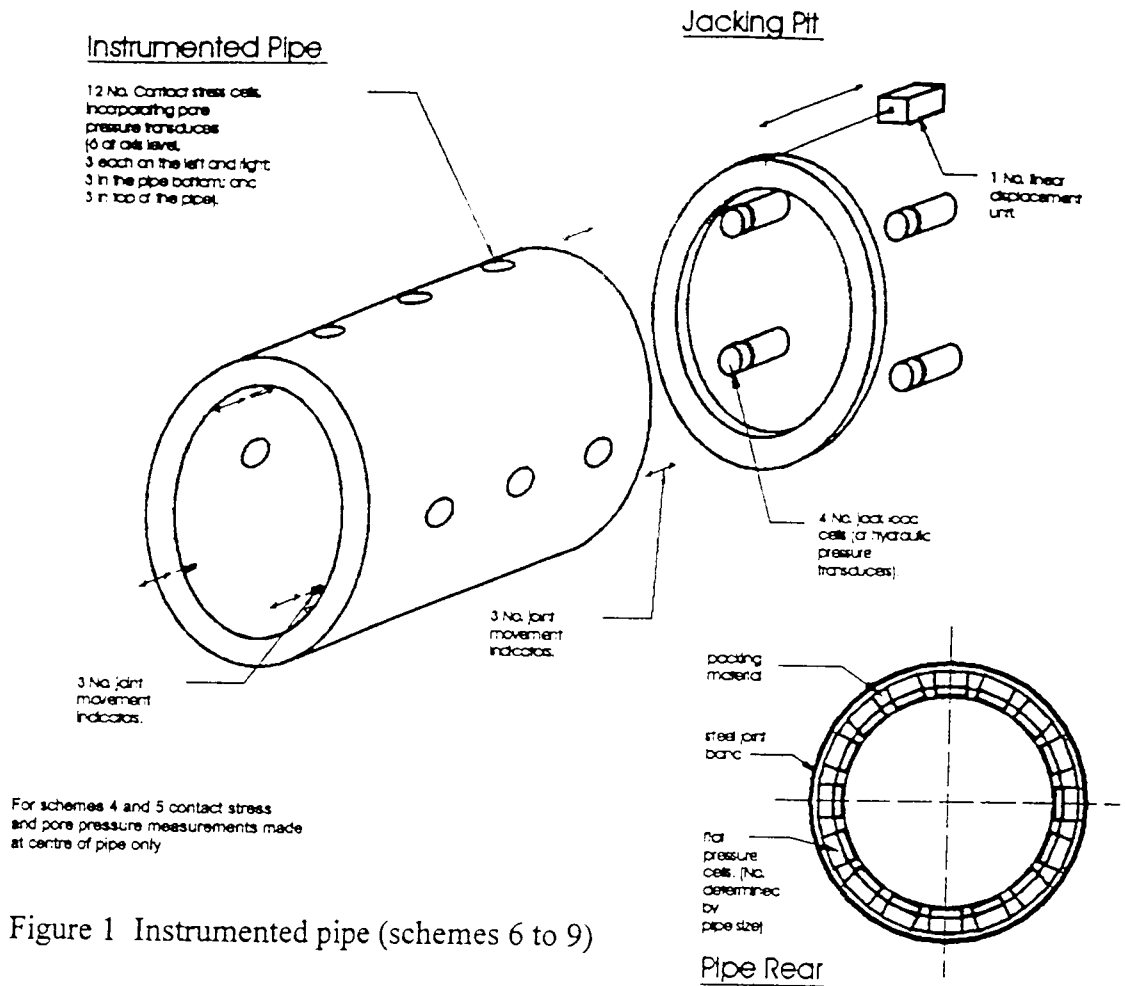


Figure 1 Instrumented pipe (schemes 6 to 9)

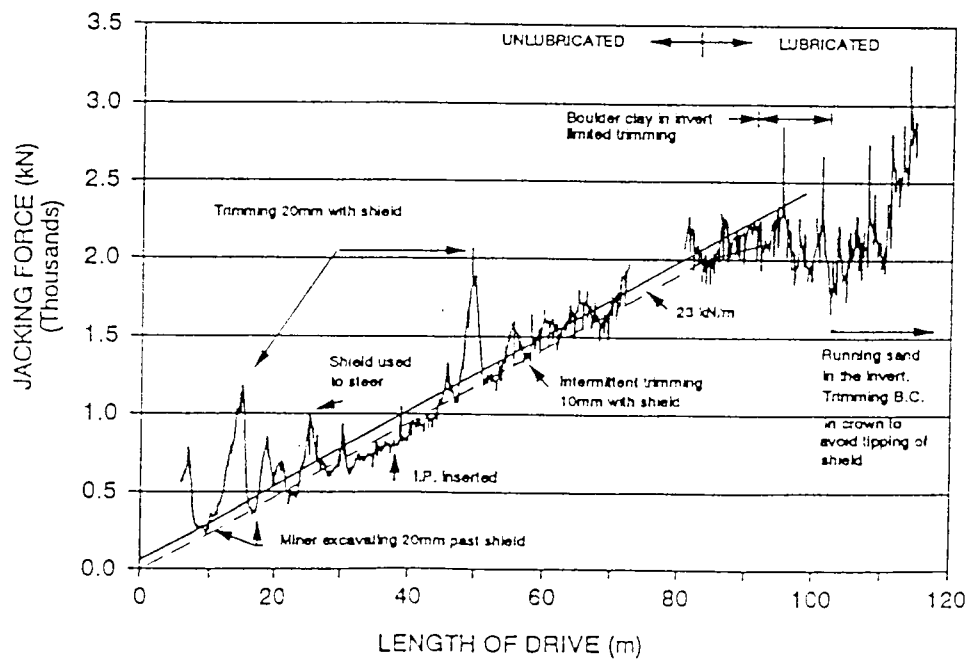


Figure 2 Jacking force record for scheme 4

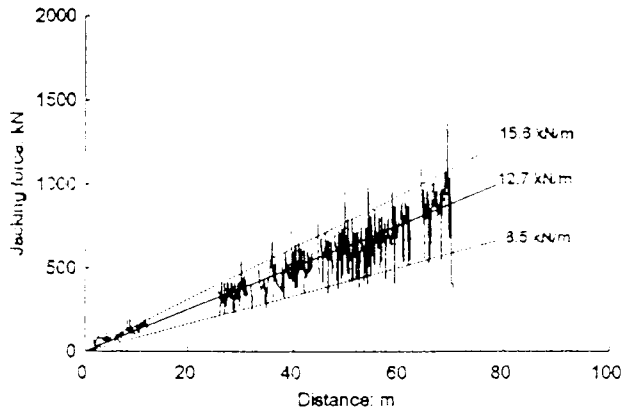


Figure 3 Jacking force record for scheme 6

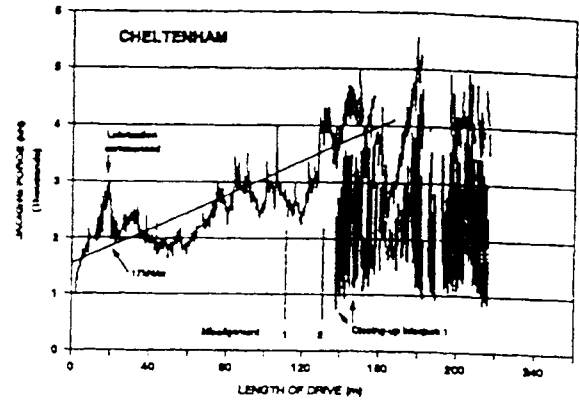


Figure 4 Jacking force record for scheme 5

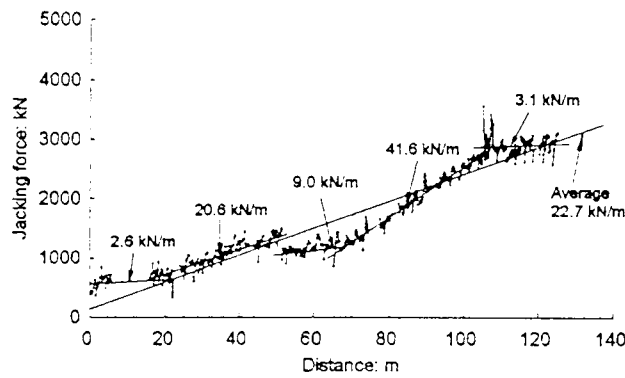


Figure 5 Jacking force record for scheme 7

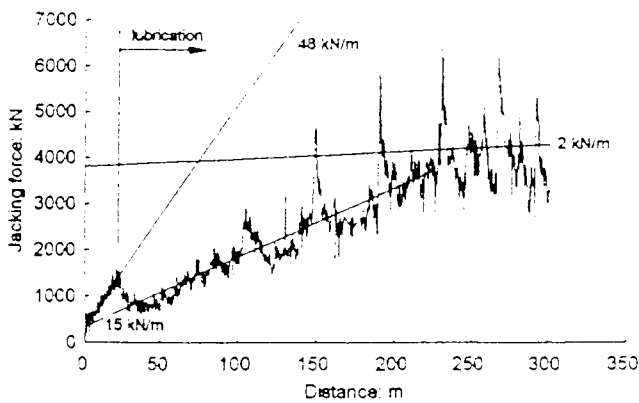


Figure 6 Jacking force record for scheme 8

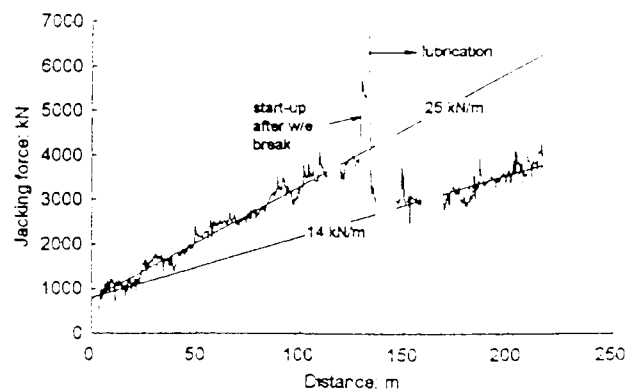


Figure 7 Jacking force record for scheme 9

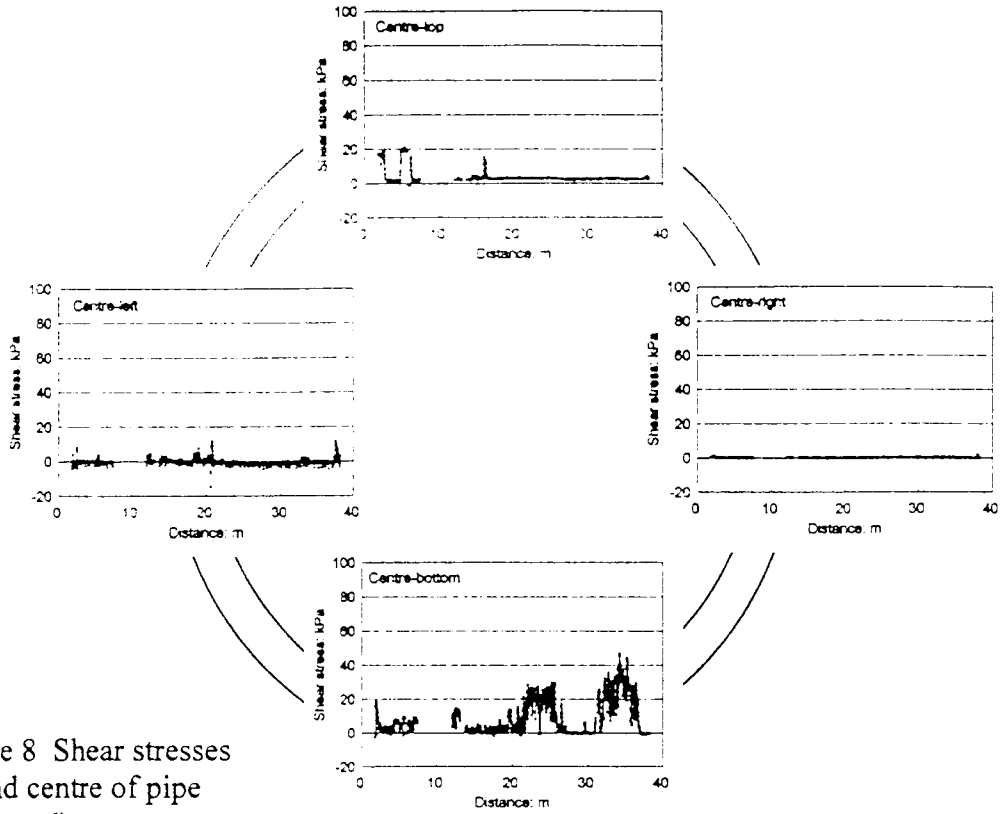


Figure 8 Shear stresses around centre of pipe (scheme 6)

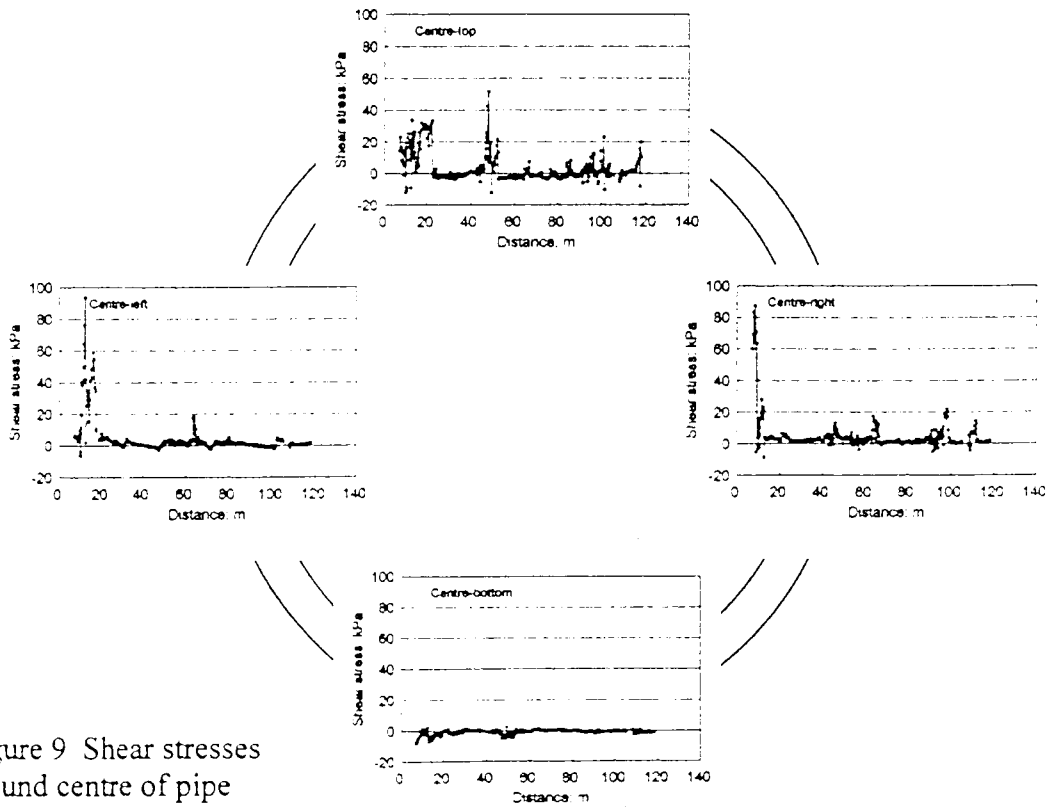


Figure 9 Shear stresses around centre of pipe (scheme 8)

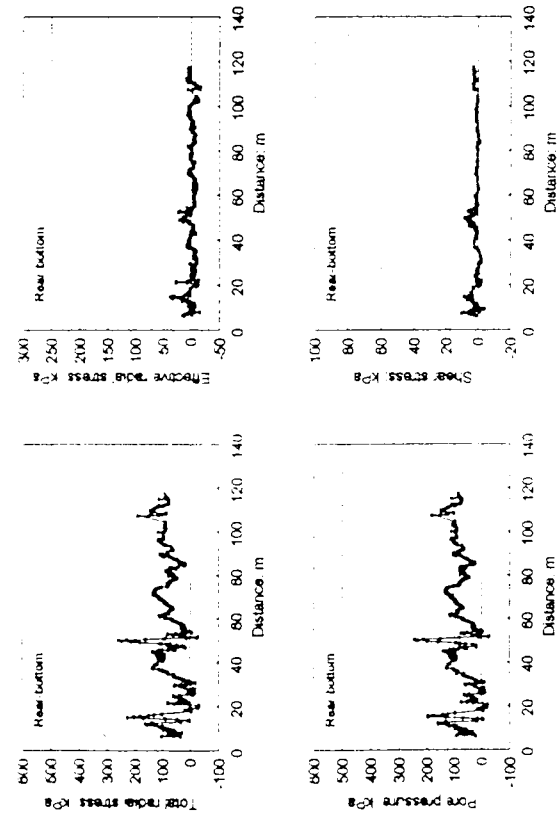


Figure 10 Stress and pore pressure measurements at rear-bottom cell (scheme 8) – note different vertical scales

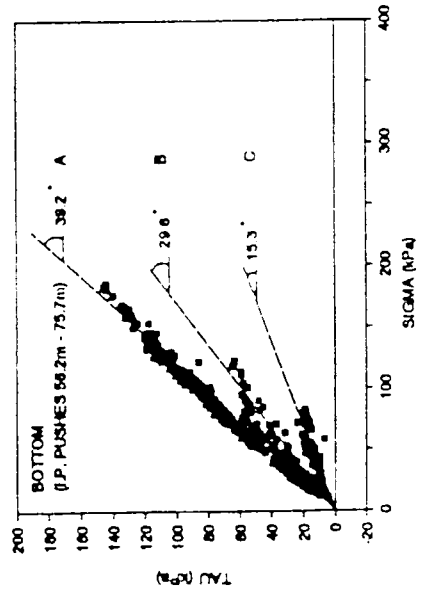


Figure 11 Interface friction angles (scheme 4)

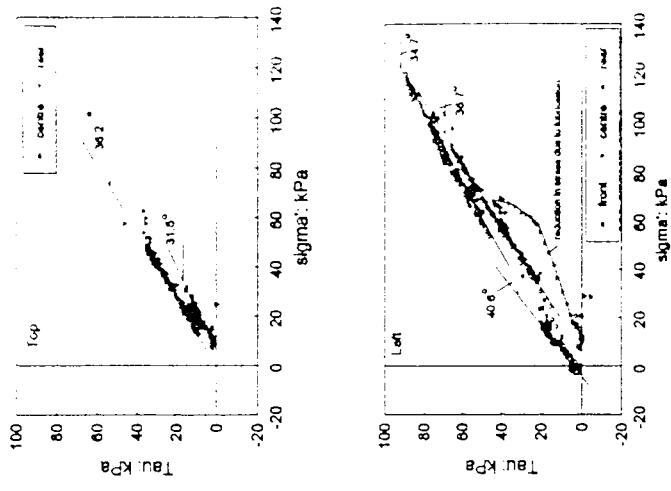


Figure 12 Interface friction angles (scheme 7)

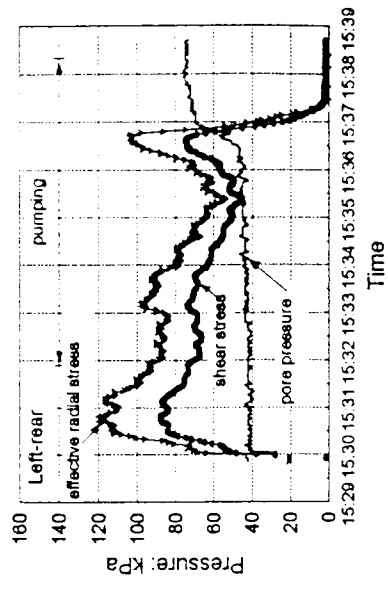


Figure 13 Local contact stress measurements during pumping of lubricant (scheme 7)

Pipe-soil interaction during pipe jacking

G. W. E. Milligan, MA, MEng, PhD, CEng, FICE and P. Norris, BEng, MEng, DPhil, CEng, MICE

■ The purpose of this paper is to introduce some simple theoretical models of pipe-soil interaction during pipe jacking, and relate these to observations made in the field. Ground conditions, construction techniques and the degree of site control all influence the resistance to jacking of pipes, but if an appropriate model is chosen it should be possible to predict jacking forces with a reasonable degree of accuracy. Deviations of the pipeline from a straight line increase the jacking resistance. A new analysis, based on observations from the field monitoring, provides an explanation for the measured increases in pipeline resistance for pipes jacked through a stable bore; it highlights the important factors and emphasizes the need for careful control of pipeline alignment. Explanations are also sought for the apparently frictional behaviour in terms of total stress at the pipe-soil interface in firm and stiff cohesive soils. Time effects are shown to be important in high plasticity clays.

Keywords: geotechnical engineering; pipes & pipelines; tunnels & tunnelling

Notation

B	half-width of 'trench' in Terzaghi analysis of soil loading on pipe
b	width of contact area between pipe and clay
C	constant of integration
D	internal diameter of pipe
D_c	internal diameter of cavity (tunnel excavation)
D_e	external diameter of pipe
E_s, E_p	Young's moduli for soil and pipe material
F	frictional resistance
H	depth from ground surface to crown of pipe
H_1	depth from ground surface to water table
K	coefficient of earth pressure
L	contact length between pipe and clay
l	half-wavelength of curvature in pipeline
N	lateral contact force between pipe and ground
P_u	contact force per unit length between pipe and clay
P	jacking force
q	normal contact stress at pipe-clay interface; surcharge on ground surface
R	radius of fan zone in failure mechanism
s_u	undrained strength of clay
W	weight of pipe per unit length

z	depth below ground surface
α	angle of shear plane to the horizontal in failure mechanism; factor relating adhesion between pipe and ground to undrained strength of clay
β	angle of joint between successive pipes
γ, γ'	bulk and submerged unit weight of soil
γ_w	unit weight of water
δ	angle of friction at interface between pipe and soil
Δ	vertical displacement at pipe-soil contact in local failure mechanism
ν_s, ν_p	Poisson's ratio for soil and pipe material
σ'_v, σ'_h	effective vertical and horizontal stresses in soil
τ	shear stress in soil or at pipe-soil interface
ϕ'	internal angle of friction for soil

Introduction

During the past seven years a number of pipe jack schemes have been monitored during construction as part of a continuing programme of research carried out by Oxford University. The nature and extent of the research work has been discussed, and results from the first five field monitoring schemes summarized, by Milligan and Norris.^{1,2,3,4}

2. Pipe jacking is a method of providing a lined tunnel or pipeline by pushing an increasingly long 'string' of pipes through the ground from a thrust pit to a reception pit, with excavation by hand or machine within a shield at the front end. It can be used for a range of pipe sizes with internal diameters from 250 mm up to greater than 3 m, or even for box culvert sections up to sizes appropriate for transport tunnels. (Jacked pipelines of non-man-entry size, less than about 1000 mm i.d., are often referred to as microtunnels.) Success of the technique is heavily influenced by the maximum jacking force required, since if this becomes excessive it may cause structural failure of the pipes, particularly at the joints, overload the thrust wall which provides reaction for the jacks, or even exceed the capacity of the jacks.

3. The purpose of the research work was to investigate the behaviour of the pipes, in particular in relation to the performance of the joints between pipes, and the interaction between the pipes and the ground, during installation by pipe jacking. For this purpose a

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specially instrumented pipe was inserted as part of the string of pipes on nine actual construction sites. A schematic of the instrumentation is shown in Fig. 1, and some details of the various schemes in Table 1. Briefly, the pipe instrumentation allowed measurement of the

- (a) joint articulation, using joint movement indicators
- (b) pipe barrel longitudinal strains, using extensometers
- (c) joint stress distribution, using thin joint pressure cells
- (d) pore pressures and normal and shear stresses at the interface between pipe and ground, using contact stress transducers and pore pressure probes.

In addition, accurate measurements of jack loads and of pipeline advance were made at the jacking pit, using load cells and a displacement transducer. All measurements were correlated with tunnel alignment surveys, ground conditions and site activities.

Theoretical models

4. It is normal procedure for the tunnelling shield to have a slightly larger external diameter than that of the pipes following it. This 'overcut', typically 10–20 mm in diameter, reduces the interaction between the pipes and the ground. In fact, in reasonably stiff, stable soils the excavation will stand open temporarily and the pipes simply slide along the bottom of the bore (Fig. 2). The stability of the tunnel may be checked by conventional analyses,⁵ because of the

relatively small diameters, most pipe jacked tunnels in clay are stable during the construction phase, except those in very soft clay. Cohesionless soils which are dry or fully saturated will of course not be stable, but in damp fine or silty sand the capillary suctions may be sufficient to maintain temporary stability.

5. For pipes sliding in a stable bore, alternative models may be considered. The first assumes that the contact is purely frictional, and the resistance to sliding simply given by $W \tan \delta$, where W is the weight of the pipe and δ the angle of interface friction between pipe and soil. The second model assumes that the interaction is cohesive, with the resistance per unit contact area related to the undrained strength of clay soil. Haslem⁶ has suggested that the width b of the contact area may be based on the solution for elastic contact between two curved surfaces

$$b = 1.6 (P_u k_d C_e)^{1/2} \quad (1)$$

where P_u is the contact force per unit length, $k_d = D_c D_e / (D_c - D_e)$, $C_e = (1 - \nu_s^2) / E + (1 - \nu_p^2) / E_p$, and D_c is the internal diameter of the cavity, D_e is the external diameter of the pipe, E_s , E_p are the elastic moduli of the soil and pipe, and ν_s , ν_p are the Poisson's ratios for the soil and pipe.

6. When the bore is stable, the overcut may be partially or completely filled with a lubricant such as bentonite slurry, to reduce the resistance to jacking. An unstable bore may be stabilized by injecting bentonite under pressure into the overcut; in cohesionless soils the slurry

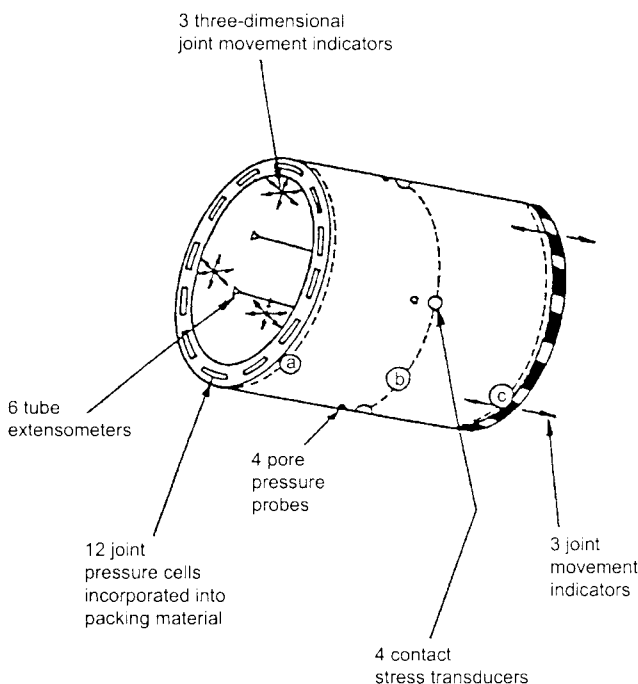


Fig. 1. Schematic of pipe instrumentation

Table 1. Details of schemes monitored

Scheme details	Scheme No.								
	1	2	3	4	5	6	7	8	9
Location	Bolton, Lancashire	Gateshead, Tyneside	Honor Oak, SE London	Chorley, Lancashire	Cheltenham, Gloucestershire	Leyton, E London	Southport, Lancashire	Seaham, Co. Durham	Thurrock, Essex
Date	August 1990	January 1991	March 1991	July 1991	December 1991	November 1993	February 1994	July 1994	January 1995
Client water company	North West	Northumbrian	Thames	North West	Sewern Trent	Thames	North West	Northumbrian	Anglian
Contractor	Laserboore	DCT	Barhale	Barhale	Lilley	Clancy	Miller	Donelon	AMEC
Pipe supplier	Buchan	ARC	Buchan	Spun Concrete	Spun Concrete	ARC	Spun Concrete	Buchan	Buchan
Pipe internal diameter	1200 mm	1350 mm	1800 mm	1500 mm	1200 mm	1500 mm	1000 mm	1800 mm	1500 mm
Ground type	Stiff glacial clay	Weathered mudstone	London clay	Dense silty sand	Sand and gravel	London clay	Dense silty sand	Stiff glacial clay	Very soft clay/peat
Cover: m	1.3—1.5	7—11	11—21	7—10	4—7	6—8	5—8	6	5.5—6
Length: m	60	110*	78	158	384	75	160	310	260
Test pipe	No. 3	No. 10	No. 15	No. 16	@ 85 m	No. 11	No. 2	No. 74	No. 19
Excavation method	Hand	Hand	Hand	Hand	Slurry TBM	Hand	Slurry TBM	Open face TBM	Slurry TBM
Lubrication	No	No	No	No/yes	Yes	Yes	Yes	No/yes	Yes
Packer material	MDF†	MDF	MDF	MDF	MDF	MDF	MDF	MDF	MDF
Settlements measured	No	No	No	No	Yes	Yes	Yes	Yes	Yes

Notes: * Monitoring for part of drive.
† MDF = medium density fibreboard.

must be correctly formulated to create a 'filter cake' layer around the cavity and then pressurized to the support pressure required for the soil. It may then also fulfil the requirements of a lubricant. When the cavity is filled with slurry, the pipes will become at least partially buoyant and their effective weight will become small or even negative. The two models described in § 5 may then be applied using the buoyant weight.

7. Unstable soils will be either very soft clays or cohesionless materials. The former will tend to close around the pipe and it might be expected that resistance to jacking would be given by the undrained strength (perhaps reduced by some adhesion factor α) acting over the full surface area of the pipe. Cohesionless soils will collapse onto the pipe, and Auld⁷ has suggested that the contact stresses between soil and pipe may be obtained from Terzaghi's⁸ analysis for soil settling in a trench (see Appendix 1). Multiplication of these stresses by the interface friction coefficient will then give the shear stresses resisting movement of the pipe.

8. A final condition is that sometimes referred to as 'squeezing ground'. This generally applies to heavily overconsolidated plastic clays, where the conditions for overall tunnel stability are met but high initial horizontal stresses cause elastic deformations and local plastic yielding around the tunnel sufficient to close the overcut and allow at least partial additional contact between clay and pipe at the sides and crown. The resulting contact stresses may be quite high. Increasing the size of the overcut to reduce the likelihood of this occurring may cause unacceptably large ground movements in some cases. Injection of slurry to try to reduce the resulting high jacking forces may exacerbate the problem by accelerating swelling of the clay with water from the slurry. This is a very complex problem which requires further research.

9. The simple models referred to in § 5 all assume that the pipe jack is straight. Pipes may be jacked along deliberately curved paths, while even a 'straightline' pipe jack is never in practice perfectly straight. Variations in ground conditions, imperfections in the tunnelling shield, variations in excavation technique, etc., may all cause the shield to stray from the intended line and level. Steering jacks in the shield or tunnelling machine are used to make corrections; with good control the pipeline alignment can readily be kept within typical specified tolerances in line of 75 mm and level 50 mm. The resulting 'wriggle' profile of the pipeline is typically a series of shallow oscillations about the true line; over-rapid correction of errors induces excessive angular misalignment between successive pipes and should be avoided. The effects of pipeline curvature on jacking forces are considered later in the paper.

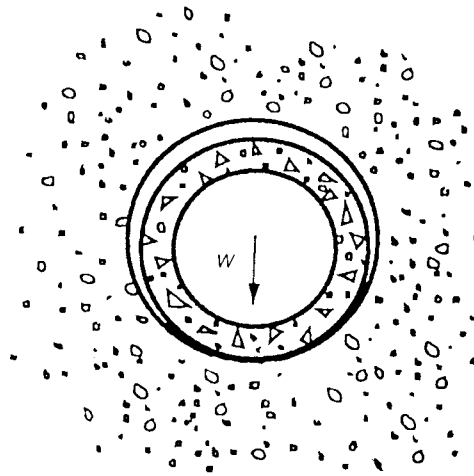


Fig. 2. Pipes sliding in a stable tunnel bore

Results from field monitoring

Pipeline alignments and joint stress distributions

10. Successive tunnel surveys made during the field monitoring work have shown that in competent ground the path followed by the shield establishes the line to be followed by all subsequent pipes, as shown in Fig. 3. There is no apparent tendency of the line to straighten with the passing of successive pipes. The joint gap measurements allow accurate determination of the three-dimensional joint angle β between successive pipes, which may also be calculated approximately from the line and level measurements; these are also plotted in Fig. 3. The maximum joint angles tend to occur at the peaks of the pipeline oscillations, with values of up to about 0.3° , or occasionally 0.5° . Where the control was not so good, joint angles of up to about 1.0° have been recorded. These angles will be experienced by all pipe joints passing the relevant point. The site measurements have also shown that there are only small changes in joint angles due to application of the longitudinal jacking forces.

11. Joint angles as small as 0.2° result in highly localized contact stresses between pipe ends, even with the use of compressible packing material in the pipe joints. A simple calculation⁹ may be used to give a reasonable prediction of these stresses, which have been confirmed by the measurements from the pipe joint pressure cells; for example, see Fig. 4. Under these conditions the 'centre of pressure' of the joint stresses will be quite close to the edge of the pipe. As the joint angles reduce, the joint pressures will become more uniform and the centre of pressure will approach the centreline of the pipe.

Pipe-soil interface stress measurements

12. The contact stress cells built into the side wall of the instrumented pipe have provided two main types of information. The magnitudes of the radial and shear stresses indicate the

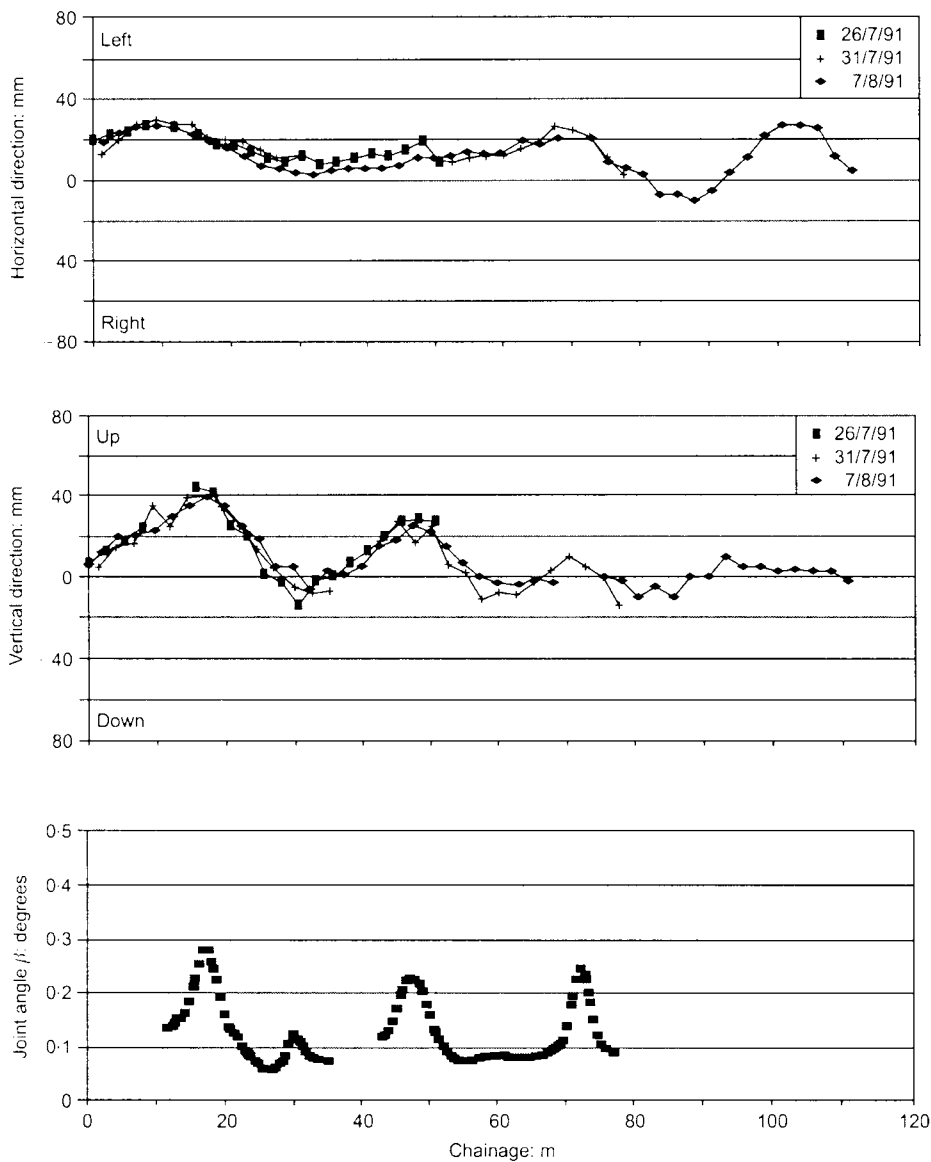


Fig. 3. Typical tunnel alignment surveys (scheme 4)

extent to which pipe and ground are in contact, while their ratio gives a direct measure of the interface friction between pipe and ground. The two combine to give the total frictional resistance to jacking.

13. Simple sliding in an open bore was apparent in a number of cases, notably in the stiff cohesive ground of schemes 1, 2, 6 and 8 (see Table 2) and in the dense silty sand above the water table in scheme 4. The pipe-soil contact stresses for scheme 4 are shown in Fig. 5; contact is primarily along the bottom of the pipe, with occasional contact at the sides and roof of the tunnel bore. Scheme 3 was a case of squeezing ground giving a high jacking resistance, scheme 9 was in very soft clay and peat and showed almost equal contact stresses all round the pipe, while schemes 5 and 7 were in

cohesionless soils below the water table. In both cases 5 and 7, high jacking resistance was recorded when the slurry support and lubrication was not effective – it was not used for the first few pipes in scheme 5 and was not properly carried out for parts of scheme 7 due to pump failure – but low resistance when it was effective.

14. Local interface friction angles obtained from the contact stress cells are summarized in Table 2. These were obtained from plots such as those in Fig. 6, from scheme 4. Here the effects of partial lubrication may be seen. Data points in line A refer to pipe-soil contact before lubrication commenced, given an interface friction angle of 39.2° . Line C was obtained shortly after injection of bentonite slurry into the overbreak; normal stresses were reduced as the pipe

became partially buoyant and the interface friction angle reduced to 15.3° . Line B was obtained some time later as the effects of lubrication were beginning to dissipate, allowing an increase in both normal stresses and friction angle.

15. Frictional behaviour like this is expected in sandy soils; more surprisingly, apparently frictional behaviour *in terms of total stress* was also observed at the pipe-soil interface in cohesive soils, even for high shearing rates in high plasticity clay. Some results from scheme 3 are shown in Fig. 7; behaviour is 'frictional' at lower stress levels, though there is perhaps a tendency towards a limiting shear stress at higher contact stresses. Frictional behaviour with no upper limit was observed in the low- to medium-plasticity stiff glacial clay and weathered mudstone of schemes 1, 8 and 2; more cohesive behaviour was observed in softened glacial clay at the end of scheme 1 and in the soft alluvial clay of scheme 9.

16. Although significant pore pressures were measured in the clay soils, plotting of results in terms of effective stress greatly increased the scatter; the intermittent contact between soil and pipe and the resulting uncertainty about full saturation of the pore pressure cells means that the data from these cells are less reliable than the other field measurements. A possible explanation of the apparently frictional behaviour in total stress terms in the cohesive soils is considered later.

Measured and calculating jacking forces

17. Plots of measured jacking resistance typically show an initial 'face resistance' intercept due to the interaction between the shield and the tunnel face, and then an increasing resistance with pipe length due to the interaction between pipes and ground. An example (from scheme 4) is shown in Fig. 8. The face resistance was normally small, as the miner excavated slightly outside the front edge of the shield, and the rate of increase of line resistance with pipe length was fairly constant. The occasional peaks were due to changes in face resistance when the shield was used to trim the excavation at the tunnel face. The introduction of lubrication greatly reduced the frictional force until problems were encountered with running sand right at the end of the drive. Apparent frictional forces (jacking resistance per unit length of pipe) were obtained from all the schemes monitored and are given in the final column of Table 2.

18. Jacking resistances calculated using one of the simple theoretical models are also listed in Table 2 and compared with the measured values. Where the simple sliding model with frictional contact is appropriate, calculated values are quite close to measured ones, but typically some 10–20% low. It is suggested that

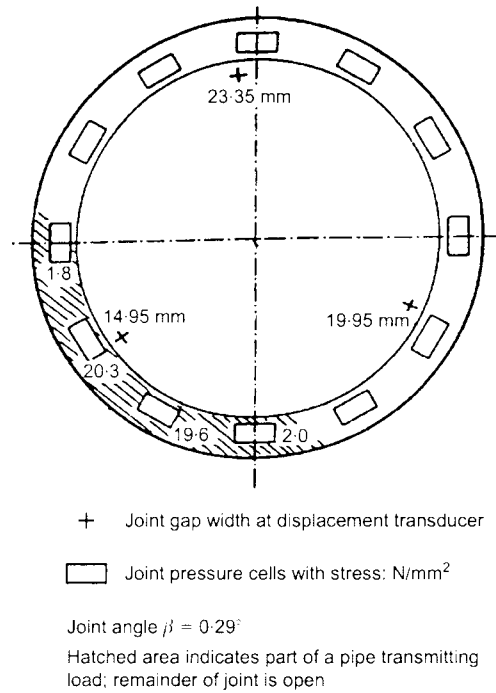


Fig. 4. Pressures in pipe joint pressure cells

this discrepancy is due to the effects of pipeline misalignment as discussed later. In two cases the simple sliding cohesive model appears to match the measured value, and in each case the resistance is much higher than with frictional behaviour; at present it is not clear when the cohesive model should apply. In the two cases of cohesionless soils below the water table, where the Terzaghi model was appropriate, the calculated jacking resistance slightly exceeded the measured one at Cheltenham, but greatly exceeded it at Southport; in the latter case some residual effects of lubrication may have reduced the field value. In scheme 9 the overall jacking resistance of 25 kN/m is equivalent to an average interface shear stress around the pipe of 4.5 kPa , which probably equates to the remoulded undrained strength of the soft peaty clay.

Interface friction in cohesive soils

Theoretical analysis

19. One of the more surprising results from the field monitoring was the apparently frictional behaviour at the pipe-soil interface, even when expressed in terms of total stresses, in cohesive soils. This was found both with low-plasticity glacial clay and high-plasticity London clay. Only when the measured shear stresses approached the undrained strength of the soil was a trend towards cohesive or adhesive interface behaviour observed. The values of interface friction angle quoted in Table 2 represent best-fit average values over the range of stress levels

Table 2. Calculated and measured jacking resistance

Scheme	Soil type	Notes	Theory	Parameters: see notes	Calculated friction force: kN/m	Measured friction force: kN/m
1 Bolton	Stiff glacial clay	Dry	Frictional	$\delta = 19^\circ$	6.1	7.2
2 Gateshead	Weathered mudstone	Wet	Haslem	$\alpha s_u = 80 \text{ kPa}$	24.1	29.8
3 Honor Oak	London clay	First 40 m	Frictional	$\delta = 17^\circ$	7.0	8.0
4 Chorley	Dense silty sand/ sandy silt	—	—	$\delta = 13^\circ$	—	54.4
5 Cheltenham	Sand and gravel (below water table)	Unlubricated	Frictional	$\delta' = 38^\circ$	18.7	23.1
		Unlubricated	Terzaghi	$\delta = 15^\circ$	6.5	9.4
		Lubricated	—	$\phi' = 32^\circ$	11.0	10.0
6 Leyton	London clay	Partial lubrication	Frictional	$\delta = 13^\circ$	5.4	10
			—			12.7 average
			—			6.7 lower bound
7 Southport	Dense silty sand (below water table)	Full lubrication	Frictional (buoyant)	$\delta' = 37.5^\circ$	2.3	2.6
		Imperfect lubrication	Terzaghi	$\phi' = 43^\circ$	7.9	41.6
8 Seaham	Stiff glacial clay	Unlubricated	Frictional	$\delta = 19^\circ$	9.2	4.8
		Lubricated	Haslem	$\alpha s_u = 120 \text{ kPa}$	4.4	2.0–15
9 Thurrock	Very soft clay and peat	Unlubricated	—	—	—	2.5
		Lubricated	—	—	—	1.4

Notes: αs_u = adhesion between clay and pipe.

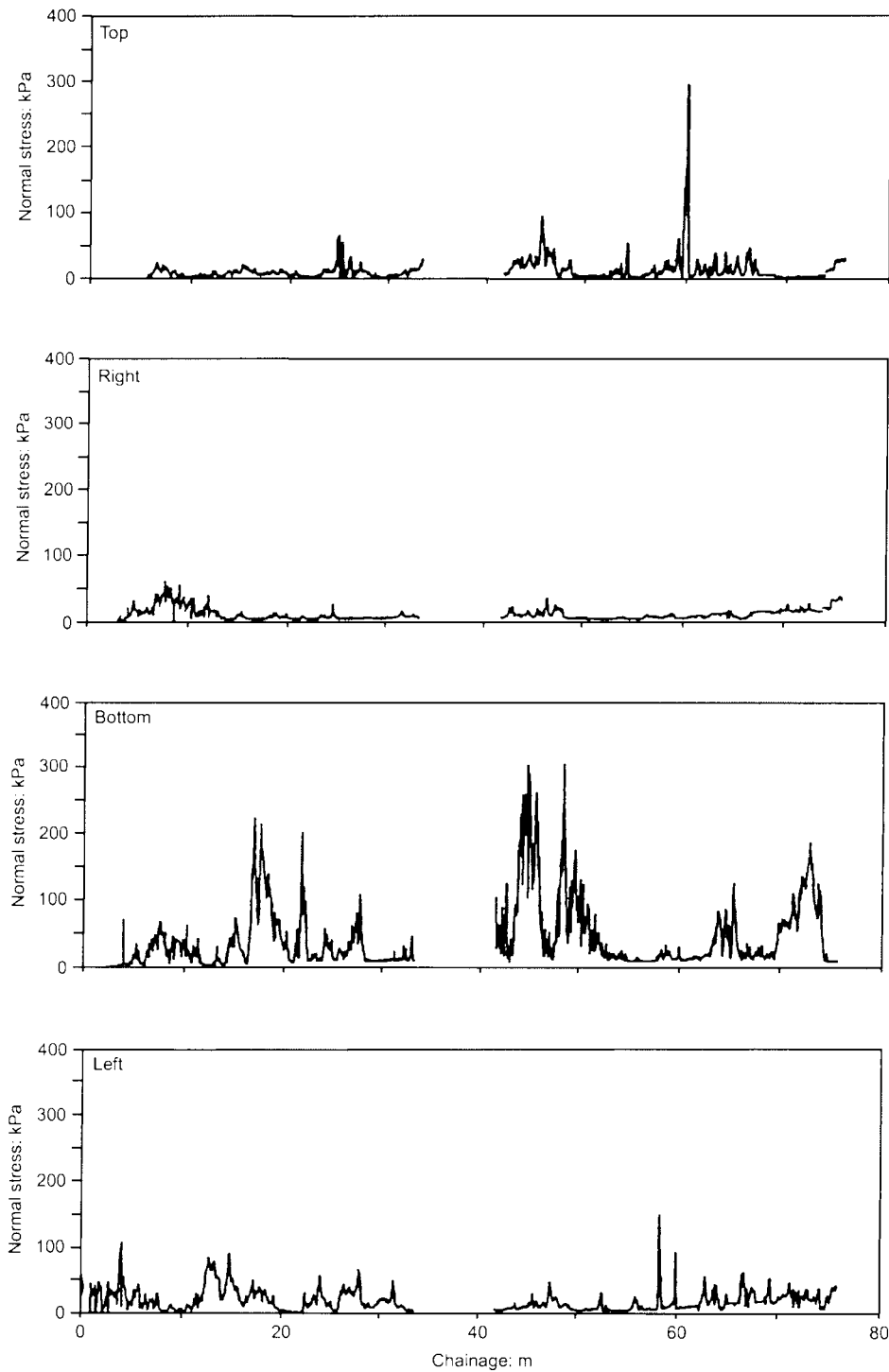


Fig. 5. Pipe-soil contact stresses (scheme 4)

measured, but much higher values were sometimes measured at low stress levels, for instance up to about 30° in London clay.

20. The other obvious feature of the measured interface stresses is that they vary very rapidly as the pipes advance. This presumably results from a combination of the pipes being a

'loose fit' within the tunnel bore and the roughness of the concrete and more particularly the soil surface, so that intimate contact is not maintained (even at the base of the pipe) between the soil and any point on the pipe. The picture is probably more of a relatively rigid surface making sporadic and irregular contact

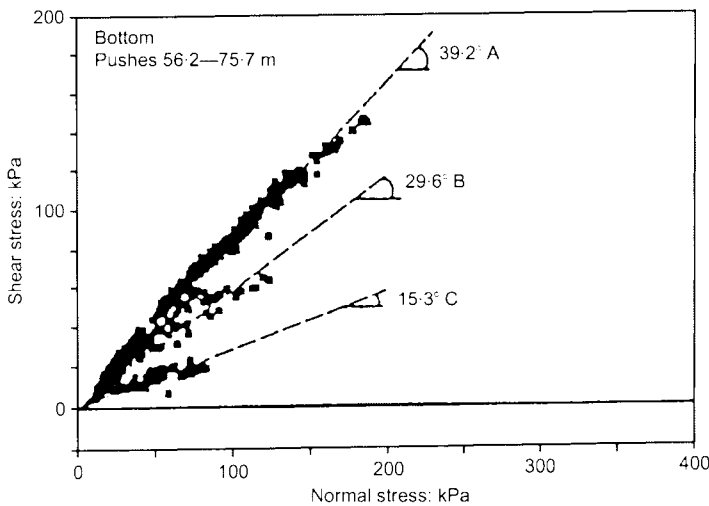


Fig. 6. Shear stress/normal stress relations (scheme 4). (Note different scales for shear and normal stresses)

with fairly localized areas of soil surface. It is suggested that this conceptual model can then give rise to the apparently frictional interface behaviour noted earlier.

21. Assume that the pipe initially makes contact with fairly localized high points or asperities of soil (Fig. 9(a)). Taking these for simplicity as being of truncated conical shape, with 45° side slopes, an interaction diagram (Fig. 10) may be obtained which relates the shear and normal stresses (τ , q) causing failure under undrained conditions in the cohesive soil (Appendix 2, calculation 1). If the pipe–soil contact is initially under normal load only, the asperity must squash until the normal contact stress is reduced to the value at point A. During jacking, the interface shear stress will increase, causing failure and further compression of the asperity so that the normal stress (for constant normal load) reduces until stable conditions are reached at point B. At this point there is a constant ratio between shear and normal stress of 1, given an apparent interface friction angle of 45°. Note that this is the lowest possible angle for this scenario; if the normal stress subsequently reduces, sliding can occur with stresses corresponding to a point such as C, for which the τ/q ratio is increased.

22. If the normal loading increases, the asperity may be squashed until it is much wider than it is high (Fig. 9(b)). The corresponding failure interaction diagram (Fig. 11) is then obtained from the standard analysis for bearing capacity under combined normal and shear loading (Appendix 2, calculation 2). The apparent friction angle at point B is then 21.3°.

23. As normal stresses increase further, the local contact areas between pipe and soil will increase until they start to coalesce; finally there will be intimate contact between pipe and soil over most of the pipe surface and failure will be at constant shear stresses equal to the un-

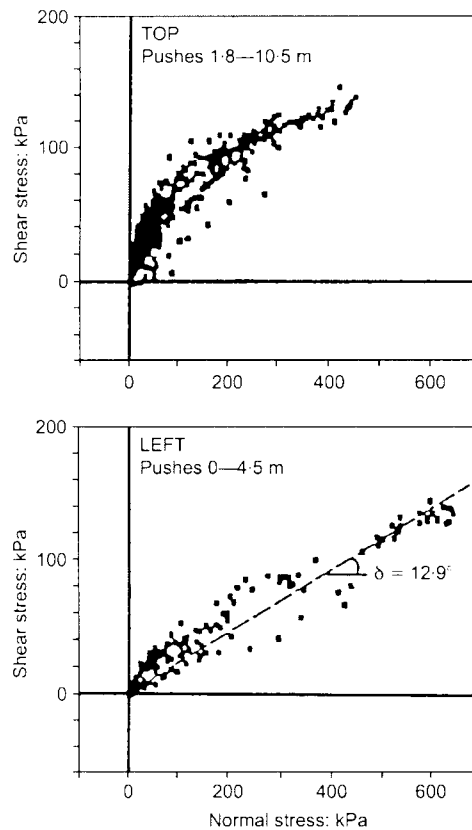


Fig. 7. Shear stress/normal stress relations (scheme 3). (Note different scales for shear and normal stresses)

drained strength (or perhaps some rather lower 'adhesion' value for pipes with smooth surfaces).

24. This proposed model of interface behaviour appears to match qualitatively the field observations. However, the pipe–soil contact in practice will be very complex, with many points of contact of different size and slope, subjected to various load paths, so that no precise quantitative treatment is possible. The apparent friction angles predicted by the calcu-

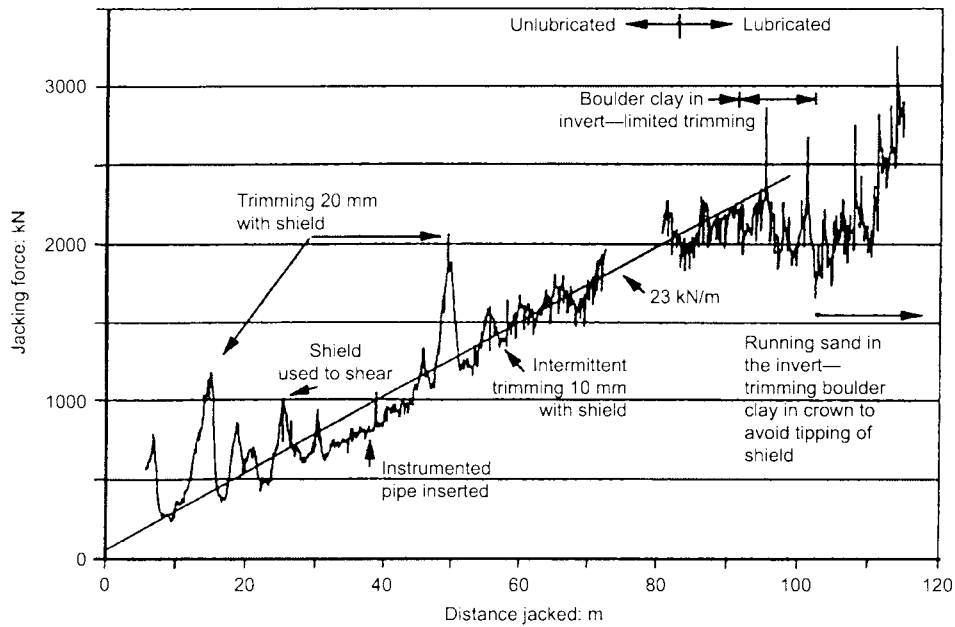


Fig. 8. Jacking load record (scheme 4). (Note: 'trimming' refers to the use of the leading edge of the shield to complete the excavation at the face after the bulk of the soil has been excavated by the miner)

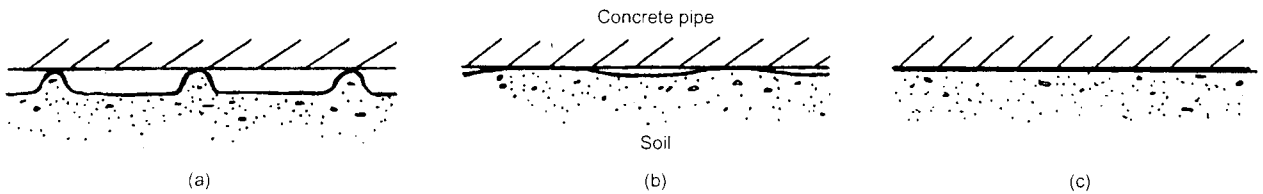


Fig. 9. Conceptual models for pipe/soil interaction showing: (a) localized 'asperities'; (b) limited contact areas; and (c) full area contact

lations cover most of the range observed in practice. The very low value observed in London clay presumably reflects residual conditions for large-displacement shearing concentrated into a thin shear band at the interface.

25. The situation may be further complicated by the fact that, even at high shearing rates, there is some drainage occurring from the thin shear zones; in addition, the clay at the excavated surface may not be fully saturated. The fact that interface shear behaviour tends, as stress level increases and shear strength reduces, towards that expected for full contact with saturated clay under undrained conditions, may reflect a combination of all the effects previously discussed.

Laboratory tests

26. An attempt was made to verify the hypothesis by interface shear tests in the laboratory.¹⁰ Tests were performed using a standard Casagrande apparatus, but to model the field conditions as closely as possible a concrete block was used with a sliding fit inside the upper part of the box (Fig. 12(a)). Tests

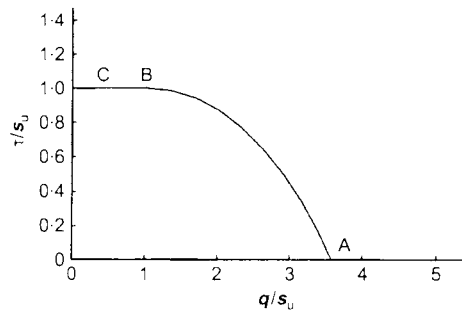


Fig. 10. Interaction diagram for failure of soil 'asperities' (q is the normal stress on asperity, τ is the shear stress on asperity, s_u is the undrained strength of clay)

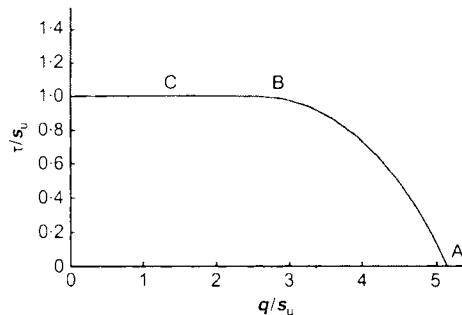


Fig. 11. Interaction diagram for failure at localize soil-pipe contact areas. (q is the normal stress on contact area, τ is the shear stress on contact area, s_u is the undrained strength of clay)

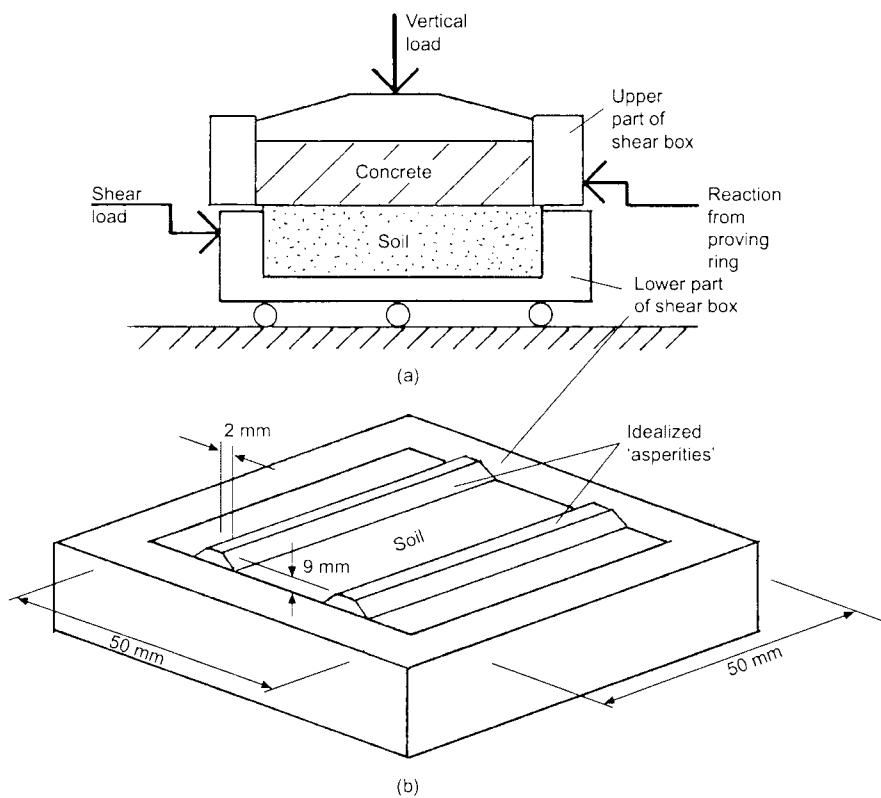


Fig. 12. Interface tests, showing: (a) schematic of general arrangement; and (b) idealized asperities (not to scale)

were performed both with a nominally flat surface to the soil in the lower part of the box and with carefully formed asperities in the surface (Fig. 12(b)). Tests were performed with London clay from one of the field test sites, but it was very difficult to prepare specimens due to the highly fissured nature of the soil. Tests with asperities therefore used modelling clay (kaolin with a high-viscosity pore fluid, which has a suitable consistency to allow easy specimen preparation).

27. The tests with asperities behaved much as expected, as shown in Fig. 13. The asperities were initially compressed under the vertical load, and then compressed further with shear load. Under low apparent vertical stress (vertical load divided by the total plan area of the shear box), this compression ceased a little after the peak shear load had been reached; at higher stresses both shear load and compression were still increasing at the end of the test. The apparent interface friction angle from the tests, based on the maximum shear load reached in each test, is 38° (see Fig. 14), which is slightly less than predicted by the theory. For the first three tests, which had already reached a reasonably steady state by the end of the test, careful measurements were made of the dimensions of the asperities at the end of the test, so that actual average contact stresses could be determined; the resulting normal stresses varied between 90 and 98 kPa and the shear stresses

between 60 and 77 kPa. In comparison, tests on nominally flat surfaces of the modelling clay suggested frictional interface behaviour with peak friction angles varying between about 45° at low stresses and 41° at higher stresses; corresponding values for tests using London clay were 29° and 22° . All these values lie within the range of the two theoretical calculations, but it might have been expected that the values for the modelling clay would be nearer the lower end of the range as the stress levels were high enough for small asperities to be squashed and relatively large contact areas established. The results were therefore somewhat inconclusive.

Effects of misalignments

28. Previous authors^{11,12} have tried to explain the increase in jacking force due to misalignments by arguing that the axial jacking forces cause increased contact stresses with the soil on the outside of a bend. While this model must apply for a pipeline following a long curve of constant radius, the field measurements have shown a very different model of behaviour to apply to pipe jacks deviating by typical amounts from a straight line. Comparison of Figs 3 and 5 shows that, in a drive with fairly rapid variations in level but little horizontal deviation (except at the end), high contact stresses occur at the high points in the base of the tunnel, at chainages of about 18, 45 and 70 m. Fig. 15 shows similar data for a drive with large

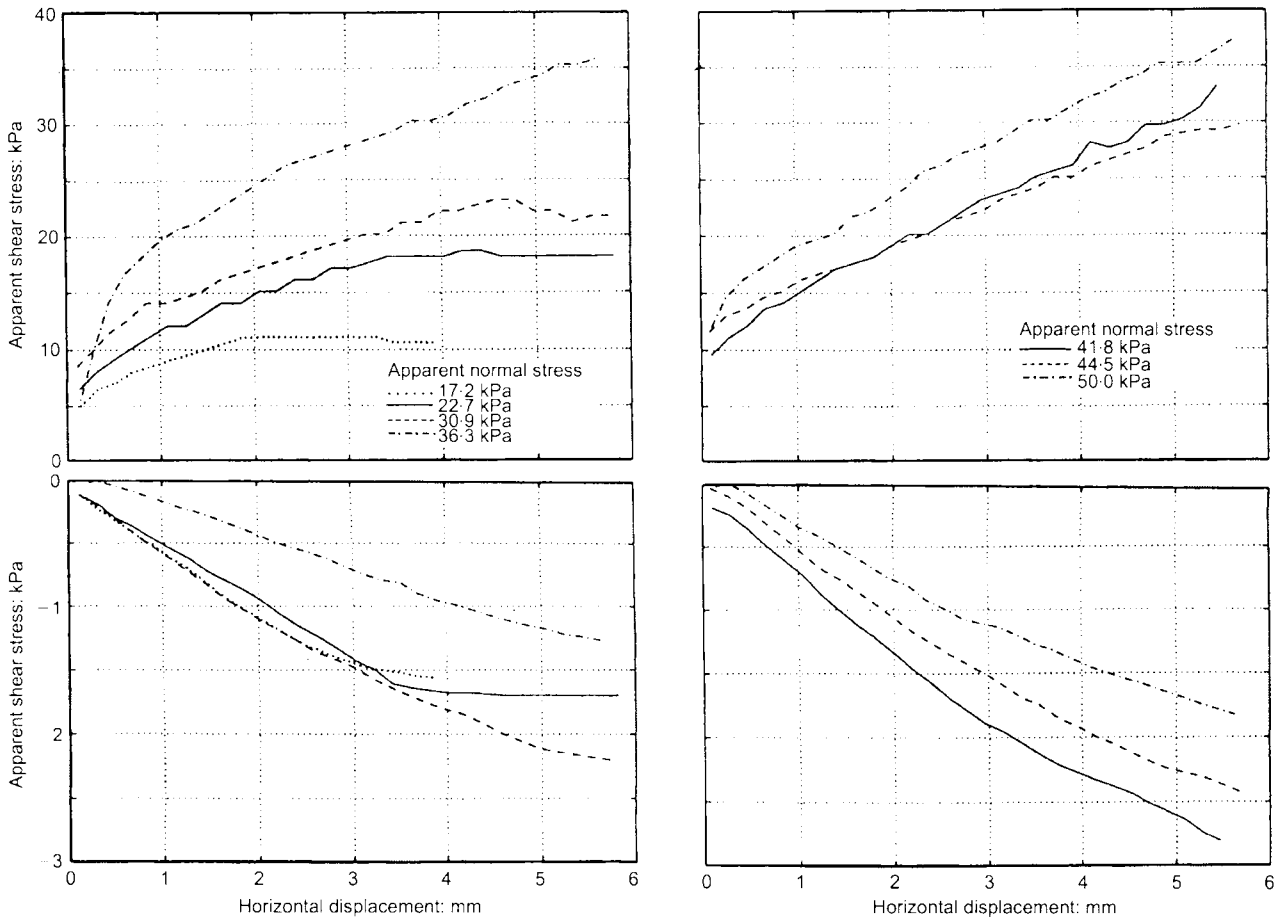


Fig. 13. Results of interface tests with asperities (modelling clay)

horizontal but small vertical deviations; here contact is made with the sides of the tunnel on the *inside* of each bend at the points of maximum curvature, as indicated diagrammatically in Fig. 16. In each case the pipeline appears to be acting as a prestressed segmental beam, tending to span between the high points for variations in level and press against the side walls for horizontal bends.

29. In the former case there does not usually seem to be significant contact with the roof of the tunnel bore and the overall frictional resistance is therefore not changed. However, contact with the sides of the tunnel causes frictional forces additional to those due to the self-weight of the pipes. A simplified model for the forces acting on a half-wave length (l) of pipeline is shown in Fig. 17. P_1 and P_2 are the axial jacking forces, N_1 and N_2 the additional lateral contact forces between pipe and ground, W the self-weight per unit length of pipeline, and δ the interface friction angle between pipe and soil. At the two points of maximum curvature the axial forces are assumed to act close to the edge of the pipe, of internal diameter D .

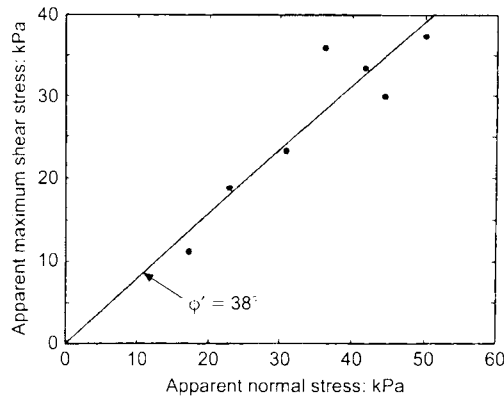


Fig. 14. Apparent interface friction angle for tests with asperities

30. Then, for lateral equilibrium

$$N_1 = N_2 = N \quad (2)$$

for longitudinal equilibrium

$$P_2 = P_1 + Wl \tan \delta + N \tan \delta \quad (3)$$

and for moment equilibrium about point X

$$P_1 D + Wl \tan \delta \frac{D}{2} = \frac{Nl}{2} - \frac{N}{2} \tan \delta D \quad (4)$$

From equation (4)

$$N = \frac{(2P_1 + Wl \tan \delta)D}{(l - D \tan \delta)} \quad (5)$$

and from equations (3) and (5)

$$P_2 = (P_1 + Wl \tan \delta) + \frac{D \tan \delta (2P_1 + Wl \tan \delta)}{(l - D \tan \delta)} \quad (6)$$

$$= \frac{P_1(l + D \tan \delta)}{(l - D \tan \delta)} + \frac{Wl^2 \tan \delta}{(l - D \tan \delta)}$$

A similar calculation may be applied to any subsequent half-wave length, with forces increasing from P_2 to P_3 and so on.

31. As an example, consider a 100 m length of pipeline with two complete waves of 50 m length. Assume a face resistance at the shield of 500 kN, and the following values

- $D = 1.5 \text{ m}$
- $W = 24 \text{ kN/m}$
- $\tan \delta = \tan 29^\circ = 0.55$
- $l = 25.0 \text{ m}$

The resulting calculated jacking forces at points along the pipeline are given in Table 3. The amount by which the jacking force exceeds that for a perfectly straight line is cumulative, giving a percentage increase of nearly 20% by the end of the drive. The second calculation is for the same pipeline but with wavelengths of 25 m ($l = 12.5 \text{ m}$). The jacking force is increased very significantly, the final value being more than doubled.

32. For a direct comparison with data from a real site, consider the first 35 m of the drive at scheme 1 (ground conditions changed significantly after this point). The alignment survey and jacking records are shown in Fig. 18. The face load was 120 kN, the measured interface friction angle 19° , and the weight of the pipes 17.7 kN/m . The horizontal deviation shows two half-wave lengths of about 17.5 m, and the value of D was 1.20 m. The calculated jacking loads at 17.5 and 35.0 m are then 235 and 355 kN respectively, compared with 227 and 333 kN for a perfectly straight pipeline. The measured

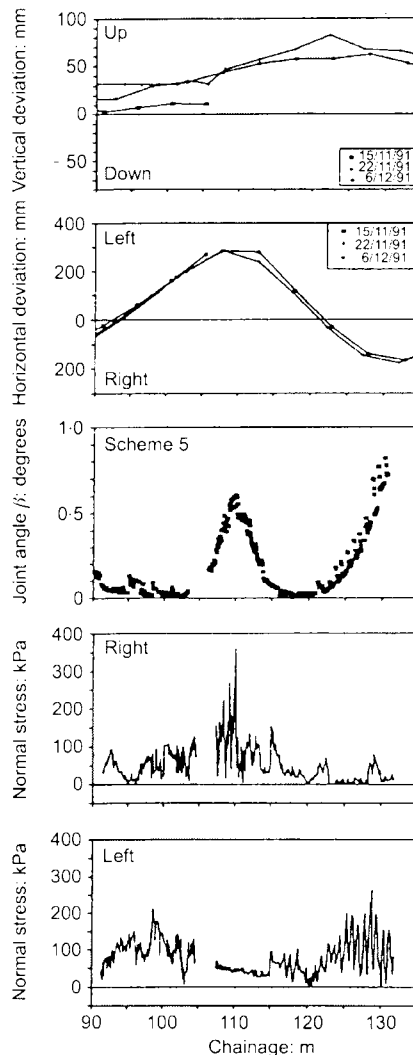


Fig. 15. Pipe-soil contact stresses and pipeline alignment

values were 246 and 372 kN respectively. The new calculation does not account for all the measured increase, but a closer agreement would be obtained if a realistic distribution of the reaction force N was used; as the lever arm for this force is reduced (in the moment equilibrium equation), its magnitude must increase, leading to greater additional frictional forces.

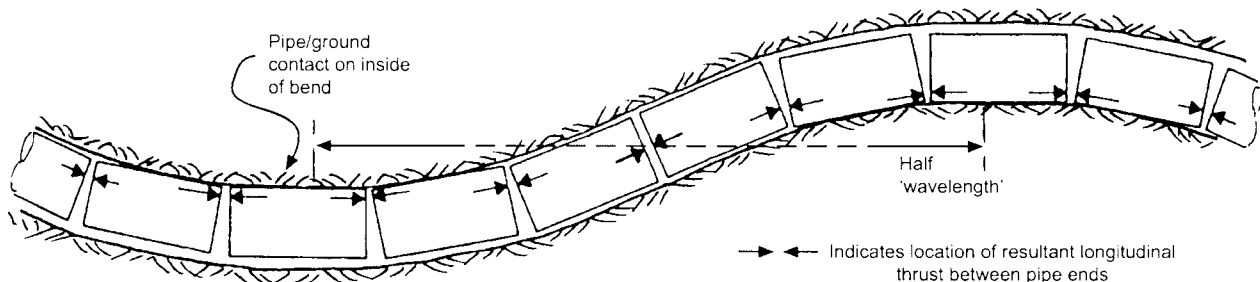


Fig. 16. Pipe-soil interaction on a 'wavy' line. (Note: pipe deviations and tunnel overcut greatly exaggerated)

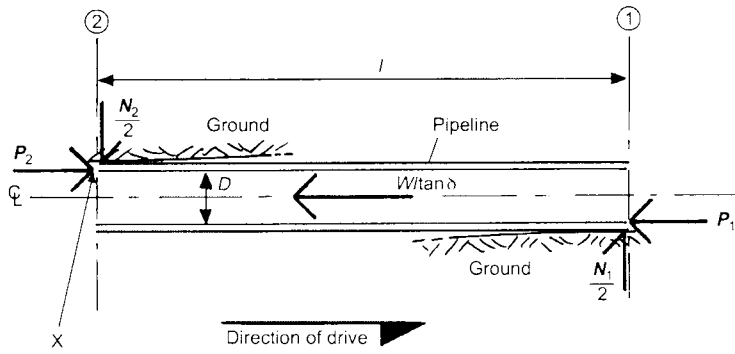


Fig. 17. Theoretical model for pipe-soil interaction

Table 3 Jacking loads for straight and 'wavy' pipelines

Description	Distance from face: m				
	0	25	50	75	100
	Face load: kN	Jacking load kN			
For straight pipe	500	830	1160	1490	1820
For 50 m 'wavelength'	500	875	1276	1704	2161
% increase in load	—	5.4	10.0	14.4	18.7
For 25 m 'wavelength'	500	1030	1719	2617	3788
% increase in load	—	24	48	76	108

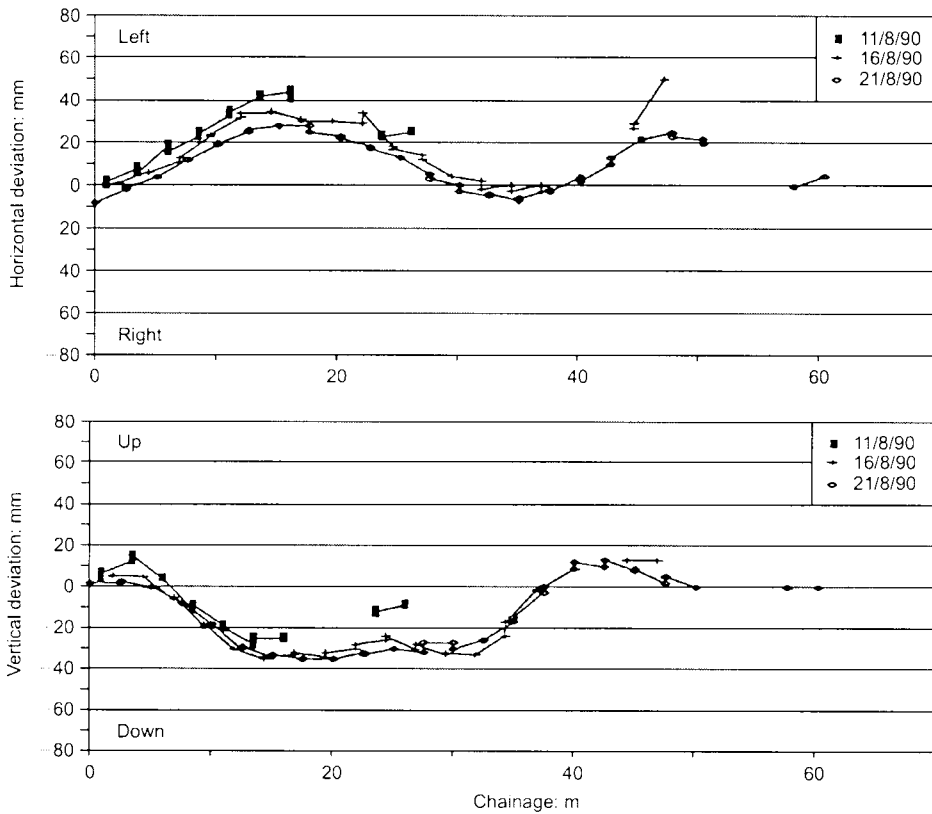


Fig. 18. Alignment survey and jacking record, scheme 1

33. The main value of this calculation is less for back-calculating jacking forces than for clarifying the important variables controlling the increase in jacking force. The additional friction is increased by reduction of the length l and by

increases in D and δ . This emphasizes the need for particularly careful control of large-diameter pipelines in sand and silty soils with relatively high friction angles; fortunately, the use of full lubrication by bentonite slurry in the overbreak

analysis will be very effective in these conditions.

Time effects

34. A further important factor to take into account is the increase in jacking load observed during periods when the pipeline is not moving. Some typical results from scheme 6, in London clay, are shown in Fig. 19. After each stoppage the jacking resistance reduces as the pipeline is jacked, to the current 'base line' value, then jumps up again during the next stoppage. In this highly plastic clay, increases in jacking force are detectable after only a few minutes, and can lead to increases of 50% or more after a few hours. Collected data from a number of stages of schemes 3 and 6 are plotted in Fig. 20, showing percentage increases with time on a logarithmic scale. Although there is some scatter, the trend

is clear. Similar increases were observed in the low to medium plasticity clays, but the magnitudes were much smaller, with a maximum of about 35% over a long weekend break. No time effects were observed in cohesionless soils or in the soft clay.

Conclusions

35. Simple theoretical models of pipe-soil interaction during pipe jacking have been presented, and values of jacking resistance calculated using them compared with values measured in the field. Reasonable agreement is obtained provided the appropriate model and interface parameters are chosen. A new analysis explains some of the increase in jacking resistance due to imperfections in alignment of nominally straight pipelines being jacked through a stable bore with a small overbreak on

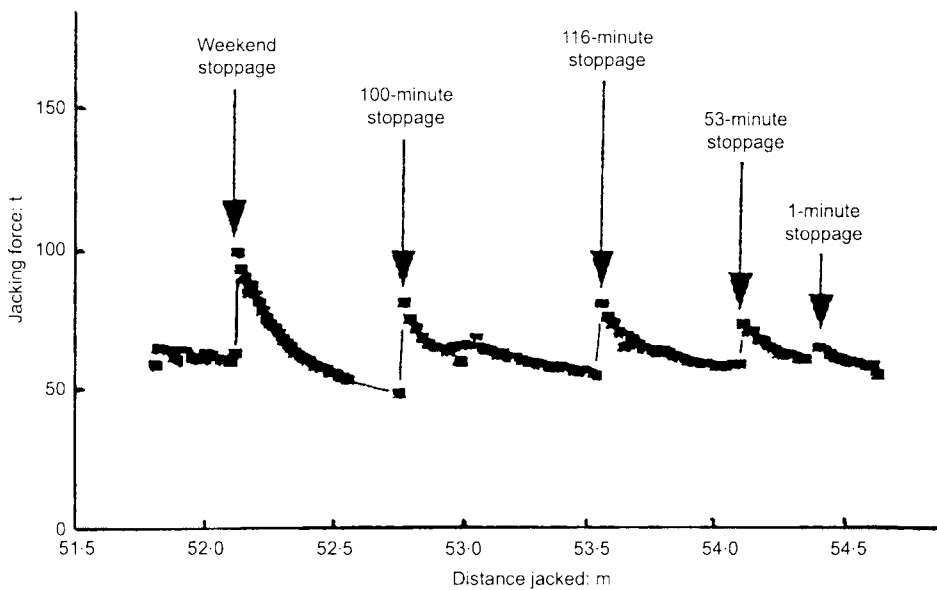


Fig. 19. Effect of stoppages on jacking force

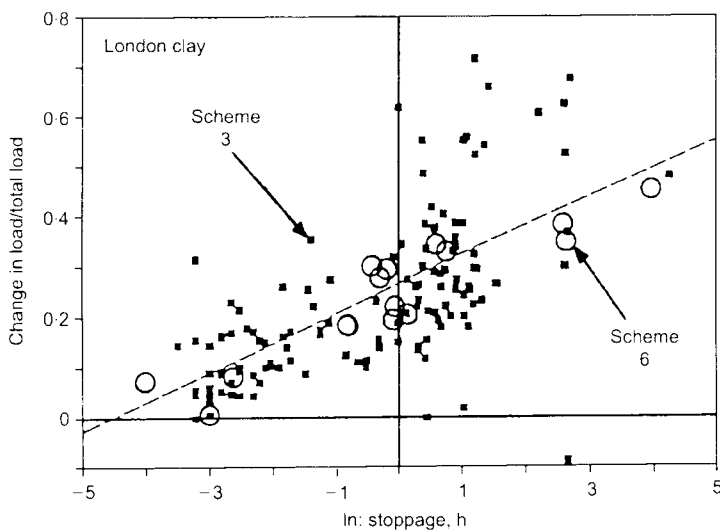


Fig. 20. Time effects for schemes 3 and 6

the outside diameter of the pipes. It highlights the important factors affecting the increase in force, and emphasizes the need for accurate site control of pipe jacks and for very gradual correction to deviations from the correct alignment.

36. Explanations have also been sought for the apparently frictional behaviour in terms of total stress at the interface between pipes and ground in firm to stiff cohesive soils. Although the analysis appears to capture the main features of results from both laboratory and field tests, accurate prediction of the interface behaviour is not possible, and jacking loads are best estimated using the interface friction angles measured directly in the field monitoring. Further work is needed to clarify the conditions under which frictional sliding changes to cohesive interface behaviour, which results in much higher resistance to jacking.

37. Time effects are very important in high plasticity clays, of less significance in stiff low plasticity clays, and negligible in soft clays and cohesionless soils.

38. High resistance to jacking occurs in unstable cohesionless soil, but the resistance may be reduced to very low levels by the use of pressurized bentonite slurry to maintain the stability of the tunnel bore and allow the pipes to become buoyant within the fluid.

Acknowledgements

39. The authors acknowledge with thanks the permission of Mark Marshall, the research assistant for sites 6-9 of the research work, to use data provided by him for this paper. The field monitoring work was made possible by substantial financial support from the Engineering and Physical Sciences Research Council, the Pipe Jacking Association and five water utility

companies: Northumbrian, North West, Severn Trent, Thames and Yorkshire. The water companies also provided the sites on which the field monitoring was carried out; Anglian Water kindly provided access to site 9. Site work was only possible with the cooperation of site operatives and supervisory staff too numerous to mention individually. The project also benefited greatly from the assistance of Ron Morton (electronics engineer) and Bob Earl and Bob Sawala (laboratory technicians). The laboratory shear box tests were carried out as a fourth-year undergraduate project by Rebecca Thompson with the support of the Department of Engineering Science, University of Oxford.

Appendix 1

40. When soil has collapsed onto the pipe, the total frictional resistance is given by

$$F = \frac{\pi D_e}{2} (\sigma'_v + \sigma'_h) \tan \delta \tag{7}$$

where the parameters are as defined in Fig. 21, δ is the angle of friction between the pipe and the soil, and σ'_h is given by

$$\sigma'_h = K(\sigma'_v + 0.5\gamma' D_e) \tag{8}$$

where γ' is the submerged unit weight = $\gamma - \gamma_w$, γ is the bulk unit weight, γ_w is the unit weight of water, and K is the coefficient of earth pressure.

41. To determine σ'_v consider the vertical equilibrium of a thin layer of soil as shown in Fig. 21. Below the water table, at depth H_1

$$2B\sigma'_v + 2B\delta z\gamma' - 2B(\sigma'_v + \delta\sigma'_v) - 2\delta z = 0 \tag{9}$$

where

$$\tau = K\sigma'_v \tan \phi' \tag{10}$$

42. Therefore, substituting and rearranging

$$\gamma' \delta z = \frac{1}{\left(1 - \frac{K \tan \phi'}{\gamma' B} \sigma'_v\right)} \delta \sigma'_v \tag{11}$$

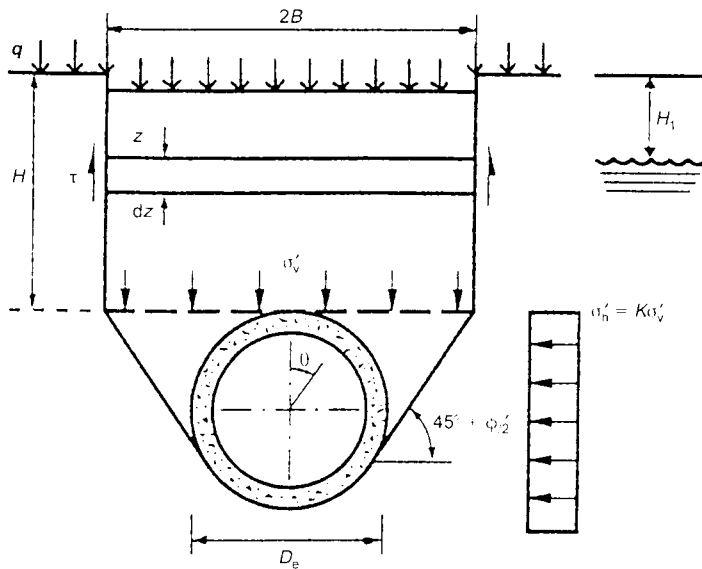


Fig. 21. Ground loading from Terzaghi model

Integrating,

$$\gamma'z = \left(\frac{-\gamma'B}{k \tan \phi'} \right) \ln \left(1 - \frac{K \tan \phi'}{\gamma'B} \sigma'_{v1} \right) + C \quad (12)$$

43. If $\sigma'_v = \sigma'_{v1}$ at $z = H_1$ then

$$C = \gamma' H_1 + \frac{\gamma'B}{K \tan \phi'} \ln \left(1 - \frac{K \tan \phi'}{\gamma'B} \sigma'_{v1} \right) \quad (13)$$

Substituting and rearranging gives, in general at depth z

$$\sigma'_v = \frac{\gamma'B}{K \tan \phi'} \left(1 - \exp \left[- \frac{(z - H_1) K \tan \phi'}{B} \right] \right) + \sigma'_{v1} \exp \left[- \frac{(z - H_1) K \tan \phi'}{B} \right] \quad (14)$$

For the stress at the top of the pipe, substitute $z = H$.

44. The value of σ'_{v1} is found from a similar analysis for the soil above the water table, using bulk unit weight rather than submerged unit weight, so that

$$\sigma'_{v1} = \frac{\gamma B}{K \tan \phi'} \left(1 - \exp \left[- \frac{H_1 K \tan \phi'}{B} \right] \right) + q \exp \left[- \frac{H_1 K \tan \phi'}{B} \right] \quad (15)$$

where q is a surcharge at the ground surface.

45. Note also that

$$B = \frac{D_e \tan(45^\circ - \phi'/2)}{2} + \frac{D_e}{2 \sin(45^\circ + \phi'/2)} \quad (16)$$

Appendix 2

Calculation 1: Failure of a conical asperity

46. Consider a flat-topped asperity with 45° side slopes carrying a normal stress q and shear stress τ . A potential failure mechanism is shown in Fig. 22(a), and the corresponding displacement diagram in Fig. 22(b).

47. Equating work terms

$$qL\Delta + \frac{\tau L\Delta}{\tan \alpha} = \frac{L \cos \alpha \Delta}{\sin \alpha} s_u + \frac{R\Delta}{\sin \alpha} s_u + \frac{2R\Delta}{\sin \alpha} \alpha s_u \quad (17)$$

where the final term represents the work done in the fan zone B, and s_u is the undrained strength of the soil. From geometry, $R = L \sin \alpha$, and equation (17) reduces to

$$q + \frac{\tau}{\tan \alpha} = s_u \left(\frac{1}{\tan \alpha} + 1 + 2\alpha \right) \quad (18)$$

48. Minimizing q gives

$$\sin \alpha = \sqrt{\frac{1}{2} \left(1 - \frac{\tau}{s_u} \right)} \quad (19)$$

49. By substitution of different values of α , the interaction diagram shown in Fig. 10 is obtained.

Calculation 2: Failure at a localized contact area

50. The corresponding failure mechanism and displacement diagram for a localized contact area are shown in Figs 23(a) and (b). A similar calculation gives

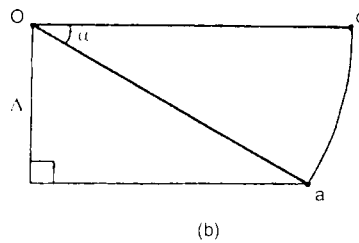
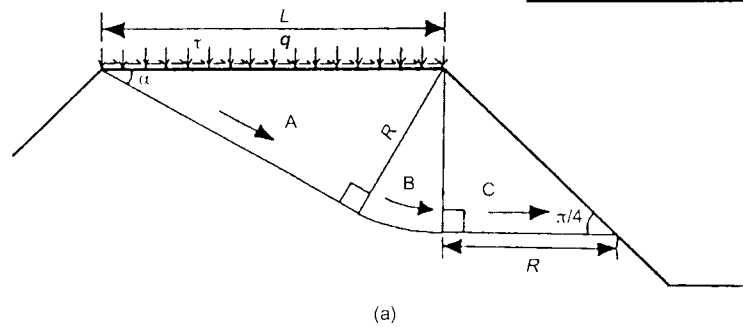


Fig. 22. Diagrams illustrating: (a) failure of a conical asperity; and (b) corresponding displacement

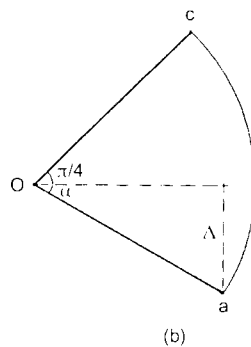
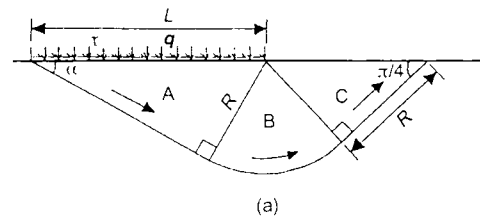


Fig. 23. Diagrams illustrating: (a) failure at a local contact area; and (b) corresponding displacement

$$q + \frac{\tau}{\tan \alpha} = s_u \left(\frac{1}{\tan \alpha} + 1 + 2 \left[\alpha + \frac{\pi}{4} \right] \right) \quad (20)$$

with $\sin \alpha$ as before. The resulting interaction diagram is as shown in Fig. 11.

51. In each case the apparent interface friction coefficient is given by the ratio τ/q .

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