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## Ductile iron jacking pipes

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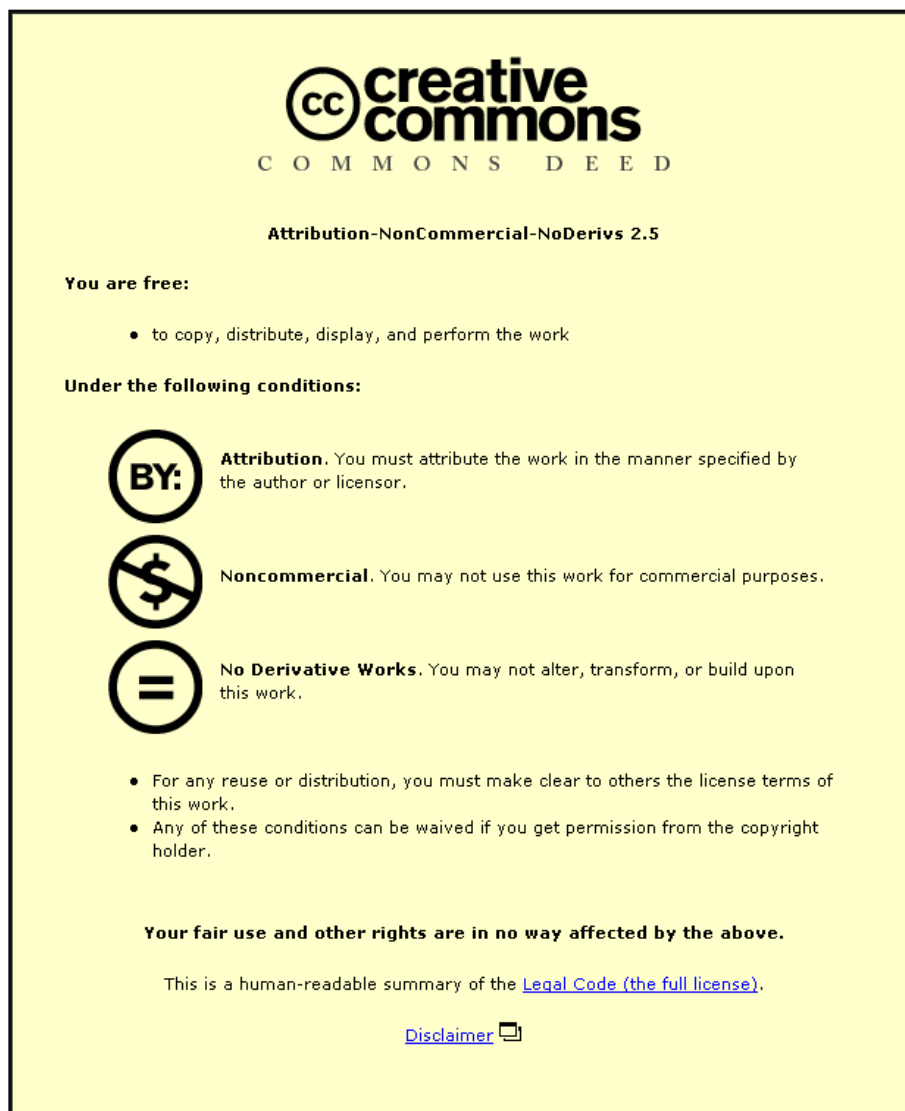
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# **DUCTILE IRON JACKING PIPES**

by

**ANDREW MARTIN SCOTT BSc**

**A Masters Thesis**

**submitted in partial fulfillment of the requirements  
for the award of Master of Philosophy for Loughborough University of  
Technology**

**JUNE 1994**

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	CONTENTS	PAGE
	CONTENTS	ii
	ABSTRACT	v
	ACKNOWLEDGEMENTS	vi
	LIST OF FIGURES	vii
	LIST OF TABLES	x
	LIST OF PLATES	xi
	NOMENCLATURE	xii
CHAPTER 1.	INTRODUCTION	1
1.1	History and Background	2
1.2	Aims and Objectives	3
1.3	Contents of Thesis	4
CHAPTER 2.	LITERATURE REVIEW	5
2.1	Introduction	6
2.2	Trenchless Pipelaying Methods	7
2.2.1	Impact Ramming	8
2.2.2	Directional Drilling	8
2.2.3	Impact Moling	10
2.2.4	Pipe Bursting	11
2.2.5	Swaging and Rolldown	13
2.3	Pressure Pipe Design	17
2.3.1	Ductile Iron Pipes	19
2.3.2	Glass Reinforced Plastic Pipes	21
2.3.3	Steel Pipes	23
2.3.4	Unplasticised Polyvinyl Chloride Pipes	24
2.3.5	Medium Density Polyethylene Pipes	25

2.4	Pipe Design for Trenchless Methods	28
2.4.1	Ductile Iron Jacking Pipes	29
2.4.2	Glass Reinforced Plastic Jacking Pipes	30
2.4.3	Other Jacking Pipe Designs	30
2.5	Pipe to Soil Interaction	31
2.5.1	Pipe Jacking Forces	31
2.5.2	Ground Movements during Pipe Installation	34
2.6	Current and Future Water and Gas Industry Requirements	38
CHAPTER 3.	RESEARCH PHILOSOPHY AND APPROACH	68
3.1	Introduction	69
3.2	Methods of Investigation	70
CHAPTER 4.	STUDY OF PIPE TO SOIL INTERACTION	71
4.1	Introduction	72
4.2	Effect of Joint Profile on Jacking Loads	73
4.3	Ground Movements During Pipe Installation	77
4.3.1	Measurement Techniques	77
4.3.2	Vertical Sand Movement	77
4.3.3	Induced Pipe Strains	79
4.4	Concluding Remarks	80
CHAPTER 5.	MECHANICAL PIPE AND JOINT DESIGN	92
5.1	Introduction	93
5.2	Joint Design and Evaluation	93
5.2.1	Joint Design	93
5.2.2	Gasket and Jointing Tests	96
5.2.3	Pressure Tests	100

5.3	Jacking Load Transfer	100
5.3.1	Model Joint Axial Load Tests	100
5.3.2	Full Length Axial Load Tests	105
5.4	Protection Against Corrosion	111
5.5	Conclusions	115
CHAPTER 6.	FIELD PERFORMANCE	142
6.1	Introduction	143
6.2	Description of Trial	143
6.2.1	Site Description and Location	143
6.2.2	On site Procedure	144
6.3	Jacking Loads	144
6.4	Ground Movements	147
6.5	Corrosion Protection	150
6.6	Site Procedure	150
6.7	Field Trial Conclusions	151
CHAPTER 7.	CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK	158
7.1	Ductile Iron Jacking Pipes	159
7.2	Recommendations for Future Work	161
REFERENCES		162
APPENDIX A		

## DUCTILE IRON JACKING PIPES

### ABSTRACT

Installation of pipes in urban areas requires a pipe product possessing high strength and pressure ratings which can be used with laying techniques causing minimum surface disruption. This thesis reviews trenchless pipe laying techniques and presents the findings from a detailed study concerning the mechanics of the pipe bursting process. The development of a new ductile iron joint for pipe bursting applications was undertaken and testing of the joint in the laboratory and in the field is described in detail.

The thesis includes a detailed analysis of the mechanics of load transfer at the joint and this shows that during jacking under deflected conditions, there is high concentration of load which limits the maximum potential jacking length. However, the laboratory trials enabled the maximum jacking length to be predicted and this is considered sufficient for most practical applications. The studies also established the extent of ground movements and the level of strain induced in adjacent services during pipe bursting operations. Methods of protecting the pipe and joint are considered with emphasis placed on the performance of various coatings.

The results from a full scale field trial are presented and the overall performance of the new joint and proposed laying system is evaluated. A successful method of installation was developed and parameters including jacking load and ground movement are compared with the laboratory results.

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Andrew Scott

June 1994

## LIST OF FIGURES

Figure 2.1	Longitudinal Section through a Pipe Jacking Scheme
Figure 2.2	The Impact Ramming Method
Figure 2.3	Directional Drilling Procedure
Figure 2.4	Directional Drilling Pullback Operation
Figure 2.5	Impact Moling
Figure 2.6	Pipe Bursting using a Shield Expander
Figure 2.7	Pipe Bursting using a Hydraulic Expander
Figure 2.8	Swaging and Rolldown Operations
Figure 2.9	Stress-Strain Curves for Ductile Iron
Figure 2.10	Ductile Iron Pipe Joints
Figure 2.11	GRP Pipe Joints
Figure 2.12	Steel Pipe Joints
Figure 2.13	uPVC Push Fit Joints
Figure 2.14	Jointing Methods for MDPE Pipes
Figure 2.15	Kubota Jacking Pipes
Figure 2.16	PAM Prototype Jacking Pipe Design
Figure 2.17	German Jacking Pipe Design
Figure 2.18	Prototype Ductile Iron Jacking Pipe (Stanton PLC)
Figure 2.19	GRP Jacking Pipe Design
Figure 2.20	Allowable Jacking Loads for GRP Pipes
Figure 2.21a	Clay Jacking Pipes
Figure 2.21b	Concrete Jacking Pipe Designs
Figure 2.22	Graph showing increases in Jacking Loads with Stoppage Time (after Milligan and Norris 1992)
Figure 2.23	Comparisons of Theoretical and Actual Jacking Loads for Pipe Jacking (after Water Services 1981)
Figure 2.24	Ground Movements caused by Pipe Bursting Operations (after Leach and Reed 1989)
Figure 2.25	Pipe Bursting Surface Damage and Proximity Charts (after Leach and Reed 1989)
Figure 2.26	Strain Gauge Readings during a Pipe Bursting Operation (after Leach and Reed 1989)
Figure 2.27	Ground Movements associated with Pipe Bursting Tests (after Chapman 1992)

Figure 2.28	Displacement Contour Plots for Pipe Bursting (after Chapman 1992)
Figure 2.29	Total Displacement Vector Plot for Pipe Bursting (after Chapman 1992)
Figure 2.30	Surface Heave Profiles for Pipe Bursting Model Tests in Sand (after Swee and Milligan)
Figure 4.1	Standard Ductile Iron Socket
Figure 4.2	Layout of Test Trench during Laboratory Trials
Figure 4.3	The Four Stages of the Laboratory Trial
Figure 4.4	Jacking Loads for Lines 1 and 2 (laboratory trial stages c and d)
Figure 4.5	Positions of Soil Displacement Rods
Figure 4.6	Locations of Strain Gauges
Figure 4.7	Sand Movements for Line 1
Figure 4.8	Sand Movements for Line 2
Figure 4.9	Comparisons of Surface Heave
Figure 4.10	Induced Hoop Strain in Line 2 due to Molding in Line 1
Figure 4.11	Longitudinal Strain in Pipe 3, Line 2
Figure 4.12	Longitudinal Strain in Pipe 5, Line 2
Figure 4.13	Modes of Pipe Deformation
Figure 5.1	Detail of Ductile Iron Pipe Joint for Trenchless Methods
Figure 5.2	Detailed Drawing of the New Joint
Figure 5.3	Detailed Drawing of the New Gasket
Figure 5.4	Gasket Test Rig
Figure 5.5	Graph of Model Jointing Force Versus Spigot Travel
Figure 5.6	Jointing Force for Prototype Joint
Figure 5.7	Pressure Test Conditions
Figure 5.8	Gasket Position during Eccentric Pressure Tests
Figure 5.9	Arrangement during Model Joint Axial Load Tests
Figure 5.10	Positions of Strain Gauges during Model Axial Load Tests
Figure 5.11	Strain Gauge Readings during Axial Load Test 1 (AL1)
Figure 5.12	Strain Gauge Readings during Axial Load Test 3 (AL3)
Figure 5.13	Layout of Trench for Full Length Axial Load Tests
Figure 5.14	Positions of Strain Gauges for Full Length Axial Load Tests
Figure 5.15	Details of K12 Pipe used for Full Length Axial Load Tests

Figure 5.16	Strain Gauge Readings during Full Length Axial Load Test (FAL1)
Figure 5.17	Strain Gauge Readings during Full Length Axial Load Test (FAL2)
Figure 5.18	Strain Gauge Readings during Full Length Axial Load Test (FAL3)
Figure 5.19	Strain Gauge Readings during Full Length Axial Load Test (FAL 6)
Figure 5.20	Details of Coating Scratch Test
Figure 5.21	Polyurethane Coating Scratch Test Results
Figure 5.22	Polyester Resin Coating Scratch Test Results
Figure 5.23	Single Layer Glass Reinforced Polyester Resin Coating Scratch Test Results
Figure 5.24	Multi Layer Glass Reinforced Polyester Resin Coating Scratch Test Results
Figure 5.25	Shield for Prototype Collar
Figure 6.1	Site Layout for Field Trial
Figure 6.2	Method of Connecting Pipe to the Bursting Machine
Figure 6.3	Procedure for Pipe Installation
Figure 6.4	Geometry of Deflected Pipe during Field Trial
Figure 6.5	Graph showing Field Trial Jacking Loads and Corresponding Contact Stresses
Figure 6.6	Sub-surface Movements Measured during Trial, Compared with Predicted Ground Movements (after Chapman, Rogers 1991)
Figure 6.7	Likely Ground Movements caused by Pipebursting Operations (after Leach and Reed , 1989)

## LIST OF TABLES

Table 2.1:	Pipebursting projects carried out by Severn Trent Water
Table 2.2:	Pipebursting projects carried out by Anglian Water
Table 2.3:	Technical Data on pipebursting projects from the market survey
Table 2.4:	Broad classification of water supply pipe materials according to method of application (after De Rosa, Hoffman, Oliff (1988))
Table 2.5:	Mechanical properties of ductile iron
Table 2.6	Lower bound pressure ratings of steel pipe to BS 534 (after De Rosa, Hoffman, Oliff, 1988)
Table 2.7:	Jacking resistance for various ground conditions (after Concrete Pipe Association of Australia)
Table 2.8:	Comparison between the observed displacements in laboratory pipebursting tests and those predicted using fluid flow analyses (after Chapman (1992))
Table 4.1:	Jacking load comparisons between socket-spigot and smooth profile pipes
Table 4.2:	Comparisons between contact stress values measured during the laboratory trial and those predicted by theoretical analysis for smooth profiles pipes
Table 5.1:	Comparison between predicted and actual jointing forces
Table 5.2:	Results of prototype jointing trial
Table 5.3:	Strains recorded at contact point during model joint axial load tests
Table 5.4:	Description of full length axial load tests
Table 5.5:	Comparison of axial loads calculated from the measured strain and those recorded during full length axial load tests
Table 5.6:	Strains recorded during cyclic full length axial load test (FAL4)
Table 6.1:	Maximum daily jacking loads recorded during field trial and corresponding contact stress
Table 6.2:	Surface heave caused by pipebursting operation during field trial (after Rogers, Robins, Scott (1990))

## **LIST OF PLATES**

<b>Plate 4.1:</b>	<b>Test trench during preparation for laboratory trial</b>
<b>Plate 5.1:</b>	<b>Detail of prototype jointing system</b>
<b>Plate 5.2:</b>	<b>Detail of prototype jointing system</b>
<b>Plate 5.3:</b>	<b>Model gasket jointing rig</b>
<b>Plate 5.4:</b>	<b>Model gasket trials showing behaviour of gasket during jointing</b>
<b>Plate 5.5:</b>	<b>Prototype joint under pressure test</b>
<b>Plate 5.6:</b>	<b>Deflected load test</b>
<b>Plate 5.7:</b>	<b>Damage to polyurethane coating on barrel</b>
<b>Plate 5.8:</b>	<b>Damage to polyurethane coating on socket</b>
<b>Plate 5.9:</b>	<b>Damage to tape wrap coating on barrel</b>
<b>Plate 5.10:</b>	<b>Damage to tape wrap coating on socket</b>
<b>Plate 6.1</b>	<b>Field Trial Jacking System</b>

## NOMENCLATURE

D	External diameter of pipe (m)
$F_A$	Adhesive force of pipe to soil ( $\text{kN/m}^2$ )
$F_D$	Force per metre diameter of leading edge of shield (kN/m)
$F_E$	Total earth force on pipe (kN)
$F_T$	Total pipe jacking force (kN)
$F_U$	Total force on upper half of pipe (kN)
$F_L$	Total force on lower half of pipe (kN)
f	Coefficient of frictional resistance due to the weight of pipe
H	Cover depth (m)
K	$1 - \sin \phi$
	$1 + \sin \phi$
L	length of drive (m)
$P_1$	Horizontal soil pressure ( $\text{kN/m}^2$ )
$P_2$	
$P_F$	Cutting face force ( $\text{kN/m}^2$ )
$P_w$	Mud water pressure for slurry machines ( $\text{kN/m}^2$ )
$P_v$	Vertical earth pressure on pipe ( $\text{kN/m}^2$ )
$P_h$	Horizontal earth pressure on pipe ( $\text{kN/m}^2$ )
$P_r$	Radial soil pressure
$P_s$	Soil pressure ( $\text{kN/m}^2$ )
R	Frictional force between soil and pipe ( $\text{kN/m}^2$ )
S	Circumference of pipe (m)
W	Weight of pipe (kN)
$W_h$	Weight of pipe per unit surface area ( $\text{kN/m}^2$ )
$\alpha$	Weight of soil per unit volume ( $\text{kN/m}^3$ )
$\lambda$	Coefficient for curved drives
$\delta$	Angle of wall friction
$\delta_v$	Maximum of surface heave
$\theta$	Angle for determining radial pressure
$\phi$	Internal friction angle of soil
$\mu$	Coefficient of friction between soil and pipe

## **CHAPTER ONE**

### **INTRODUCTION**

## 1.1 HISTORY AND BACKGROUND

The Greeks and later the Romans placed great importance on providing water supply and sewerage systems which are considered fundamental to the development of these civilisations. Much laudable engineering took place to create these systems.

Provision of a potable water supply and efficient sewerage system is now taken for granted in the European way of life. The total installed length of public water supply in Europe is estimated to be 2.1 million kilometres. The UK water supply system consists of 350,000 km of water mains and some 75% of this system is estimated to be laid in cast iron having either grey or spheroidal graphite metal structure.

In the UK, estimates of the water loss occurring through the pipe systems vary between 20 and 40%. In Germany and France this figure is believed to be approximately 5 - 10%. For Germany the figure is a reflection of the good condition of many urban networks which were replaced after the second world war. In contrast, some 45% of the total length of UK water mains was laid before 1945 and furthermore, much of this was laid before 1940 in unlined grey cast iron.

These figures show that there is an enormous potential market for renovating and/or replacing the ageing water supply system in the UK. Open cut methods are normally used when renovating water mains and these often require sections of road to be closed down causing disruption to traffic flow. With the number of road vehicles expected to double within the next twenty years, there will be an increasing demand to reduce the number of road closures in order to avoid unnecessary congestion.

In 1985 the Horne Report (Horne, 1985) was issued which recommended an update of the 1950 Public Utilities Streetwork Act. This report studied the quality of road repair, the responsibilities of statutory undertakers, duration of road works and disruption to traffic flow. Lane rentals were recommended whereby the contractor pays a rental to the utility companies, thereby providing an incentive to devise construction systems which minimise disruption. These proposals were generally welcomed by local authority associations and utility companies, yet almost a decade later they have still not been adopted. However, with the growing need for renovation of an ageing water supply network together with the increasing traffic volume, there is a concomitant growing need for trenchless pipelaying methods.

Trenchless pipelaying has been widely used for installing and replacing sewer systems. Sewers are well suited to trenchless methods because they are often laid at depths exceeding 2 metres, where there is little risk of damage to adjacent services and where open cut costs are greater due to the increased depth. Water mains are commonly laid near the surface where there is greater inherent risk of damaging other services and hence there has been reluctance to use trenchless methods for installing and replacing water mains. Where trenchless methods have been used for water mains, they have generally utilised plastic pipe materials. Thus there is a perception that trenchless methods are not suited to traditional pressure pipe materials such as ductile iron and steel. The work reported in this thesis aims to determine whether there is any validity in this perception.

## **1.2 AIMS AND OBJECTIVES**

This thesis aims to review trenchless pipelaying techniques and pressure pipe design in sufficient detail to be able to devise a programme of work to satisfy the research objectives which were as follows:

- 1.2.1 To establish an understanding of the physical parameters concerned with trenchless pipe-laying methods in order to enable jacking loads and ground movements to be quantified.**
- 1.2.2 To determine the most suitable trenchless method for laying ductile iron pipe, taking account of potential markets and technical considerations.**
- 1.2.3 To develop a ductile iron pipe product to be used in association with the recommended method chosen in 1.2.2.**
- 1.2.4 To establish an effective site procedure for installation of the product developed using the chosen installation method.**

The research was thus directed towards gaining a greater understanding of the technical parameters concerned with trenchless techniques and towards developing a technique for installing more traditional pipe materials such as ductile iron by trenchless methods.

### **1.3 CONTENTS OF THE THESIS**

**Chapter 2 contains a review of trenchless pipelaying methods and pressure pipe design. The requirements of pipes used in association with trenchless methods methods are discussed and current designs are presented. A concise review of literature concerning pipe jacking forces and ground movements is given, and this is followed by a discussion concerning water industry requirements.**

**Chapter 3 discusses how the research was directed and gives an overview of the main methods of investigation.**

**Chapter 4 is a description of a full scale laboratory trial which assesses the overall suitability of ductile iron as a replacement pipe material used with the pipe bursting process. Particular emphasis is placed on determining the effect of the external profile of such pipes on the level of jacking load and on the extent of ground movements.**

**Chapter 5 describes the design and development of a ductile iron pipe joint suitable for use with the pipe bursting process. Particular emphasis is placed on determining the criteria for a gasket design. The mechanics of jacking load transfer at the pipe joint are studied and a number of pipe corrosion protection systems are assessed.**

**In Chapter 6 a field trial is described, the aim of the fieldwork being to assess the criteria established in Chapters 2, 4 and 5 and to confirm the laboratory observations. This involved replacing a section of grey iron main with ductile iron pipes using the pipe bursting technique and the new jointing system described in Chapter 5.**

**Chapter 7 concludes the work and describes practical applications for the results. Recommendations are made for future investigations.**

## **CHAPTER TWO**

### **LITERATURE REVIEW**

## 2.1 INTRODUCTION

Trenchless pipelaying was first introduced into the UK in the late 1950's in the form of pipe jacking. However, it is only since the beginning of the 1980's that significant advances in trenchless technology have taken place. Such progress includes the introduction of new pipelaying techniques and has led to a greater understanding of the engineering parameters such as ground movements and jacking loads. Perhaps more importantly, the capabilities and limitations of the various techniques are better understood. The series of International No-Dig Conferences have served as a forum for exchanging ideas and information and have led to a greater awareness of the work being undertaken in countries such as Japan and Germany in addition to the UK. There are two main reasons for this increased interest in trenchless pipelaying. The first is the growing awareness of the many social or indirect costs of laying pipes by traditional open-cut methods. Such costs range from wear and tear on vehicles using badly reinstated roads, to loss of productive time for travellers. The second reason for this growing interest is the increasing problem of traffic congestion in large cities and smaller urban areas.

There are several different types of pipe material available for use with trenchless pipelaying techniques including plastics, concrete, steel and ductile iron. In general, each trenchless pipelaying method has developed to suit a particular pipe material although many techniques can be used with a variety of materials. In order to fully understand which materials are suited to particular trenchless methods, it is necessary to have a good understanding of both the pipelaying methods and the pipe design particular to each material. This chapter gives a review of five trenchless methods and presents design criteria for the five main pressure pipe materials. This is followed by a review of pipe designs which have been developed specifically for trenchless applications.

It is important to understand the extent of ground movements caused by the tunnelling operation and also the jacking loads which are likely to occur. Some laboratory based research has already been carried out to gain a better understanding of both ground movements and jacking loads. The results have been compared with site measurements by a number of authors (Rogers et al 1989, Chapman 1992, Leach and Read 1989). A review of predicted and measured ground movements and jacking loads is presented in this chapter.

## 2.2 TRENCHLESS PIPELAYING METHODS

Developments in pipelaying methods have produced a variety of techniques which enable pipe installation with minimum disturbance to surface features such as roads rivers and railways. The term "Trenchless Pipelaying Technology" is now widely understood and accepted as a method of describing the variety of techniques used to install pipes with minimum surface disturbance. Trenchless pipelaying systems can be categorised as either new pipe installation or renovation. Within each of these categories the soil can either be excavated or displaced by the operating machine and the pipes may be pulled or pushed into place directly behind the machine.

New pipe installation involves construction of a completely new pipeline and includes the most well known methods of installation such as Pipe Jacking, Microtunnelling and Auger Boring. These three methods are based on the basic technique of pushing pipes into the ground following some kind of excavation. A typical pipe jacking scheme layout is shown in Figure 2.1. This type of work is perhaps the most documented of all trenchless techniques and is described by numerous authors. Kramer, McDonald and Thomson (1992) give full descriptions of the techniques and highlight some of the more salient technical features. This work includes an introduction to trenchless methods and this is followed by a discussion on the economics of such methods when compared with traditional open cut. The work then describes seven case histories and reviews the future needs of trenchless technology. Ripley (1990) studied the load distribution on model concrete jacking pipes and described the pipes in some detail. Stevens (1989) describes the load carrying capacity of concrete coated ductile iron jacking pipes. Other authors have produced a variety of papers on pipe jacking, thereby establishing a useful reference source (Craig and Moss, 1984, Hough, 1986 and Thomson, 1993). Since other techniques are less well documented, the following gives a description of the principal methods. A brief outline of each technique is given and any important features are highlighted.

### 2.2.1 Impact Ramming

The impact ramming technique involves forcing the pipe into the ground from a launch pit as shown in Figure 2.2. Pipe advance is normally achieved by percussive hammer loading using an impact device fixed to the trailing pipe in the launch pit. The impact device thus travels the length of the launch pit and is returned to the rear of the pit as each new section of pipe is added. For pipe diameters of less than 150mm, the leading pipe may be closed and soil is displaced and compacted as the pipe advances. For larger diameters, the end of the pipe is left open and the core of soil is subsequently removed by water jetting, air jetting or mechanical cutting.

The percussive impact device generates extremely large thrust forces of up to 1000 tonnes. This thrust is transferred to the pipe via a tapered add-on nose cone. If the thrust forces are sufficiently large, the end of the pipe may spread out radially and if this occurs the end of the pipe is cut off. With these large thrust forces and since there is no method of controlling direction once the drive has commenced, it is important that a rigid joint is used for impact ramming work. Consequently, steel pipe with welded joints is used which ensures full contact is maintained round the circumference of the pipe thereby avoiding any deflection. Present designs of mechanical joints allow angular deflection and are not able to withstand high thrust forces due to concentrated loading when this occurs.

Impact ramming can be carried out in clays, silts, peats, sands, gravels and cobbles. The percussive action assists in breaking up and displacing stones or other obstructions. There may be some risk of excessive ground movement as soil is either displaced in front of the pipe or flows into it in unstable ground. The length of drive attainable by impact ramming is heavily dependent upon the ground conditions. Most work has been carried out using a drive length of less than 40m. Once the drive is underway there is very little control over alignment and obstructions can push the pipe off line. Overall accuracy depends on initial alignment of the pipe and drive equipment, ground conditions and length of bore.

### 2.2.2 Directional Drilling

Directional drilling is primarily used for rapid installation of pipelines under large obstacles such as rivers and multiple lane transport routes. The technique is derived from that used in oil field drilling technology, drilling being carried out from a large surface mounted rig. A pilot hole of approximately 80mm diameter is first drilled using a "down hole" mud motor head or a water

jetting head. Both devices cut through the soil without rotating the pilot drill string. Special drilling fluid (or "mud") is pumped to the head through the hollow drill string. The fluid is pressurised to provide power for the motor and additives in the fluid provide lubrication and hole stabilisation. Soil cuttings are carried back along the drill string by the returning fluid. During the drilling operation a washover pipe of approximately 125mm diameter is installed over the pilot string, following approximately 80 - 100m behind the drill head to provide rigidity to the pilot string as shown in Figure 2.3.

Once the drilled pilot hole has been completed, the pulling head of the pipeline to be installed is connected to the washover pipe via a swivel, universal joint and barrel reamer. The washover pipe is then rotated and pulled back by the drill rig, simultaneously over reaming the drilled hole and pulling the pipeline into the drilled hole as shown in Figure 2.4.

Ground conditions suitable for directional drilling are described as firm to very stiff clay having 0.002 to 0.06mm particle size and coarse sands having 0.6 to 2mm particle size. Very coarse granular material is not suitable nor is rock with the exception of very soft rock.

The oil industry regularly uses directional drilling with drive lengths of several kilometres. However there are high costs attributed to installing the drilling rig and associated equipment, so most pipelaying operations are undertaken over 200 - 500m drive length. Drilling can be undertaken in the 50 - 1000mm diameter range.

The drill entry angle is controlled by raking the drill carriage by between 5 and 20 degrees. Control of direction once drilling is underway is achieved by rotating a small bend or "bent" positioned close behind the drill head. The directional equipment casing is also located in the drilling head. This contains instruments to give magnetic bearing of drilling direction, angle of inclination and position.

The considerable stresses involved in a long pull require the use of a pipe with adequate tensile strength. Most frequently, steel pipe with welded joints is pulled into place either as a permanent line or to form a duct into which most types of pipe can be installed. Reynolds and Sczupak (1987) describe two directional drilling projects carried out in 1986. Hair and Shiers (1985) give a full description of the method and conclude by stating that directionally drilled river crossings offer an acceptable alternative to conventional methods. In addition, they state that directional drilling allows otherwise difficult river crossing sites to be reconsidered and offer further benefits to the designer and client.

### 2.2.3 Impact Moling

Impact or percussive moling is a technique which relies on displacement and compaction of the ground to form a void into which the pipe can be inserted. As shown in Figure 2.5, the impact mole is essentially a torpedo shaped percussive hammer. The majority of these machines are air-powered and are driven by a medium sized compressor. The compressed air drives a hammer against an anvil on the chiselling head and the frictional forces between the soil and the outer casing resist the recoil forces produced as the hammer moves backwards. The net result is that the mole is hammered forward through the ground. The product pipe is normally pulled into the ground by the advancing mole or can be jacked into place behind the mole.

It is stated in the literature that impact moles can operate in a wide range of soil conditions from soft to very stiff. The type, homogeneity and water content of the soil all affect mole performance. In soft saturated clays undrained shearing will take place and an approximation to fluid flow will occur, coupled with consolidation on dissipation of the excess porewater pressures that will be generated by the operation. In loose sands, densification will occur in the vicinity of the mole and progress will similarly be swift. In dense granular soils and very stiff, heavily overconsolidated clay soils radial displacement is difficult due to their low compressability and so progress can be expected to be slow. A further observation suggests that soils with a higher water content are able to deform, and recover and this allows rapid progress of the mole. However, friction between the mole and the soil is required to resist backward movement during recoil of the hammer and thus the presence of excess water reduces the overall efficiency of the operation.

In ideal conditions runs of over 100m have been successfully completed, but most work is undertaken over short spans of 25m or less. The overall accuracy of the drive is largely dependent upon the initial alignment of the mole, which is determined by the use of special alignment equipment. The homogeneity of the soil is also important since presence of voids, soft pockets or obstructions will cause the mole to deviate. Once in progress there is no method of control and direction is not normally monitored.

Impact moling can be used to install pipes from 50 to 250mm inside diameter. In the larger sizes a two or even three stage process may be employed. Plastic or steel pipes are normally pulled into place by the mole.

British Gas have made extensive use of moling techniques (Howe and Hunter, 1985 and Smith and Jameson, 1987) and claim advantages which include reduced costs, reduced highway interference and maintenance along with less disruption to road users.

British Telecom have also made use of moling techniques (Renshaw, 1987) using PVC pipe up to 130mm inside diameter. Although modern moles have stepped heads which aid directional stability British Telecom considered the development of a steerable mole would be a distinct advantage. Recently a steerable water jetting system was developed using technology based on the directional drilling technique.

#### 2.2.4 Pipe Bursting

Pipe bursting is a pipe replacement technique by which an existing pipe is expanded using radial force from inside the pipe. This breaks the existing pipe into fragments which are forced into the surrounding soil creating a void for the new pipe. The concept was originally developed in the UK in the early 1980s by British Gas in conjunction with contractors DJ Ryan and Son. Stein et al (1989) present a thorough discussion of moling and pipebursting techniques and their equipment. This work gives a good introduction to expansive installation techniques.

Traditional pipe bursting methods use torpedo shaped percussive moles, similar to those used for pneumatic impact moling described in section 2.2.3. A tapered shield or expander which has an external diameter greater than the existing pipe is fitted over the head of the impact mole as shown in Figure 2.6. As the mole proceeds inside the pipe the expander transmits radial forces to the pipe wall since it has a larger diameter than the inside diameter of the existing pipe. The pipe is consequently broken into fragments which are pressed into the surrounding soil.

In 1988 a hydraulic pipe expander was developed which uses hydraulic energy to apply the radial forces. This type of mole has a series of interleaved segments which form the nose cone. Once entered into the pipe the mole is expanded whilst stationary and the radial forces rupture the existing pipe, pressing the fragments into the surrounding soil. The series of operations for this type of machine is shown in Figure 2.7.

Whether an impact or hydraulic energy is used, the mole is generally guided by a constant tension winch connected to the nose of the mole. The new replacement pipe is pulled into place directly by the mole or is jacked behind the mole as it advances.

Pipe bursting was originally developed to replace ageing grey cast iron gas and water mains. For these mains a Polyethylene (PE) or Polyvinyl Chloride (PVC) liner was installed behind the mole and a medium density Polyethylene (MDPE) product pipe was inserted into the liner once the pipe bursting operation was complete. The liner was used to guarantee against the possibility of abrasive damage to the MDPE. The new pipe must withstand service pressures of up to 16 bar and its long-term life may be seriously affected if its external surface were damaged. Poole et al (1985) found after "considerable investigation" that the abrasive characteristics of fragmented cast iron were potentially detrimental to the MDPE product pipe. Therefore, the use of a thin walled liner pipe of diameter slightly larger than the product pipe provided the necessary protection.

Although originally intended for grey cast iron pipe, bursting methods have also been used to replace pipes constructed from asbestos cement, unplasticised polyvinyl chloride (uPVC), unreinforced concrete, vitrified clay and pitch fibre.

Medium density polyethylene (MDPE) pipe is usually inserted as the new product pipe but materials such as glass reinforced plastic (GRP), clay and uPVC have also been used. The method was originally developed to provide size for size replacement, but developments have shown that it is possible to expand the existing pipe sufficiently to enable larger pipe to be inserted, a process known as upsizing. Poole et al (1985) claim the cross sectional area may be increased by over 180 percent. This is considered rather ambitious but may be possible in the right ground conditions.

Pipe bursting machines are readily available on a commercial basis and are suitable for replacing DN 100 to 300 diameter pipe although equipment is manufactured to replace pipes of up to 670mm diameter. Drive lengths are generally around 50m but drives of up to 140m have been undertaken. Case histories of typical pipe bursting projects are given by Poole et al (1985), Asquith et al (1989) and Boot et al (1987).

Scott and Huetson (1988) presented details of a market survey during which Water Companies were consulted to determine their interest in pipe bursting techniques. Details of past projects were also obtained and these are presented in Tables 2.1 and 2.2. This work

showed that very few contracts on pressure mains had been completed and that the replacement pipe was exclusively plastic. A summary of the technical details obtained during this survey is presented in Table 2.3.

#### 2.2.5 Swaging and Rolldown

These two techniques involve relining an existing main with polyethylene (PE) pipe. Prior to installation, the diameter of the replacement PE pipe is reduced either mechanically between rollers (Rolldown) or by heating and passing through a die whilst the pipe is under tension (Swaging). The pipe is then pulled into the existing main and after a period of time the PE pipe relaxes and returns to near its original diameter (see Figure 2.8). The dimensions are usually arranged so that the expanding new pipe presses against the existing pipe wall. This allows replacement with very little loss in flow capacity since the replacement pipe final diameter is similar to that of the existing main. The reduction in diameter is therefore dependent upon the thickness of the replacement pipe. If the existing pipe is in reasonable structural condition the loss in flow capacity may be kept to a minimum.

Swaging and rolldown have been particularly popular in the gas industry and it is expected that the techniques will become more widely accepted by the water industry. While other techniques exist, particularly in the case of pipe relining where new techniques are constantly being developed, the above methods represent the major processes of pipe installation using trenchless techniques.

Table 2.1: Pipebursting projects carried out by Severn Trent Water  
( after Scott and Huettson , 1988 )

Project Location	Cannock	Walsall	Cheltenham	Western Blvd.	Leicester	Newark	Duffield	Heanor
Original Diameter (mm)	525	150	225	225	225	375	300	225
New Diameter (mm)	450	225	225	225	375	300	375	315
Length Replaced (m)	140	118	57	75	45	40	63	140
Number of Drives	1	3	2	1	1	1	1	1
Depth of Drive Drives(m)	2 - 5	2.7 - 3.4	3	2	5	3	3 - 5	0.5 - 2.3
COSTS Pipe Bursting Costs (£)		11 000	21 569				30 000	
Other Costs (£)		15 000	18 000					
Pipe Bursting Cost Per Metre (£)		93.22	378.40					
Total Cost Per Metre (£)		220.34	694.19					
Original Pipe Material	Concrete	Clay	Clay	Clay	Brick	Clay	Clay	Clay
New Pipe Material	P.E.	Plastic	HDPE	P.E.	P.E.	P.E.		MDPE

**Table 2.2: Pipebursting projects carried out by Anglian Water**  
( after Scott and Huetson , 1988 )

Project Location	Cringle-ford	Whimpton	Stanion	Blisworth	Linton
Original Diameter (mm)	102 76		150	225	225
New Diameter (mm)	125 90	450	175	250	400
Length Replaced (m)	1150 200	100	750	90	47 & 160
Number of Drives			25	2	2
Depth of Drives (m)					2 - 2.5
COSTS					
Total Cost (£)	60 000	20 000	55 500	9 900	10 500 29 500
Cost Per Metre (m)	44.44	200	74	1100	223 184
Original Material	Cast Iron		Pitch Fibre	SGW Pipe	
New Material	MDPE		HDPE	MDPE SPLK	MDPE

**Table 2.3: Technical Data on pipebursting projects from the market survey**  
( after Scott and Huetson , 1988 )

Wall Thickness	-	Standard Dimension Ratio (SDR) 11 is used almost exclusively for pressure mains. This is rated at 10 Bar nominal maximum working pressure. SDR 17 is used for sewer and drain applications. This is rated at 6 Bar nominal maximum working pressure. There is no increase in diameter made for pipe bursting work. For pressure mains a sacrificial PVC liner is first installed and the PE product pipe is sliplined into this liner on completion of the pipe bursting process.
Diameters	-	For pressure mains 3, 4, 6, and 8 inch diameter cast iron pipes are replaced by 90, 125, 180, and 250 mm diameter polyethylene pipe. Sewer replacement is undertaken up to 400mm using pipe bursting.
Lengths	-	Drives up to 150 m have been undertaken. Typically 100 m represents a good days work.
Drive Rates	-	Machines operate at between 1 and 1.5 m/minute.
Welding Time	-	It takes approximately 20 minutes to weld a PE joint (including cooling time ). 15 to 20 metre lengths are usually welded on site.
Depths	-	Pressure mains are usually at depths of between 1 and 1.5 metres. Sewers are laid deeper in order to maintain gradient.
Laterals / Connections	-	Before the main is burst existing connections are removed by digging down and breaking the connection. On completion of the pipe bursting process connections are installed by electro fusion welding. The distance between laterals depends upon the environment or location. In a terraced street connections will be made at every house giving a lateral every 6 to 8 m. This may prove uneconomical for pipe bursting.
Concrete Surrounds	-	These are a problem with pipe bursting and must be identified and removed prior to commencing the operation.
Proximity of other mains	-	Care must be taken in locating nearby mains (especially gas ). WRe recommendations state a 750 mm minimum clearance for size for size replacement and 1000 mm for upsizing. However, recent work by WRe is understood to have reduced these clearance allowances. This work is due to be published in 1989.

## 2.3 PRESSURE PIPE DESIGN

For design purposes pipes may be classified between the two extremes of rigid or flexible. Rigid pipes support loads by virtue of the ring bending resistance of the pipe, whereas flexible pipes depend upon the lateral support of the soil to resist vertical loading. A rigid pipe may be defined as one which under its maximum load, does not deform sufficiently to require a significant amount of passive support at the sides from the surrounding soil. A flexible pipe is capable of deforming, from a circular to an oval cross-section, without fracture. Some types of pipe are referred to as "semi-flexible" and these are mainly rigid pipes which are capable of carrying a significant amount of extra load by virtue of their flexibility.

The deformation under load is usually expressed as the percentage of diameter by which the vertical diameter decreases. Flexible pipes are normally capable of sustaining at least 10% deformation without any risk of damage to the material.

A thin-walled steel pipe is a typical example of a flexible pipe whilst concrete and asbestos cement pipes are in general classified as rigid. Large diameter ductile iron pipes are examples of semi-flexible pipes.

The size of a pipe is identified by quoting the nominal diameter DN. This is a number used largely for convenience in standards and is only approximately related to manufacturing dimensions. In some kinds of pipes such as thermoplastics, the outside diameter is the controlling dimension and the DN corresponds to the outside pipe diameter. In other kinds of pipe such as ductile iron, the inside diameter of the pipe is the controlling dimension and the DN corresponds to the inside diameter or bore of the pipe. In all cases the DN is quoted in millimetres.

The UK Water Industry currently uses eight pressure pipe materials for the construction and maintenance of the water supply system. These are:

- asbestos cement (AC)
- copper
- ductile iron (DI)
- glass fibre reinforced plastic (GRP or RPM)
- medium density polyethylene (MDPE)
- prestressed concrete (PSC)

- steel
- unplasticised polyvinyl chloride (uPVC)

Each of these materials is used for one or more categories of pipe laying (ie trunk, distribution, services) and the useage of materials within these categories is not well defined.

Nevertheless they tend to be used as indicated below in Table 2.4

Table 2.4 Broad Classification of water supply pipe materials according to application (after De Rosa et al, 1988).

NOMINAL BORE (mm)	TRUNK $\varnothing > 300$	DISTRIBUTION $50 < \varnothing < 300$	SERVICE $\varnothing < 50$
Asbestos Cement	•	•	
Copper			•
Ductile Iron	•	•	
Glass fibre reinforced plastic	•		
Medium density polyethylene	•	•	•
Prestressed concrete	•		
Steel	•		
Unplasticised polyvinyl chloride	•	•	

In the UK, the use of trenchless methods for laying pressure pipe has been extremely limited and only five of the eight materials have been used, these being:

- Ductile iron
- Glass fibre reinforced plastic
- Medium density polyethylene
- Steel
- Unplasticised polyvinyl chloride

The characteristics of these materials are discussed below whilst further details of the design requirements of these pipes for use with trenchless methods is discussed in section 2.4.

### 2.3.1 Ductile Iron Pipes

Iron pipes first came into general use in the first half of the 19th century when they were manufactured by a simple sand casting technique. In the 1920s centrifugal casting was introduced which involves casting the molten iron into a horizontal water cooled mould spinning about its longitudinal axis, thereby driving the air out of the metal. In 1948, spheroidal graphite (ductile) iron was introduced and this material replaced the traditional grey cast iron in pipe manufacture during the 1960s.

In ordinary grey cast iron, graphite is present as flakes which tend to have sharp edged rims. Since these flakes have negligible strength they act as wide faced discontinuities in the structure whilst the sharp edged rims introduce regions of stress concentrations. In spheroidal graphite or ductile iron the graphite flakes are replaced by spherical particles of graphite so the metallic matrix is much less broken up and the sharp stress raisers are eliminated. The formation of spheroidal graphite is achieved by adding small amounts of magnesium to the molten iron just before casting. The quantity of magnesium is usually that required to give a residual magnesium content of 0.1%.

Centrifugally cast pipes are subsequently heat treated (annealed) to eliminate the brittle matrix structures which are developed during solidification due to the rapid cooling rates. The resultant mechanical properties of the ductile iron used in pipe manufacture are high tensile strength, ductility and impact resistance. Table 2.5 gives typical properties of a fully annealed ductile iron used in pipe manufacture.

Table 2.5 Mechanical Properties of Ductile Iron

NOMINAL SIZE DN (mm)	TYPE OF CASTING	TENSILE STRENGTH (N/mm <sup>2</sup> )	0.2% PROOF STRESS (N/mm <sup>2</sup> )	ELONGATION MIN %
80 - 1000	CENTRIFUGAL CASTING	420	300	10
> 1000	CENTRIFUGAL CASTING	420	300	7
ALL SIZES	SAND CASTING	420	300	5

Figure 2.9 shows a typical stress-strain curve obtained from a tensile specimen from the wall of a ductile iron pipe. This shows that the metal is an elastic material because the stress-strain relationship is linear over a portion of the ultimate strength range. In the linear range the modulus of elasticity is approximately  $169 \text{ GN/m}^2$  and this figure is used in design calculations. Beyond the linear range the metal exhibits substantial plastic flow or ductility before ultimate failure.

Pipe joints may be classed as either rigid or flexible. Rigid joints are those that do not allow angular or axial movement between adjacent pipes and typical examples include flanged joints or the traditional socket and spigot joints where lead was used as the seal. Flexible joints allow the adjacent pipes to move with respect to one another. This can allow either axial deflection, angular deflection or both depending upon design. Flexible pipes will therefore allow movement following settlement and subsidence without inducing high stresses in pipes and joints. However, many flexible joint designs are unrestrained and support must be provided at intervals along the pipeline in order to ensure that the joints do not blow apart when the test pressure is applied. Typical examples of the types of ductile iron pipe joints which are commercially available are given in Figure 2.10.

The sealing of modern joints is usually achieved by compression of a rubber sealing ring either radially or axially. The radial compression of the ring induces circumferential tensile stress in the pipe socket or collar, the intensity of which depends upon the hardness of the rubber and the degree of compression imposed on the ring. This stress is in addition to the stresses imposed by external and internal pressures when the pipe is in service. In practice the level of compression in the rubber is dependent on the manufacturing tolerances of the pipe and varies between controlled lower and upper limits. The lower limit is the minimum required to achieve a seal whilst the upper limit is typically the maximum which the rubber can endure without excessive risk of premature ageing.

As detailed in Figure 2.10, most flexible joints allow 4 to 5 degrees of deflection depending on size whilst joints which offer axial restraint generally allow 2 to 4 degrees of deflection.

Ductile iron pipe is available in sizes ranging from 80mm to 1600mm nominal diameter (DN). Flexibly jointed ductile iron pipe is available in standard lengths of 5.5m up to and including DN 800 and 8m lengths are supplied in the size range DN 900 to DN 1600.

The maximum hydraulic pressure rating of standard thickness ductile iron pipe is generally far in excess of normal water supply requirements and ranges from 25 bar for DN 1600 pipe to 60 bar for DN 100 pipe. Using standard components the maximum system working pressure is generally governed by the ratings of branched fittings or flanges. This normally limits the working pressure to 16 bar (PN 16) although PN 25 and PN 40 rated fittings are available depending on the nominal diameter.

### 2.3.2 Glass Reinforced Plastic Pipes

Glass reinforced plastic (GRP) or reinforced plastic matrix (RPM) pipes are composed of three basic constituents; glass fibre rovings, polyester resin and sand. The glass fibre rovings are made from high grade borosilicate E glass which possesses high strength and chemical resistance. The standard resin is an isophthalic polyester which forms a matrix for the glass fibres and renders the structure impermeable to fluids. The wall thickness required to provide adequate strength in service would be too thin to withstand the rigours of handling transportation and installation without some form of additional protection. Consequently, graded inert sand is added to the wall of low pressure pipe to increase the thickness and so increase the stiffness.

GRP pipes are produced by two main methods, a centrifugal process or by filament winding. In the centrifugal process, the glass reinforcement, resin and sand are introduced into a rotating mould and are compacted by centrifugal action. By this process, the resin completely penetrates the glass reinforced rovings and the resultant pipe wall is free of excess air and other vapours (for example, styrene) which leads to consistent well engineered material. The filament wound GRP pipes are produced by winding glass roving previously dipped into resin around a mandrel turning at low speed. The very nature of the filament winding process limits the possibility of consistently obtaining a well engineered surface which is free of voids.

Six pressure classes of GRP pipe are produced, these being 6, 10, 12.5, 16, 20 and 24 bar, although the actual ranges available depend on the type and size of pipe. There is very little guidance currently available on the surge and internal pressure fatigue limitations of GRP pressure pipe in UK standards and codes of practice. Greatorex (1984) showed that the initial burst pressure reduces with time and the strength regression is logarithmic with time. The paper concludes that current British standards allow GRP pipes to be designed to very low long term safety factors. Schlehafer and Carlstrom (1985) describe long term testing of GRP pipes and highlight some of the important modes of failure.

The mechanical properties of GRP deteriorate with time, so in order to ensure long service life, high initial safety factors are required. Hoop strength is required to resist the internal pressure and this is provided predominantly by the glass. GRP pipes are classed as flexible conduits since they gain most of their load carrying capability from the passive resistance to the surrounding ground. In a typical installation only 2 - 20% of the total support is provided by the pipe compared with 40 - 98% in the case of ductile iron. The flexural strength of GRP pipes can be improved by positioning the hoop glass near the inner and outer surfaces of the wall thickness.

GRP pipes generally have a socket and spigot type joint similar to that shown in Figure 2.11. The socket is integrally wound with the pipe barrel whilst the spigot is manufactured from polyester resin and is formed by the plane end of the pipe. The gasket is located in a groove and can be moulded in either natural or synthetic rubber. The differential stiffness between the joint spigot and the socket and the inherent flexibility of GRP pipes is such that, unless made to very close tolerances, joints are prone to leakage.

GRP pipe is available in standard diameters ranging from 300mm to 2500mm; although smaller sizes down to 200mm nominal diameter can also be obtained from some manufacturers. All sizes manufactured in the UK are supplied in 6m lengths, although for convenience DN 2500 pipes are usually supplied in 3m lengths.

### 2.3.3 Steel Pipes

Steel pipes first came into use in the second half of the nineteenth century and were either riveted, solid drawn or less commonly welded. Modern steel pipes are manufactured using two main processes and are consequently categorised as either welded or seamless. Welded pipe is normally produced by passing a strip of steel through a series of rollers which form the tubular shape. The longitudinal joint is then welded by either electric arc welding, submerged arc welding or pressure welding. In an alternative process, continuous steel strip is passed through angled rollers which helically wind the strip so that the edges abut along a spiral line. The pipe is then welded by an automatic submerged arc process. Seamless pipe is produced by heating an ingot to forging temperature and shaping it into a cylinder (bloom) using an hydraulic piercer. Depending upon the process employed the bloom is then forged using rollers or dies to either increase or decrease the diameter and increase the length of the bloom until the diameter and thickness equal the desired values for the finished pipe.

The main advantage of steel pipes is that they can be welded together to form an inherently leak free and end load resistant pipeline. Where required, standard couplings can be used to joint steel pipes and the typical types of joints used are shown in Figure 2.12

Steel pipes are characteristically strong and tough with typical tensile strengths of  $450 \text{ N/mm}^2$ . Structurally steel pipes are designed using flexible pipe design principles and so care must be taken to ensure that the surrounding soil provides adequate support. As these pipes are susceptible to corrosion, adequate protection must be provided. In aggressive soil conditions such protection systems often include cathodic protection using the impressed current technique.

The British Standard for steel pipe for water supply applications (BS534) specifies nominal diameters from 60mm to 2200mm, although larger sizes up to 3000mm are available in the UK. Steel pressure pipes for conveying water are usually designed individually in terms of their pressure rating. Consequently, there are no standard pressure classes for steel pipes. BS534 does however suggest minimum pipe wall thicknesses and when used in conjunction with grade 430 steel tube, give the lower round pressure ratings shown in Table 2.6 (after De Rosa et al, 1988). Steel pipes can, however, be designed to the pressure required depending on the grade of steel and pipe wall thickness.

Table 2.6: Lower bound pressure ratings of steel pipe to BS 534 (after De Rosa et al, 1988) assuming grade 430 steel.

PIPE OUTSIDE DIAMETER mm	MINIMUM WALL THICKNESS mm	LOWER BOUND PRESSURE RATING BAR
323.9	4.0	34.0
610	6.3	28.4
914	7.1	21.4
1220	8.0	18.1
1820	11.0	16.6
2220	14.2	17.6

For trenchless techniques steel pipes with flush welded joints has long been the basic casing on many projects. For two stage microtunnelling work the initial temporary casing is often made of steel with bolted internal joints. For auger boring and impact ramming, steel is the most commonly used material with the product pipe sliplined inside the steel liner.

#### 2.3.4 Unplasticised Polyvinyl Chloride Pipes

Unplasticised polyvinyl chloride (uPVC) first came into widespread use in the 1960s. uPVC pipes appeared to offer significant technical and operational advantages over traditional materials. However, the performance of these pipes in terms of impact loading, point loading, fatigue and long-term integrity was not fully evaluated and this led to a considerable loss of confidence in the material. These problems were compounded by the variable performance of field-made solvent welded joints initially used for uPVC pipes. The benefit of these past experiences has now led to greater care in handling and design of these pipes and the number of failures continues to fall.

uPVC pipes are produced by initially forming a PVC polymer resin which is treated with various stabilisers, lubricants, fillers and pigments to form a dry compound. This is then extruded through a die, calibrated in a sizing sleeve then cooled and drawn away from the extruder at constant speed. The pipes are cut to length and normally one end is post formed to produce an integral socket.

The development of design procedures for uPVC pipes has been limited although WRC (De Rosa et al 1988) has published guidance on the structural design of non-pressure PVC pipelines. This is based on providing adequate stability against buckling failure and on limiting the pipe deflection to 6%. For pressure pipelines this approach may be conservative and the preferred approach is to analyse the combined stress in the pipe wall due to internal pressure and external loading. The failure stress of uPVC is approximately 26 N/mm<sup>2</sup> and a safety factor of 1.5 is usually applied to this value (De Rosa et al 1988).

The British Standard for uPVC pressure pipes covers nominal sizes from DN 10 to 600. In practice pipe sizes DN 80 to 300 are used for pressure pipelines. Three pressure ratings are available, 9 bar up to DN 600, 12 bar up to DN 450 and 15 bar up to DN 400. uPVC pipes may only be used where pressure surge occurs (ie in pumped mains) up to DN 300 provided that the maximum pressure including surge does not exceed the pipe rating and provided that the amplitude of pressure fluctuation does not exceed half the pressure rating.

On-site solvent welded jointing is no longer recommended for uPVC pressure pipe due to the problems encountered when this material was first introduced. The push-fit type joint with elastomeric jointing ring, as detailed in Figure 2.13, is now commonly used for uPVC pipes. These joints are easy to assemble on site and require no special jointing equipment, unlike the welding process. The maximum angular deflection of push fit joints is limited to one degree in order to minimise the risk of point loading of the inside of the socket by the spigot. Any directional changes are accommodated using standard bend fittings. There are a range of couplings available which enable uPVC pipes to be jointed to standard fittings and valves, etc.

### 2.3.5 Medium Density Polyethylene (MDPE) Pipes

Low and high density polyethylene water pipes were first introduced during the 1950's, principally for service connections below DN 50. More recently medium density polyethylene (MDPE) has been developed for the water industry following its introduction by the UK gas industry. Consequently, MDPE pipes are now used for both distribution and trunk mains. MDPE offers potential advantages over traditional mains pipe materials in terms of it being lightweight, flexible, weldable and corrosion resistant. There has been extensive research into the performance of MDPE pipes, principally aimed at establishing its fast fracture behaviour and its performance under external and internal pressure fatigue loading. Long-term research has also been established to monitor any changes in performance which may occur over extended periods of time.

MDPE pipes are produced from granules which comprise the base polymer with additives including antioxidants, pigments and ultra violet stabilisers. These granules are heated to produce a homogeneous melt and this is extruded under pressure through a die to give the required diameter and wall thickness.

MDPE pipes are flexible conduits and can be used to great cost advantage when the structural design takes proper account of pipe performance. The maximum design deflection prior to initial pressurisation should not exceed 6% and the combined stress in the pipe wall due to internal pressure and external loading should not exceed  $6 \text{ N/mm}^2$ . This design stress value is obtained by applying a factor of safety of 1.3 to the minimum 50 year failure stress of  $8.3 \text{ N/mm}^2$  for MDPE pipe. The resistance of the pipe to buckling should also be carefully evaluated.

Unlike most pressure pipe materials, MDPE pipe nominal diameters refer to the outside diameter of the pipe. The pressure ratings of these pipes depends on the thickness of the pipe wall, which is classified by the ratio of outside diameter of the pipe to its wall thickness and is known as the Standard Dimension Ratio (SDR). In the nominal diameter range DN 90 to 630 there are two pressure classes: 10 bar (SDR 11) and 6 bar (SDR 17.6). However, SDR 17.6 pipe is normally only recommended for pipes of nominal diameter greater than DN 180 where the design calculations indicate that it is suitable for sliplining applications. SDR 17.6 pipes are also available up to nominal diameter DN 1000 at 6 bar pressure rating. However, in these larger sizes, the actual pipe pressure rating will depend on the application and corresponding additional design criteria and test data.

The performance of MDPE pipelines under surge conditions is currently under research by the WRC but until this work is complete, similar design criteria to those used in uPVC pipes is employed. The maximum surge pressure should not exceed the maximum allowable working pressure of the pipe and the amplitude of pressure fluctuation should not exceed one half of the pipe's maximum allowable working pressure.

MDPE pipes are available in either coils or in straight lengths depending on pipe size. There are five basic methods for jointing MDPE pipes: butt fusion welding, socket fusion welding, electrofusion welding, push-fit socket and spigot joints and mechanical couplings. These methods are illustrated in Figure 2.14.

MDPE pipes have been used extensively with trenchless techniques. In particular, impact moling and pipe bursting systems have been adapted to suit MDPE pipes whilst swaging and rolldown were developed specifically for this material.

## 2.4 PIPE DESIGN FOR TRENCHLESS METHODS

The design of pressure pipe used in conjunction with trenchless pipelaying methods must take account of the general requirements for water pipelines and in addition, must fulfill further requirements to suit the installation method employed. The general requirements for water pipelines are discussed in section 2.3 and by De Rosa et al, 1988. The additional requirements for trenchless techniques may include the following:

- a) ability to transmit and withstand tensile or compressive axial loads developed during installation,
- b) sufficient structural strength in unprepared ground conditions,
- c) ability to resist abrasive action from the soil during installation,
- d) joints must be capable of specified deflections under specified axial loads,
- e) joints must not allow soil infiltration into the pipe during installation,
- f) the pipe profile should be designed to minimise jacking or pulling loads as required, and
- g) where individual pipes are to be jointed on site the length of each pipe should suit the installation method.

Hence the requirements of trenchless pipes are not only dependent upon the pipe material but also on the method of trenchless installation employed. As a result, it is not normally possible to design a universal trenchless pipe due to the differing characteristics of each pipe material and of each trenchless method. In fact trenchless methods have been developed in conjunction with a particular pipe material. For example, there is extensive use of concrete pipe with microtunnelling whilst impact ramming has only employed steel pipe and impact moling and pipe bursting are largely associated with uPVC and MDPE pipe.

In this section pipe products which have been developed specifically for use with trenchless techniques are presented. In some cases, trenchless techniques have been adapted to suit standard pipe products and, since no pipe design was involved, these are not discussed below. The reader is therefore referred to section 2.3 where the standard pipe is discussed.

### 2.4.1 Ductile Iron Jacking Pipes

The Japanese company Kubota was one of the first manufacturers to develop ductile iron pipes for use with microtunnelling and pipe bursting machines. As shown in Figure 2.15 there are two designs, the UD and the UD-F, which are available in nominal diameters DN 700 to 2600. The UD design incorporates a flange welded to the spigot and the joint is anchored using a stud and nut arrangement screwed through this flange. Thus the flange provides the method for transferring the jacking load. The UD-F design is anchored using a locked ring located in a recess in the spigot and this ring is located by a set pin screwed through the socket. This locked ring arrangement provides the means for transferring the jacking force.

Kubota jacking pipes are available in 4m lengths although the company will supply 6m pipe lengths for DN 700 - 1500 as required. The concrete coating provides uniform outside diameter and this also acts as a resistance to abrasion and corrosion. The concrete contributes little to withstanding axial thrust.

The French company Pont a Mousson has also produced a prototype jacking pipe to the design shown in Figure 2.16. In this design the jacking thrust is transmitted through a reinforced concrete coating and the maximum jacking load is dependent upon the shear strength of the concrete to iron interface. The pipe ends are provided with reinforced bars of increased diameter and wooden packing pieces are used to distribute the load when the pipe deflects.

The concrete coating and reinforcement arrangement is extremely substantial with coating thickness quoted at 100mm for DN 400 to 800 and 70mm for DN 800 to 1600. Such thicknesses greatly increase the weight of pipe and make site handling difficult.

German manufacturer Meyer has produced a third jacking pipe design as shown in Figure 2.17. In this design the spigot abuts directly onto the heel of the socket and although this utilises the inherent strength of the ductile iron, the bearing area is small and large stresses are expected.

The UK manufacturer Stanton plc has produced various designs of jacking pipe which have been extensively tested and a typical design is shown in Figure 2.18. The aim of this development work was to produce a DN 1000 jacking pipe with a thrust capacity of 6MN whilst the adjoining pipes were misaligned by 0.5 degrees. This work was reported by Stevens

(1989) and the main conclusion stated that the pipe met the specified design criteria but that further development work was required, including investigation of stress concentrations at the socket to barrel junction; buckling of pipes; strength of welds in compressive shear; the influence of misalignment and the influence of reinforcement in the concrete coating.

#### 2.4.2 Glass Reinforced Plastic Jacking Pipes

For jacking purposes a recessed collar type joint, similar to that shown in Figure 2.19, has been successfully used. These pipes were manufactured by centrifugal casting and a sandwich construction was incorporated into the wall thickness with the glass fibres concentrated at the external and internal diameters and with a standard mix of fibre, filler and polyester resin in between. The joint is formed by a steel collar which is glued into a recess cast onto one end of the pipe. Sealing is achieved using rubber sealing rings which are compressed between the GRP pipe and the steel collar.

These pipes are available with either 40, 50 or 60mm wall thickness depending upon expected jacking load. Such wall thicknesses are in excess of standard GRP pipe thicknesses in order to ensure that the pipe wall stress is within required limits. A semi-compliant wooden packing piece is used between each pipe to distribute the jacking loads when the joint is deflected. The jacking load capacity of these pipes is shown graphically in Figure 2.20.

#### 2.4.3 Other Jacking Pipe Designs

Trenchless techniques which use steel, uPVC and MDPE pipes have been developed specifically for use with these pipe materials and as a result use the standard pipes normally available (see sections 2.3.3, 2.3.4 and 2.3.5). There has been development work undertaken for clay and concrete sewer pipes used for microtunnelling purposes and some designs are shown in Figure 2.21. Of the variety of designs available there are basically two generic types, (a) and (b), the rebated in-wall type and the butt joint collar type. The collar joint is generally preferred because for a given wall thickness the pipe contact area and the load carrying capability is greater than the in-wall type joint.

## 2.5 PIPE TO SOIL INTERACTION

### 2.5.1 Pipe Jacking Forces

Despite numerous attempts to predict jacking forces, the provision of sufficient jacking capacity continues to be based on previous experience from schemes in similar ground conditions.

The total jacking force comprises two components, the force required to push the shield head into the excavation, referred to as the face resistance, and the frictional resistance along the pipe length. Consequently, the factors influencing jacking load include:

- a) length and outside diameter of jacked line
- b) weight of pipe
- c) height of overburden
- d) nature of ground
- e) face resistance
- f) continuity of operations
- g) lubrication
- h) degree of overcut
- i) joint deflection

The degree of influence of each of these parameters is highly variable and inter-dependent. However, in all cases the continuity of operations is of prime importance since jacking forces are reported to increase quickly following stoppages. This is shown graphically by Norris and Milligan (1992) in Figure 2.22. The pipe jacking resistance per unit area of external surface ranges from 2 to over 40 kN/m<sup>2</sup> and typical values for various ground conditions were quoted by the Concrete Pipe Association of Australia, Table 2.7

Table 2.7 Jacking Resistance for various ground conditions (Concrete Pipe Association of Australia)

TYPE OF GROUND	JACKING RESISTANCE kN/m <sup>2</sup>
ROCK	2 - 3
BOULDER CLAY	5 - 18
FIRM CLAY	5 - 20
WET CLAY	10 - 15
SILT	5 - 20
DRY LOOSE SAND	25 - 45
FILL	< 45

#### 2.5.1.1 Empirical Methods of Jacking Load Estimation

The methods used to estimate jacking loads are based on calculating the frictional resistance between the soil and the outside diameter of the pipe using factors based on past experience. The total jacking load ( $F_T$ ) is generally the sum of the face resistance and the frictional sliding resistance. Typical formulae include those quoted by Craig and Kirkland (1982) from Japan and Australia.

## JAPAN

$$F_T = (P_F + P_W) \cdot \frac{\pi D^2}{4} + (R.S + w.f.) \cdot L \cdot \lambda \quad \text{.....(1)}$$

**where:**

$F_T$	=	total pipe jacking force (kN)
$P_F$	=	average cutting pressure (kN/m <sup>2</sup> )
$P_w$	=	mud water pressure for slurry machine (kN/m <sup>2</sup> )
$D$	=	external diameter of pipe (m)
$R$	=	frictional resistance between soil and pipe
$S$	=	circumference of pipe (m)
$w$	=	weight of pipe per unit length (kN/m)
$L$	=	length of drive (m)
$\lambda$	=	coefficient for curved drive
$f$	=	Coefficient of frictional resistance due to the weight of pipe

## AUSTRALIA

$$F_T = F_{A\Pi.D.L.} + F_{D.D.} \dots\dots\dots(2)$$

**where:**

$F_D$  = force per metre diameter for leading edge to penetrate (kN/m)  
 $F_A$  = adhesive force per unit area of soil/pipe contact (kNm<sup>2</sup>)

In both these formulae the unit cutting resistance ( $P_F$  and  $F_D$ ) and the unit sliding resistance ( $R$  and  $F_A$ ) are based on experience.

### 2.5.1.2 Theoretical Methods of Jacking Load Estimation

Theoretical methods for estimating jacking loads use earth pressure calculations together with coefficients of friction. They generally take account of the overburden pressure, the lateral earth pressure and the self weight of the pipes.

#### Earth Pressure Theory

An analysis based on vertical and horizontal earth pressures was used to estimate jacking loads in Japan (Water Services, 1981) and the comparison between theoretical and actual jacking loads is shown in Figure 2.23. The basis for the calculation was the following formula which assumes all of the overlying soil becomes loose:

$$F_T = L \left[ \frac{\pi D}{2} \mu \left[ P_s + \frac{1}{2} (P_1 + P_2) \right] + \frac{1}{4} W \mu \right] \quad \text{.....(3)}$$

where:

$\mu$	=	frictional coefficient between soil and pipe
$P_s$	=	soil pressure (kN/m <sup>2</sup> )
$P_1, P_2$	=	soil pressure horizontal to pipe
$P_1$	=	$K \cdot \alpha \cdot H$
$K$	=	$\frac{1 - \sin \phi}{1 + \sin \phi}$
$H$	=	cover depth (m)
$\phi$	=	internal friction angle of soil
$\alpha$	=	weight per unit volume of soil (kN/m <sup>3</sup> )
$P_2$	=	$K \cdot \alpha \cdot (H + D)$

The coefficient of friction was given as 0.4 in gravel and 0.2 in clay but a value of 0.4 was found to be more appropriate when restarting the drive each day.

Auld (1982) analysed jacking loads using Terzaghi's earth pressure theory to determine the vertical and horizontal stress on the pipe allowing for the effect of arching of the soil above the pipe. The total jacking force was determined by adding components from the top and bottom halves of the pipe.

For the total force on the top half of the pipe per metre run,  $F_u$

$$F_u = D (P_v + P_h) \quad \dots\dots\dots(4)$$

where:

$$\begin{aligned} P_v &= \text{vertical earth pressure on pipe (kN/m}^2\text{)} \\ P_h &= \text{horizontal earth pressure on pipe (kN/m}^2\text{)} \end{aligned}$$

For the total force on the bottom half of the pipe per metre run,  $F_L$

$$F_L = D (P_v + W_s + P_h) \quad \dots\dots\dots(5)$$

where:

$$W_s = \text{weight of pipe per unit area (kN/m}^2\text{)}$$

and the total force on the pipe per metre run is then

$$F_E = F_u + F_L \quad \dots\dots\dots(6)$$

the total jacking force is then

$$F_T = F_E \cdot \tan \delta \cdot L \quad \dots\dots\dots(7)$$

where  $\delta$  is the wall friction. As a maximum Auld stated that the value of  $\delta$  cannot exceed the angle of internal friction  $\phi$  and will generally be slightly less, being approximately  $0.67 \phi$  to  $0.75 \phi$ .

## 2.5.2 Ground Movements during Pipe Installation

Soil displacements associated with trenchless techniques fall into two broad categories as identified by O'Rourke (1985). When the volume of excavated soil exceeds the volume of pipe installed, the surrounding ground will generally displace, or converge, toward the pipe and this type of installation is referred to as convergent. When the volume of the installed pipe or construction equipment exceeds the volume of the excavated soil, the surrounding ground will generally displace or expand from the opening, this type of installation being referred to as expansive.

Convergent installation techniques include pipe jacking, microtunneling and various types of thrust boring. The excavation performed with these methods will result in stress relief and subsequent ground deformation into the excavated cavity. Expansive techniques include impact moling and pipe bursting. These techniques will result in increased soil stress, outward soil displacement and are generally more prone to result in surface heave.

Chapman (1992) provided a thorough discussion of ground movements associated with pipe jacking and pipe bursting. This work gives an extensive description of work published on ground movements. Experimental studies were undertaken in the laboratory to create a model which can predict ground movements. Extensive laboratory results are compared with predicted results using a modified fluid flow analysis and good correlation between the two was obtained. A typical set of results is reproduced in Table 2.8. Other than this work there are limited published data on the ground movements associated with pipe bursting.

Reed (1987) provides some details of work conducted by the Water Research Centre (WRC) to determine the effects of pipe bursting on adjacent pipelines in uniform ground conditions. The work consisted of installing strain gauged ductile iron pipe above and perpendicular to the pipelines which were subsequently replaced by pipebursting. Hence the effect of the moling operation on these adjacent pipes was determined. The strain readings were found to increase as the pipeburster was adjacent to the measuring point, but were found to decrease once the pipeburster had passed. Thus there was very little residual strain at the end of the trial. Displacement transducers were also installed 300, 600 and 900mm above the pipe being replaced in order to record ground movements. The results of this work are compared with the authors own studies in Chapter 4.

Dorling (1984) also working for the Water Research Centre, studied the influence of pipebursting on adjacent services when replacing an existing 229mm diameter sewer with 250mm diameter Polyethylene pipe. A strain gauged DN80 ductile iron pipe was laid parallel to the sewer at a distance of 1.1m. The strain values induced in the ductile iron pipe were well below the maximum allowable levels. Strains in the order of 14 microstrain were recorded against a maximum allowable of 100 microstrain.

Leach and Reed (1989) expand on the work presented by Reed (1987) and ground movements are discussed in some detail. Subsurface ground movements are described in three distinct stages, illustrated in Figure 2.24. The ground is initially forced outward as the burster expands the existing pipeline, this is the expansion stage. There is then a period of

Table 2.8: Comparison between the observed displacements in laboratory pipebursting tests and those predicted using fluid flow analyses (after Chapman (1992))

COORDINATE (from pipe axis) (m)	LABORATORY TEST RESULT (mm)		THEORETICAL ANALYSIS RESULT (mm)	
	V	H	V	H
<b>Centreline values above pipe axis</b>				
0.15	32.00	0.00	32.00	0.00
0.20	25.00	0.00	15.00	0.00
0.25	18.00	0.00	8.50	0.00
0.30	13.00	0.00	5.50	0.00
0.35	9.00	0.00	4.00	0.00
0.40	6.50	0.00	3.10	0.00
0.45	3.50	0.00	2.70	0.00
0.50	2.50	0.00	2.60	0.00
0.1,0.1	23.00	8.00	21.00	12.00
0.2,0.2	6.00	2.00	4.00	3.00
0.3,0.3	1.00	1.00	1.40	1.00
-0.05,0.1	-4.00	5.00	-2.50	3.50
<b>Surface values (from centreline)</b>				
0.0	2.50	0.00	2.60	0.00
0.1	1.75	2.50	2.30	0.00
0.2	1.00	2.50	1.80	0.00
0.3	0.30	1.00	1.20	0.00
0.4	0.00	0.00	0.80	0.00
0.5	0.00	0.00	0.45	0.00
0.6	0.00	0.00	0.30	0.00
0.7	0.00	0.00	0.10	0.00
0.8	0.00	0.00	0.00	0.00

Note: Positive values indicate vertical movements are upwards and horizontal movements are to the right.

recovery during which the ground converges towards the new pipe and this stage is associated with an increase in jacking load. The final stage involves full contact with the new pipe. On the surface a transient heave is associated with the expansion and recovery stages whilst the final soil to pipe contact relates to a permanent surface heave. Surface movements are dependent upon soil conditions, replaced and installed pipe sizes and cover depth.

Perhaps the most useful result of this work is the presentation of a surface damage and safe proximity chart derived using data collected in the field (Figure 2.25). These charts are useful guides for engineers considering the use of pipebursting techniques. Figure 2.26 shows the typical strain gauge readings in relation to the position of the pipeburster. This work is substantiated by Chapman (1992) who presented the general form of ground movements observed during laboratory pipebursting trials, as shown in Figure 2.27. This work describes some of the most detailed studies on pipebursting and pipe jacking ground movements. Figures 2.28 and 2.29 are examples of the displacement contours and displacement vectors obtained during trials. All this work was carried out in dry sand.

Swee and Milligan (1990) presented results of laboratory pipebursting trials using a 55mm diameter model burster in sand, clay and sandy clay backfill. This work essentially studied the ground movements associated with a radial expansion of the soil. No attempt was made to model the convergent stage observed by Leach and Reed (1989) and Chapman (1992). Displacement of the soil was found to be primarily vertically upwards with very little movement below the pipe axis. Surface heave profiles were produced as shown in Figure 2.30. These are compared with the authors results in Chapter 4.

## 2.6 CURRENT AND FUTURE WATER AND GAS INDUSTRY REQUIREMENTS

The five main utility suppliers (water, sewerage, gas, electricity and telecommunications) throughout North America and Europe install in the order of 400 000 km of services per annum, in addition to the millions of individual house connections. The water industry accounts for some 16 percent of this total market (Thompson 1987).

In the United Kingdom around 3800 km of water mains are laid per annum forming an investment of some £140 million at 1986/87 prices. In addition approximately 200 000 service connections are made each year at an estimated cost of £20 million. Some 90 percent of the total length of water mains laid each year are less than 300 mm in diameter and as discussed in section 2.3, plastic pipe materials are now dominant in this size range. Pressure pipes are not required to be laid to a set gradient and depth and so water mains are usually buried approximately 1 metre below the surface and are therefore ideally suited to open cut installation methods.

In England and Wales there are some 250 000 km of sewer pipe and approximately 2500 km of new pipe are installed each year; 76% by developers and 24% by water companies. Sewers are required to follow set gradients and so have tight tolerances on line and level. The minimum depth of cover is specified as 900 mm but sewers are often laid well below the surface at depths of 5 to 10 metres in order to avoid other services and also to ensure the correct gradient is maintained. This is considered an advantage for those concerned with trenchless techniques since there is potential for avoiding deep trenching and less risk of causing damage to other services at these increased depths.

British Gas has approximately 250 000 km of buried mains pipeline which serves over 16 million customer connections. 87% of existing mains are in cast iron or steel, the remaining being plastic. However, over 90% of new gas and service connections are now laid in plastic pipe materials and British Gas have developed methods of installing plastic pipes with minimum surface disruption (ie rolldown, pipe bursting etc).

Considering the water supply market alone, there is a total of 350 000 km of water mains in service in the UK and 75% of this system is estimated to be laid in cast iron having either grey or spheroidal graphite metal structure. It is estimated that 45% of the total length was laid before 1945 and much of this was laid before 1940 in unlined grey cast iron. As a result, great

emphasis is now being placed on rehabilitating this ageing network and more than 50% of annual expenditure on water mains is now allocated for this purpose.

These statistics show that there is a need for developing methods of replacing this ageing network using trenchless pipelaying methods in order to ensure minimum disruption to above ground services. Water Companies were approached to determine their interest in ductile iron pipe for replacement of existing grey iron mains by pipe bursting. The response was favourable, with South West Water, Northumbrian Water and Anglian Water expressing great interest in this concept, the only reservation being that of cost compared to alternative materials or installation methods. Studies by Scott and Huetson (1988) showed that the most attractive market for the ductile iron product is in the DN 150 to 400 diameter range since this is best able to compete on cost with plastic pipe materials. Water companies emphasised that such a product would be used in urban areas where integral pipe strength is important. Since the product is likely to be laid in built up areas, the individual pipes must be suitable for installation from a small launch pit of 2 to 3 metres in length.

Ductile iron pipelines must be properly protected against corrosion to ensure long-term product life. Trenchless pipelaying methods are potentially aggressive to pipe coatings and it is essential that corrosion protection systems must remain effective during and subsequent to installation.

Therefore, the water industry requires a high strength pipe which is available in short individual lengths and can be used with current pipe bursting technology and is fully protected against corrosion to ensure long service life.

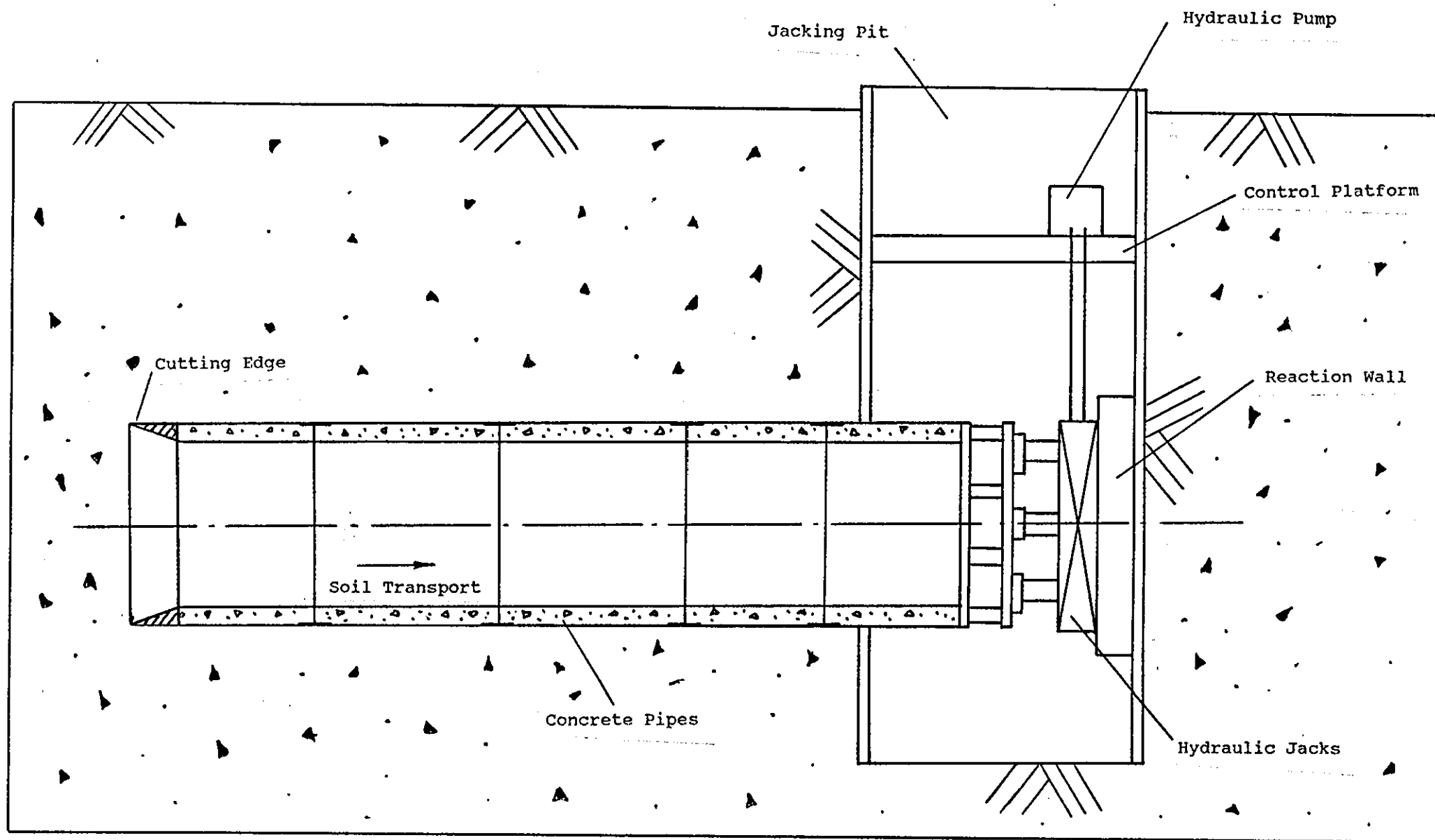


Figure 2.1

Longitudinal Section through a Pipe Jacking Scheme

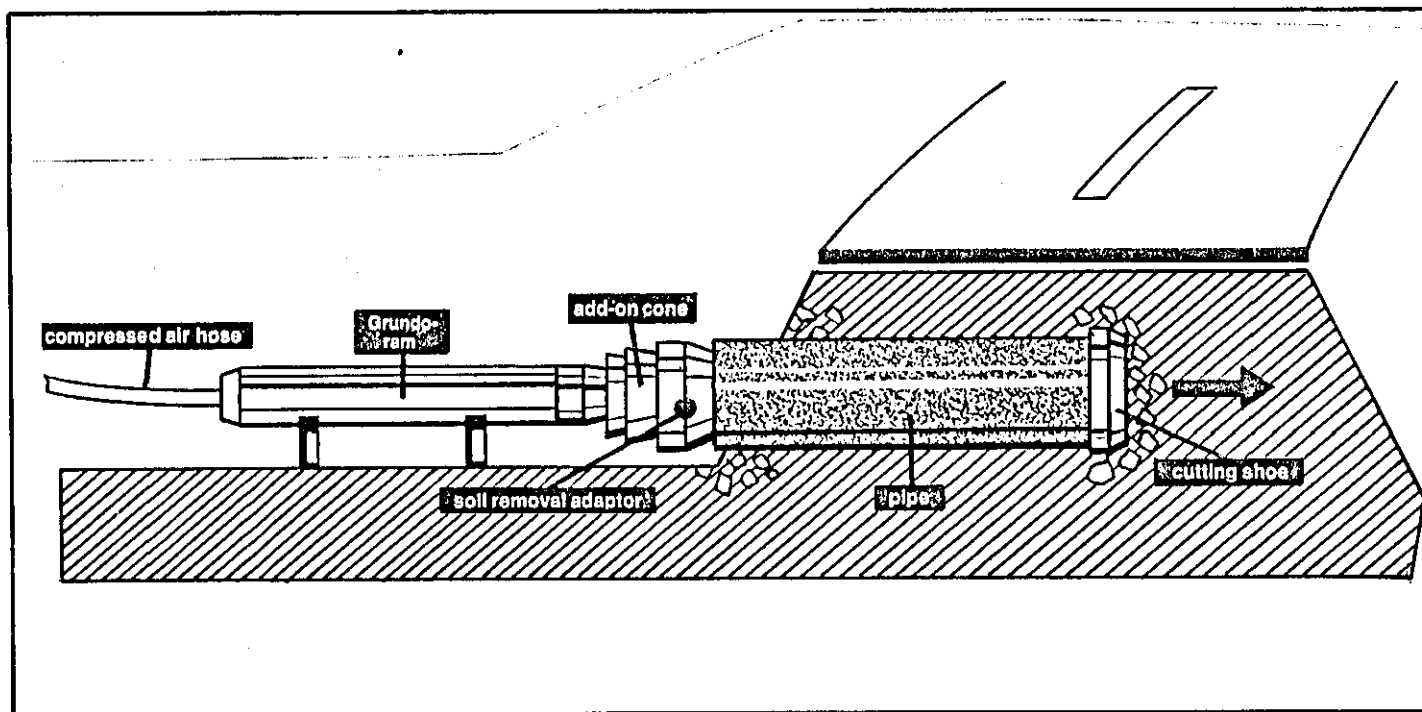
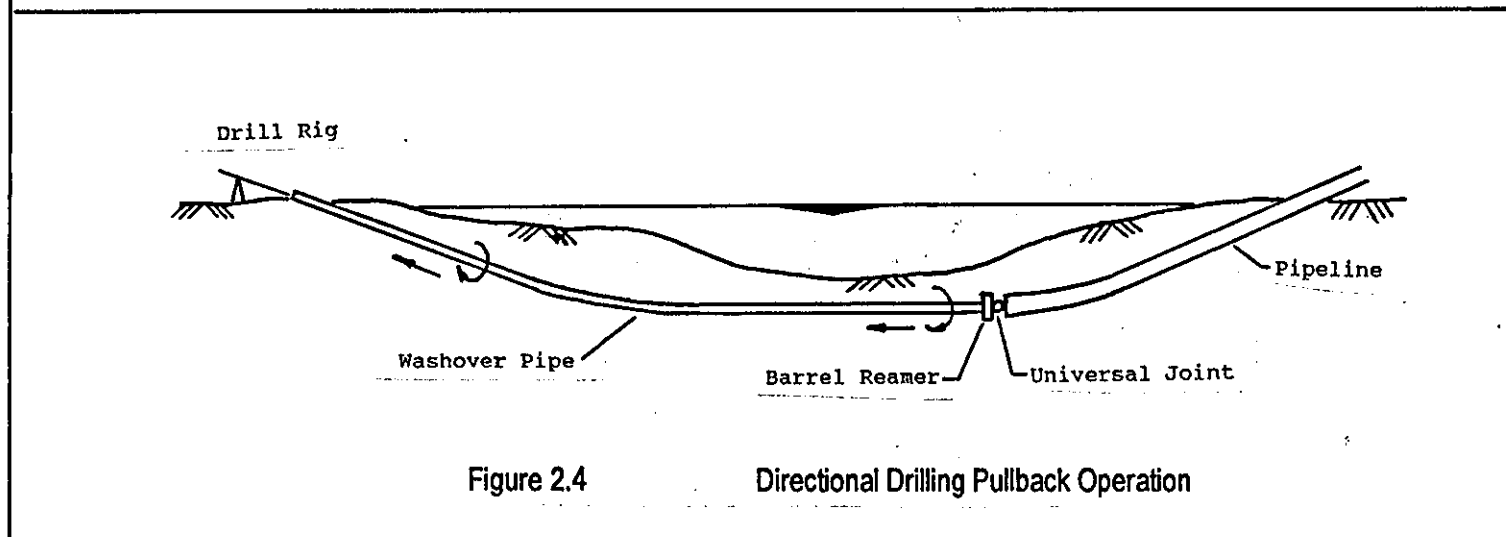
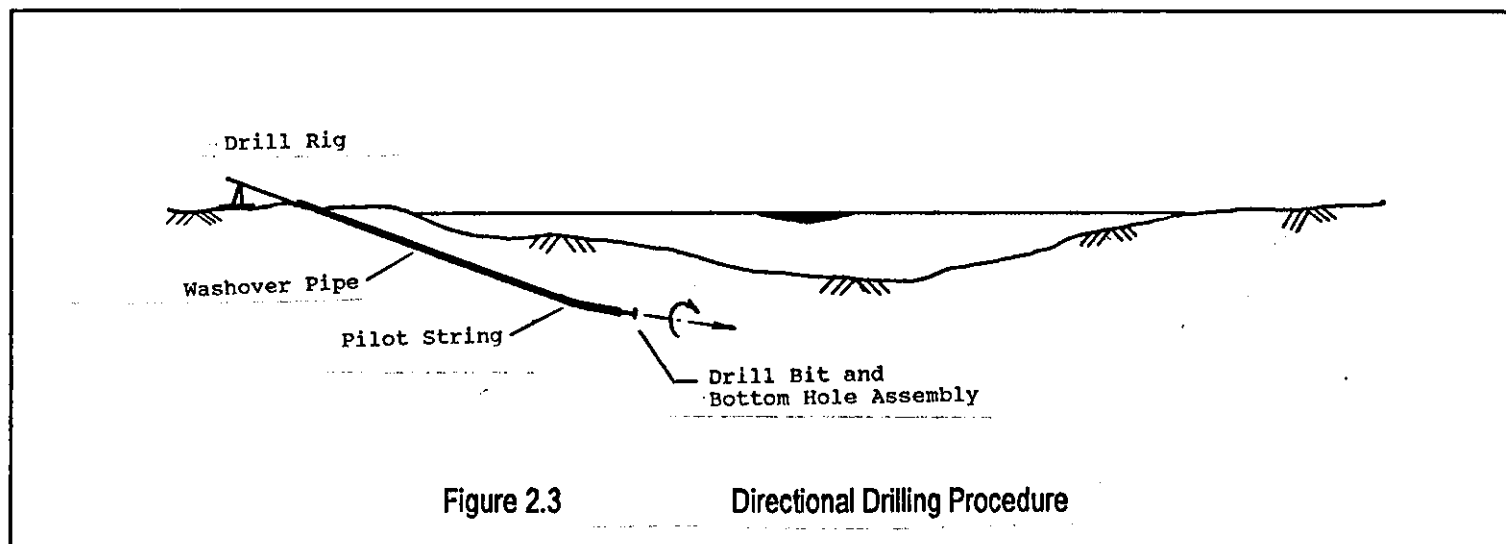
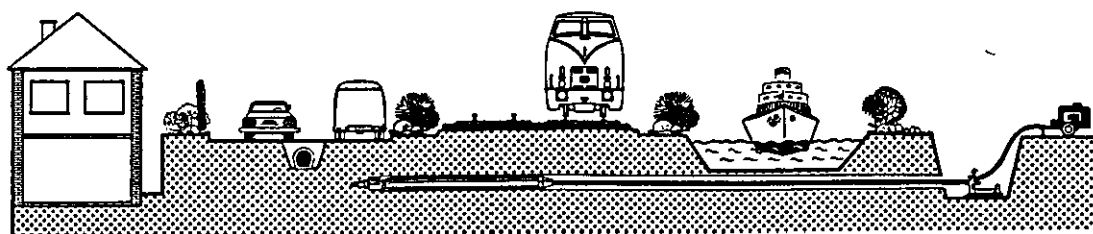


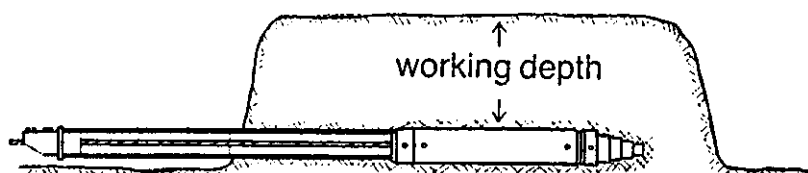
Figure 2.2

The Impact Ramming Method

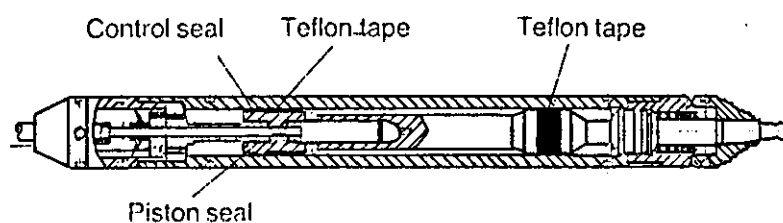




GENERAL LAYOUT



Method for Pulling Pipe  
Directly Behind Mole



Detail of Soil Displacement Hammer

Figure 2.5

Impact Moling

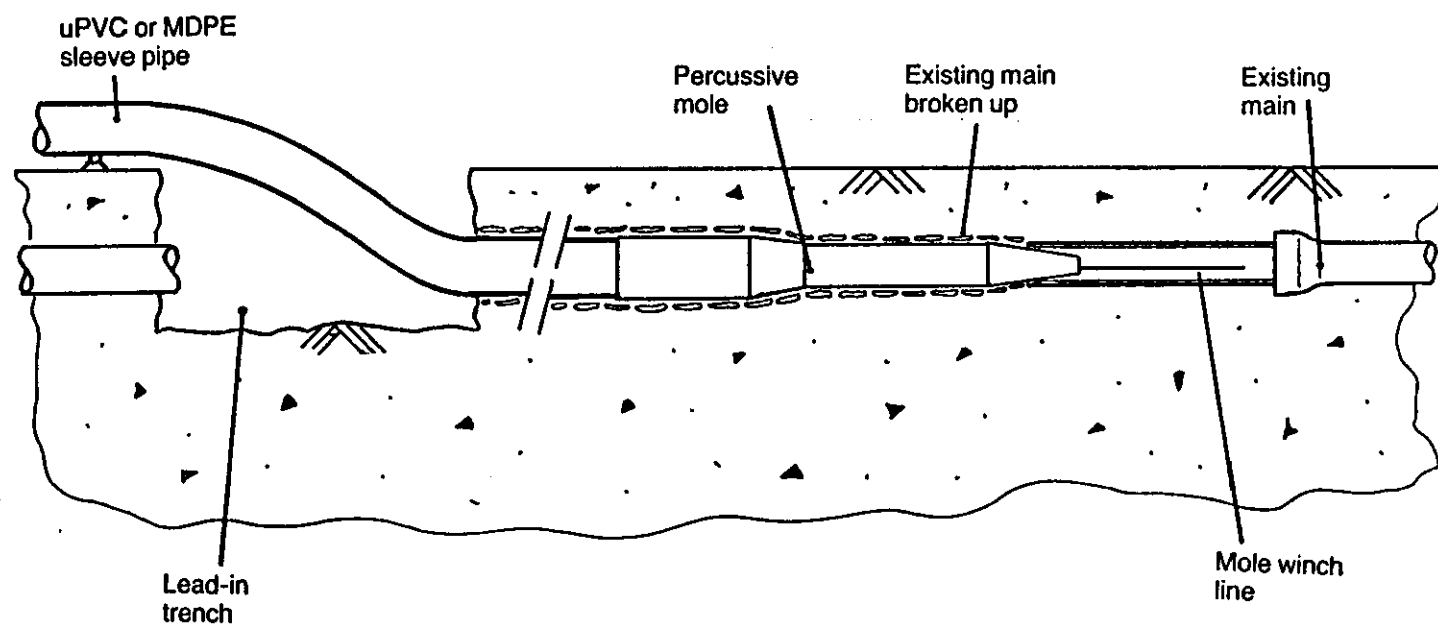
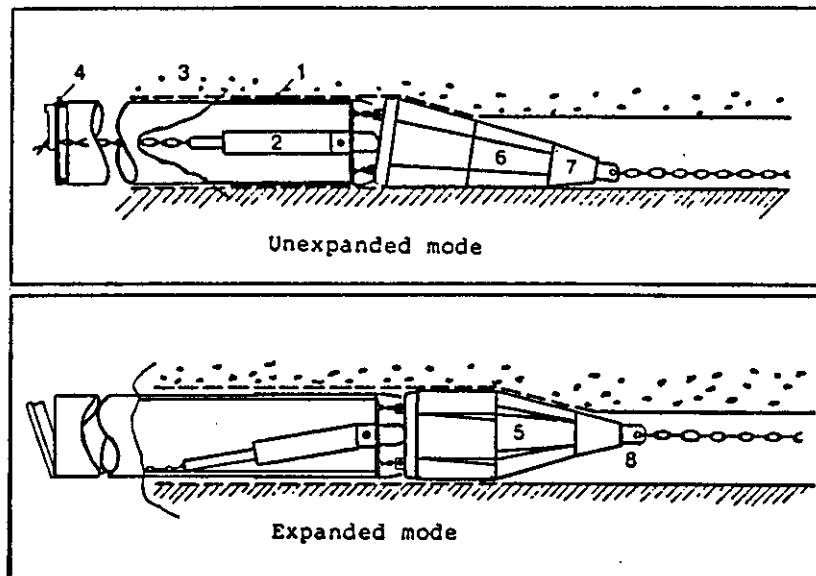


Figure 2.6

Pipe Bursting using a Shield Expander



1. Broken Pipe Fragments
2. Hydraulic Ram - Retains compression in replacement pipe
3. Replacement Pipe
4. End Plate
5. Expanded Mole
6. Contracted Mole
7. Nose Piece
8. Chain to Winch

Figure 2.7

Pipe Bursting using a Hydraulic Expander

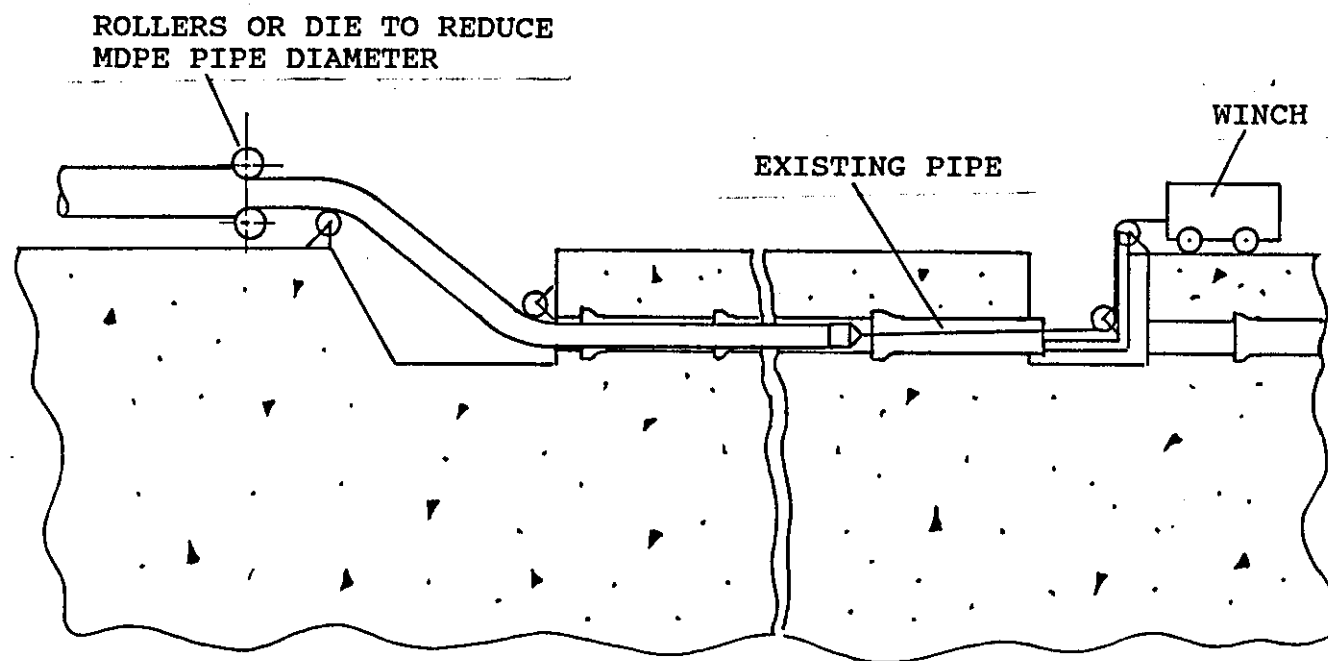


Figure 2.8

Swaging and Rolldown Operations

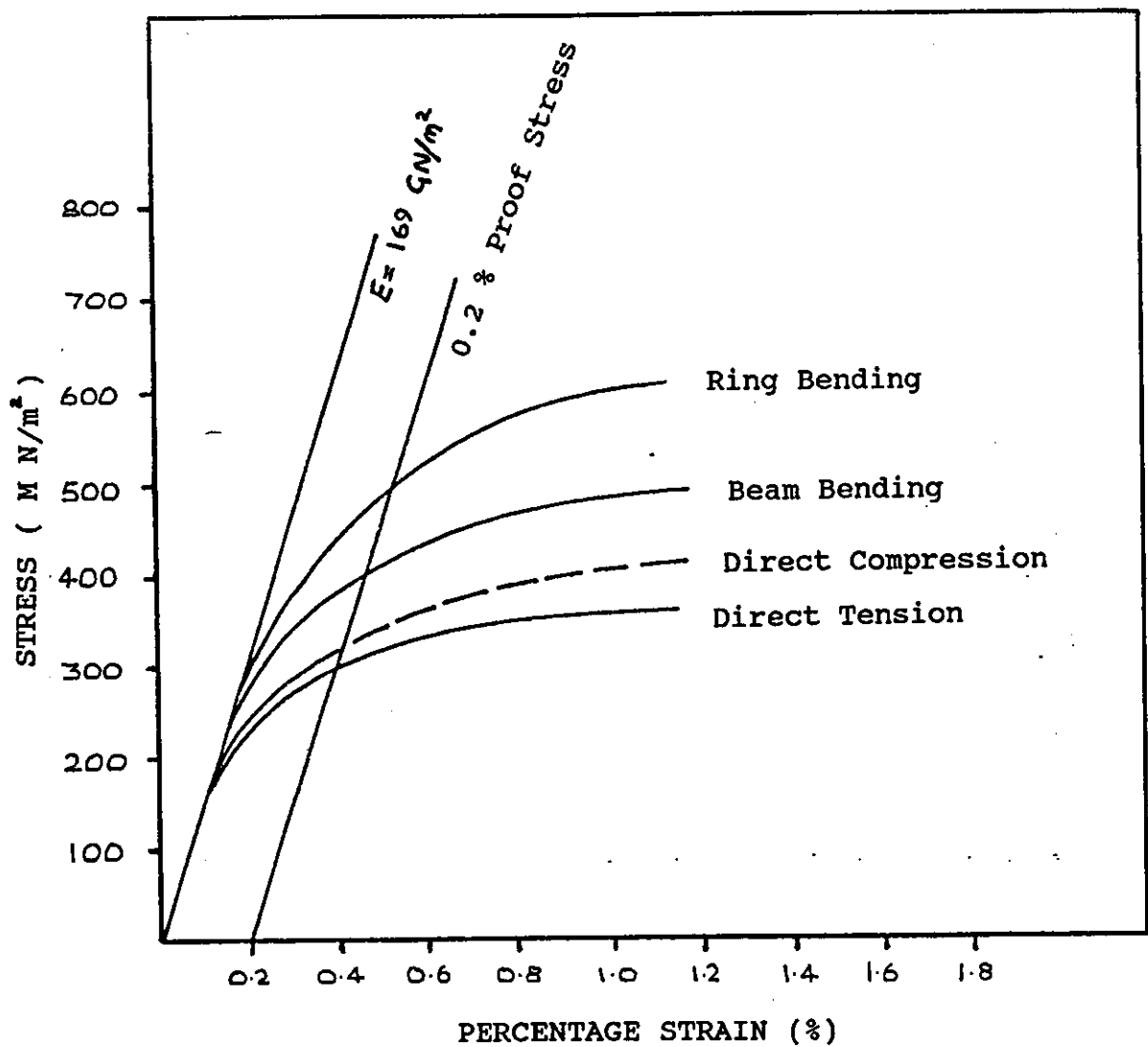


Figure 2.9.

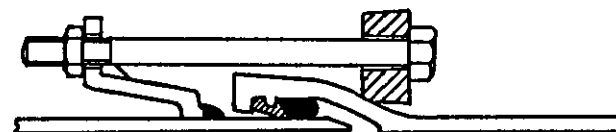
Stress-Strain Curves for Ductile Iron

### TYTON JOINT



DN 80 - 600  
4° - 5° Maximum Deflection

### TYBAR JOINT



DN 80 - 1600  
4° - 5° Maximum Deflection

### STANTYTE JOINT



DN 700 - 1600  
4° Maximum Deflection

### STANLOCK JOINT



DN 100 - 600 (Excl DN 350 )  
4° Maximum Deflection

### TYTON ANCHOR JOINT



DN 80 - 300 and DN 400  
3° Maximum Deflection

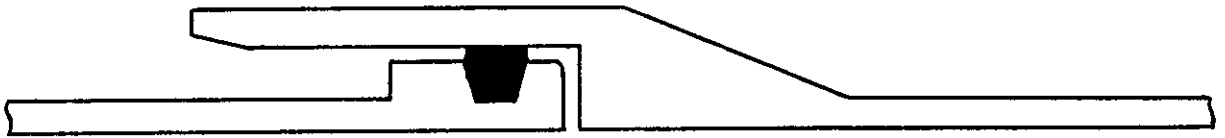
### STANLOCK ANCHOR JOINT



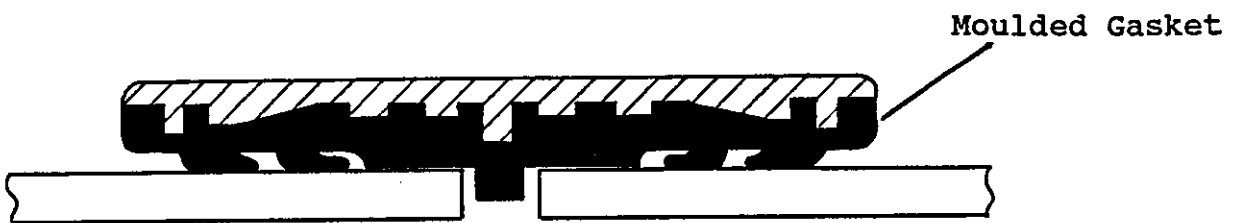
DN 100 - 450 (Excl DN 350 )  
2° Maximum Deflection

Figure 2.10

Ductile Iron Pipe Joints



G.R.P Spigot - Socket Joint



G.R.P Collar Joint

Figure 2.11

GRP Pipe Joints

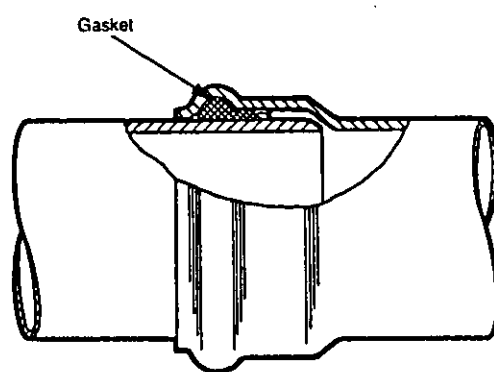
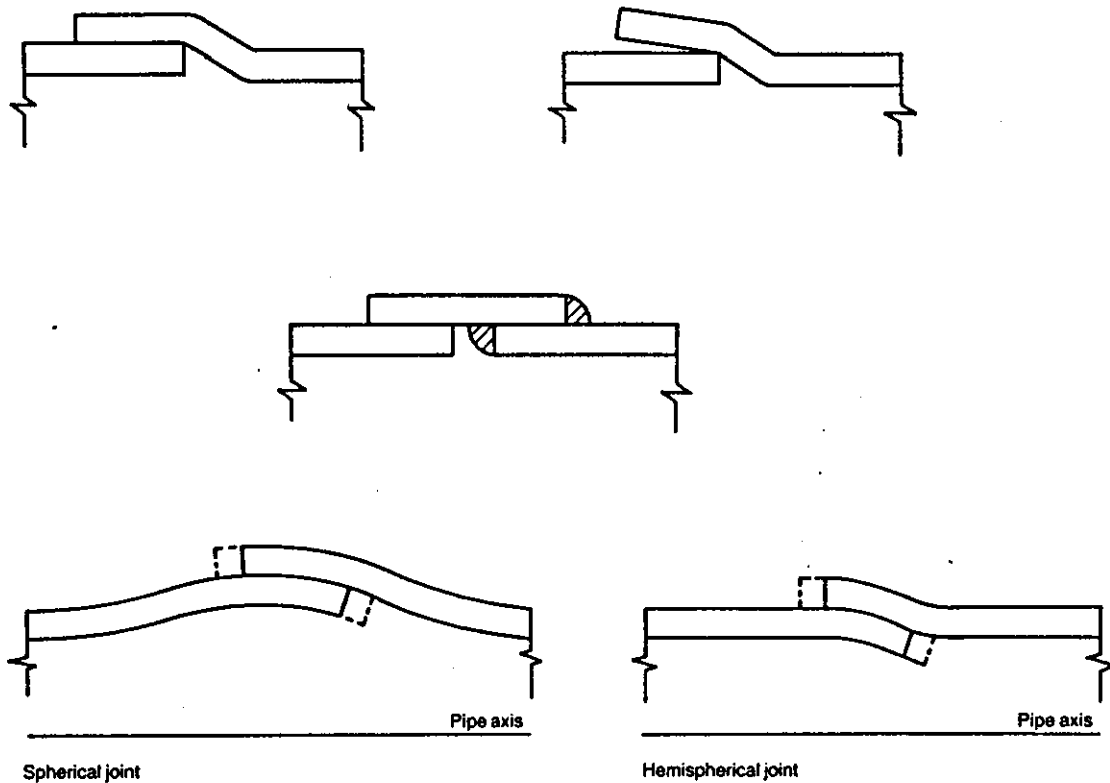


Figure 2.12

Steel Pipe Joints

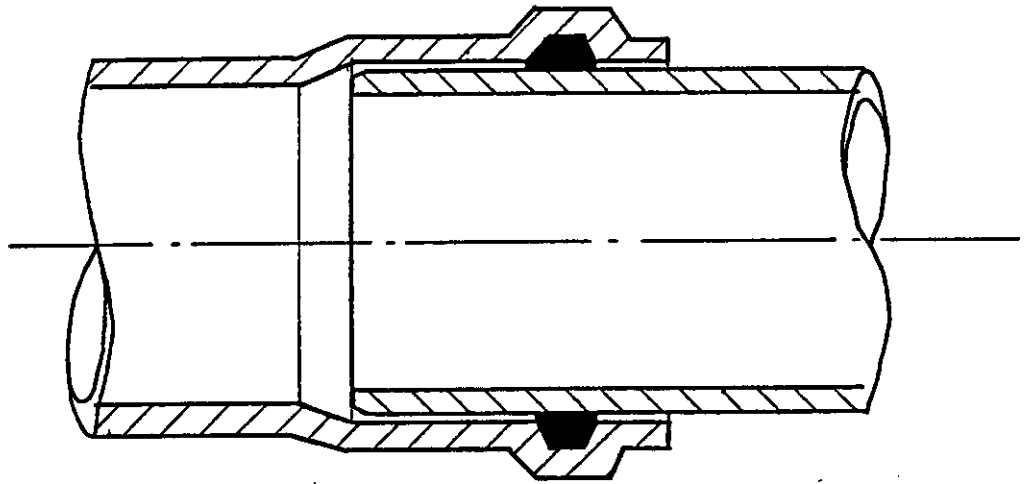
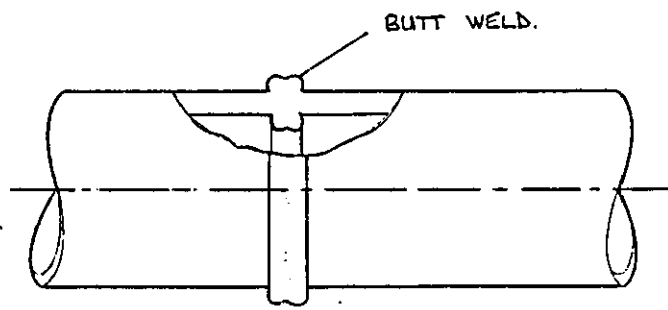
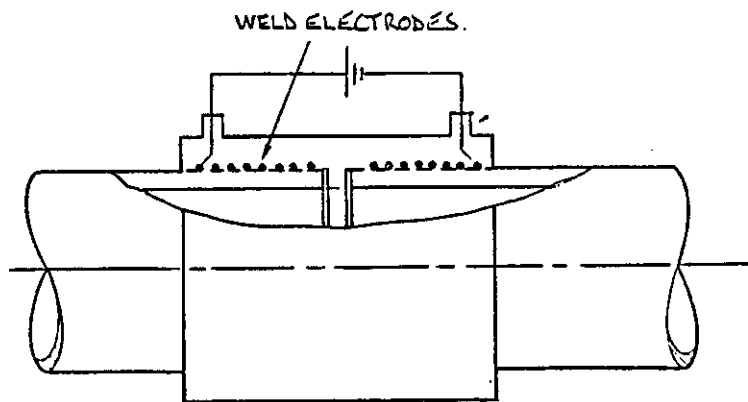


Figure 2.13

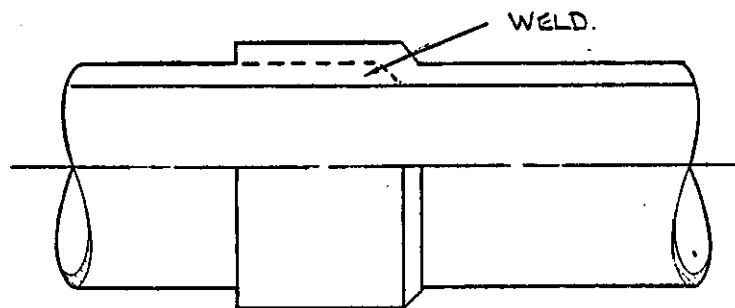
uPVC Push Fit Joints



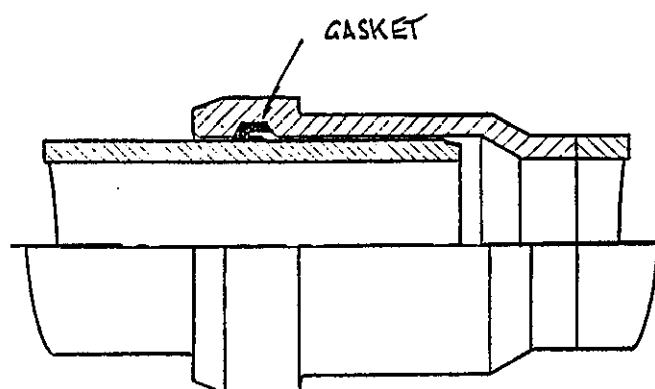
Butt Fusion Joint



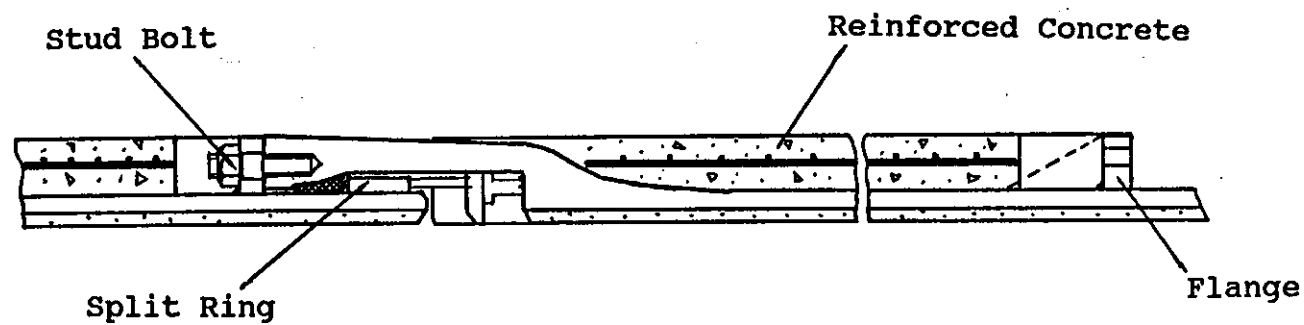
Electrofusion Joint



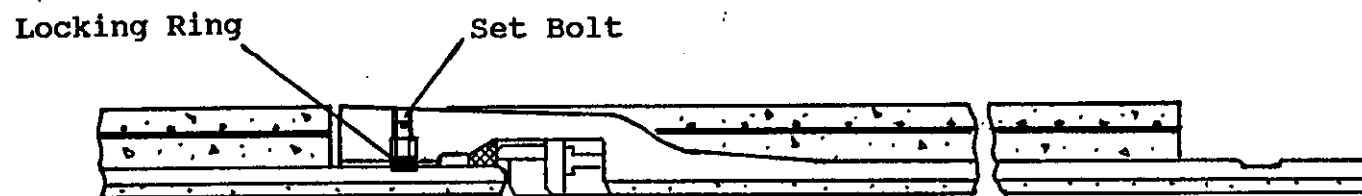
Socket Fusion Joint



Socket - Spigot Joint



U-D TYPE JOINT



UF-D TYPE JOINT

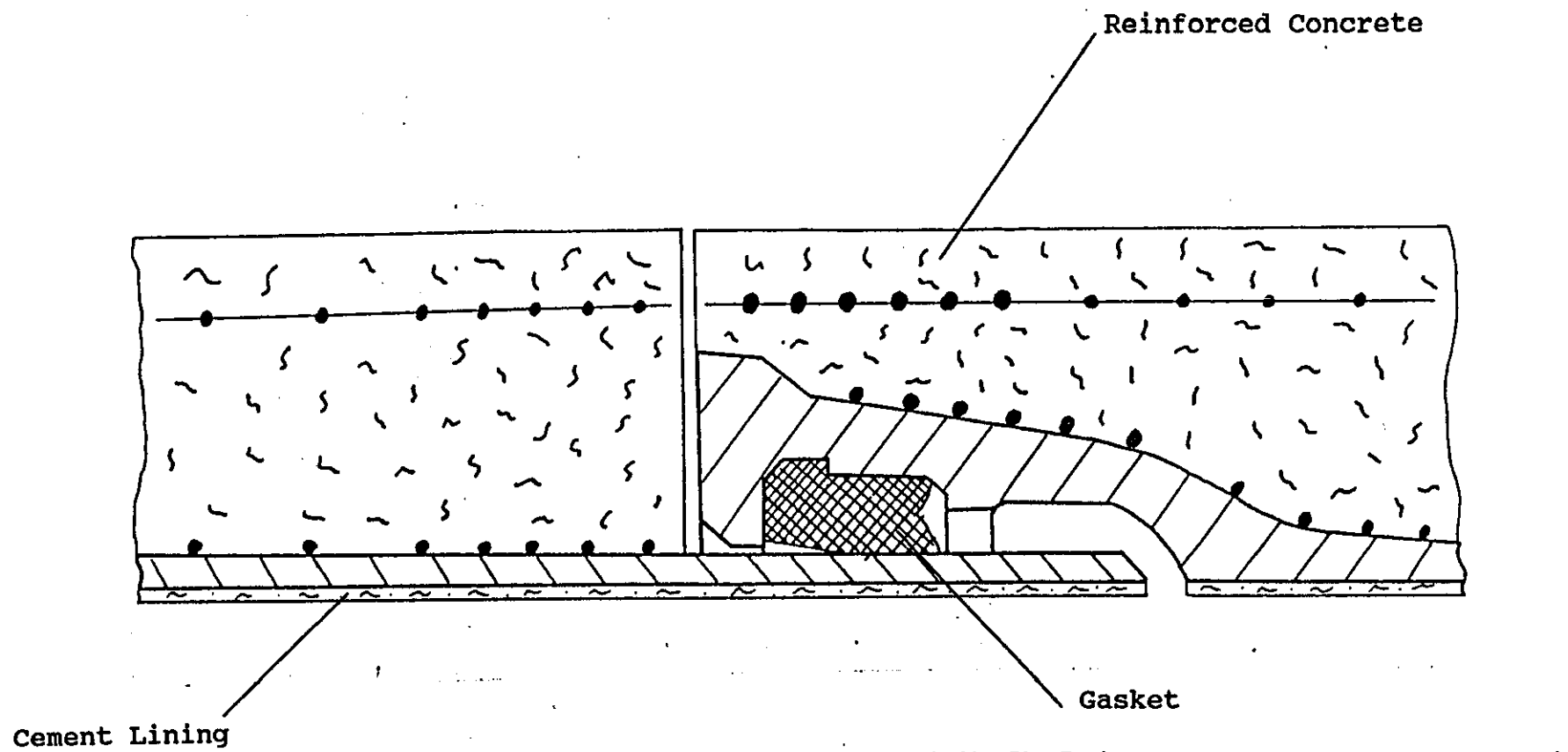


Figure 2.16

PAM Prototype Jacking Pipe Design ( after Stevens , 1989 )

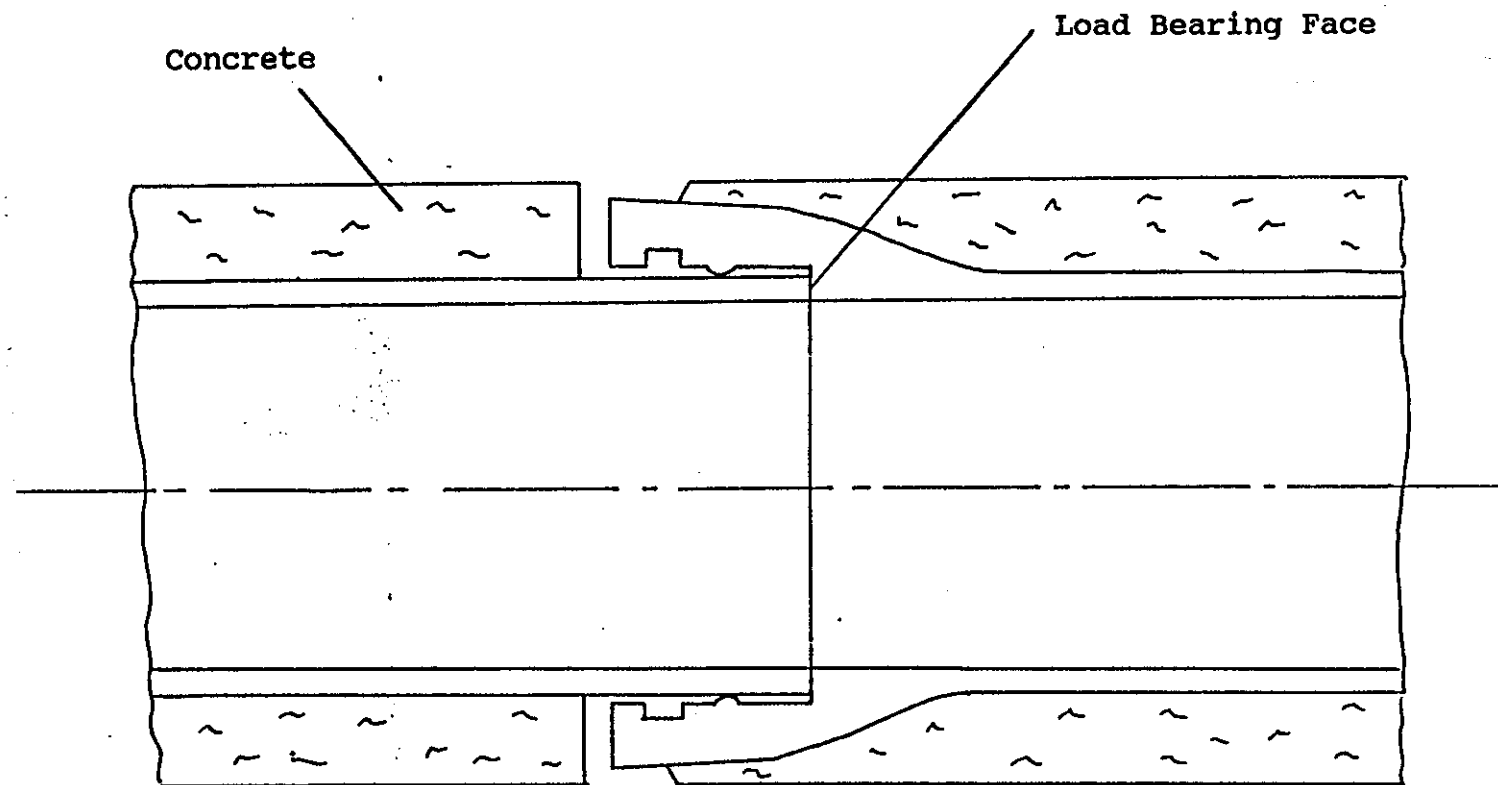


Figure 2.17

German Jacking Pipe Design

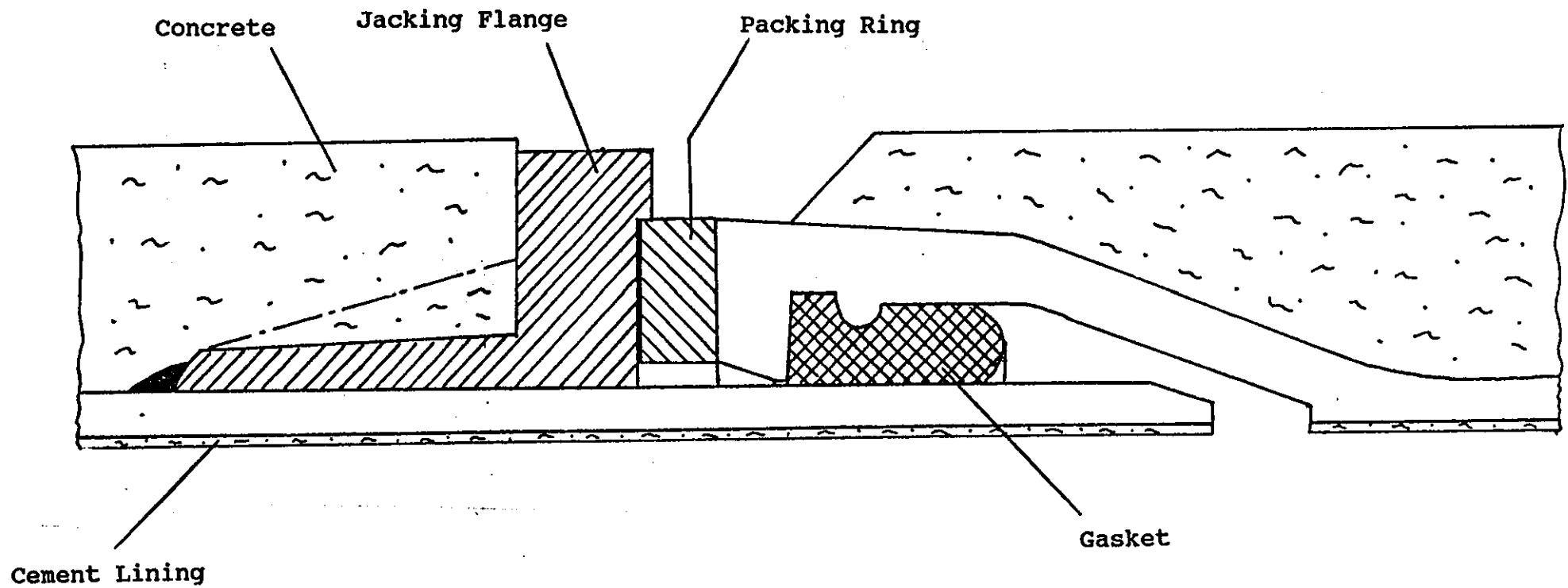


Figure 2.18

Prototype Ductile Iron Jacking Pipe. ( after Stevens , 1989 )

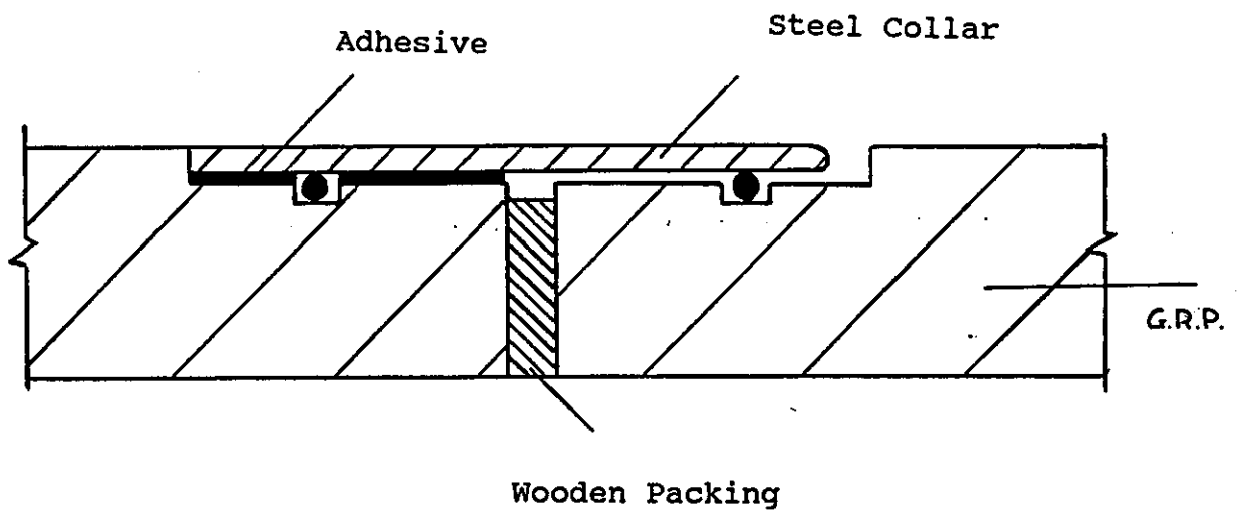


Figure 2.19

GRP Jacking Pipe Design

# GRP Jacking Pipes

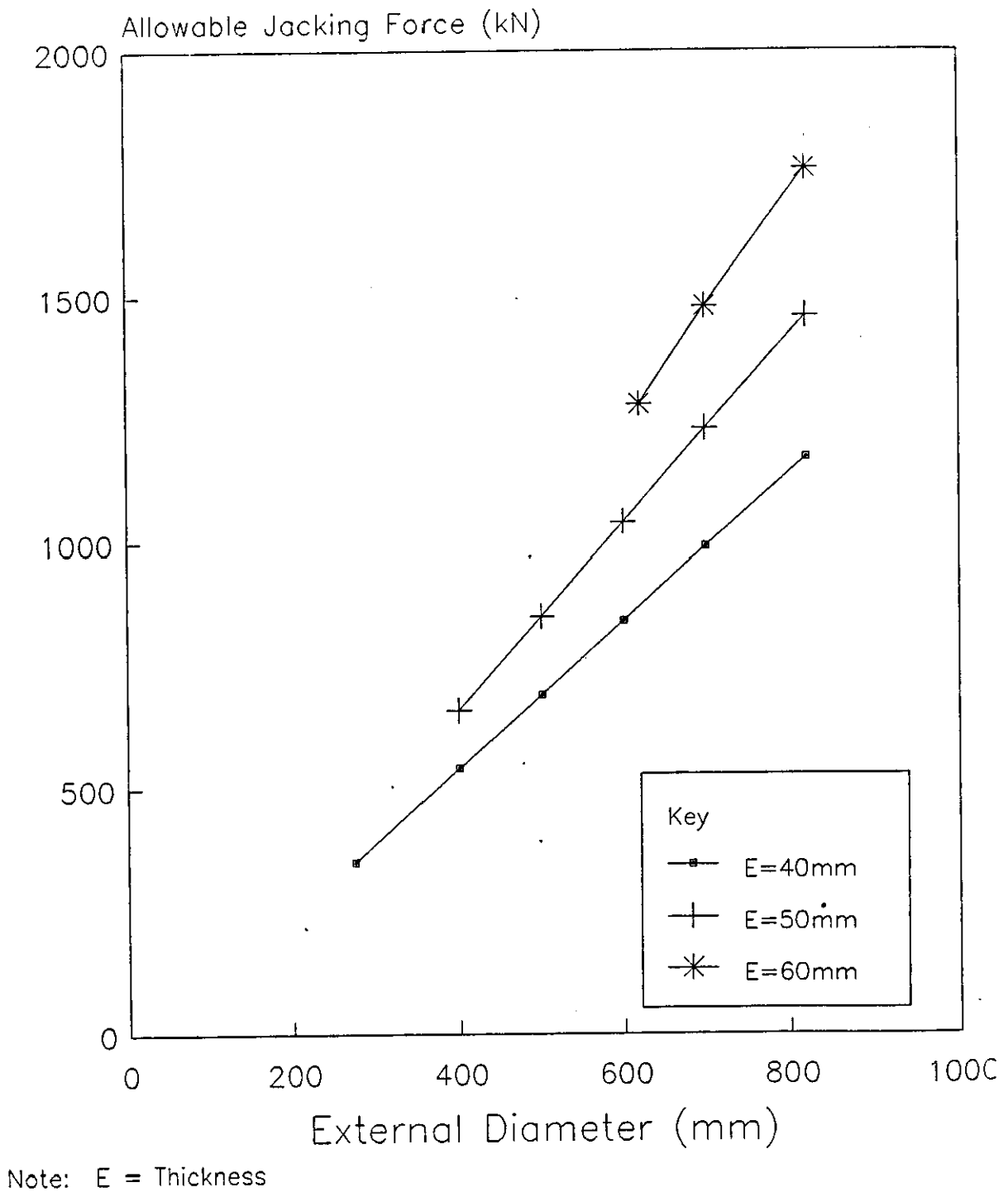
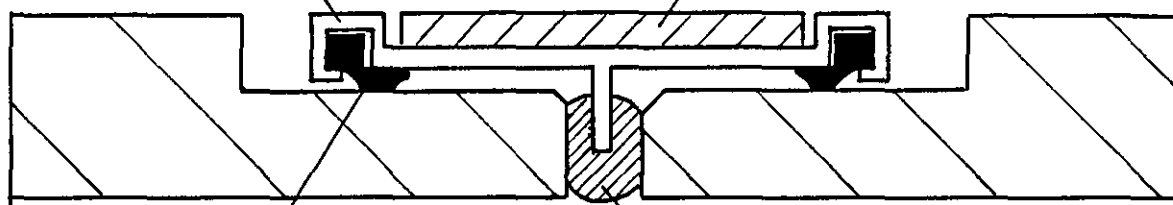


Figure 2.20

Allowable Jacking Loads for GRP Pipes

Polypropylene Coupling

Rubber Sleeve

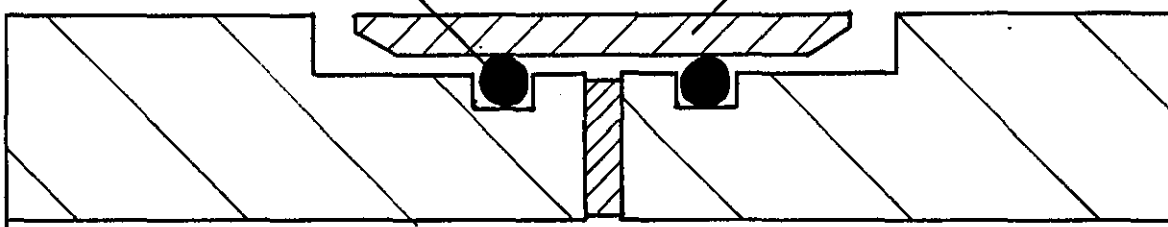


Gasket

Rubber Packing Ring

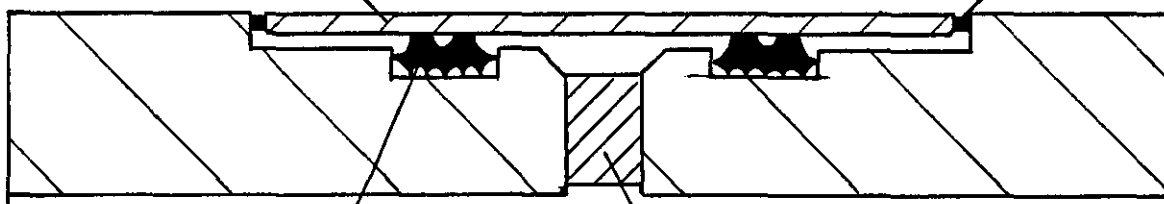
Gasket

Rigid Plastic Ring



Steel Reinforced  
Composite Sleeve

Rubber Filler



Gasket

Packing Ring

Figure 2.21a

Clay Jacking Pipes

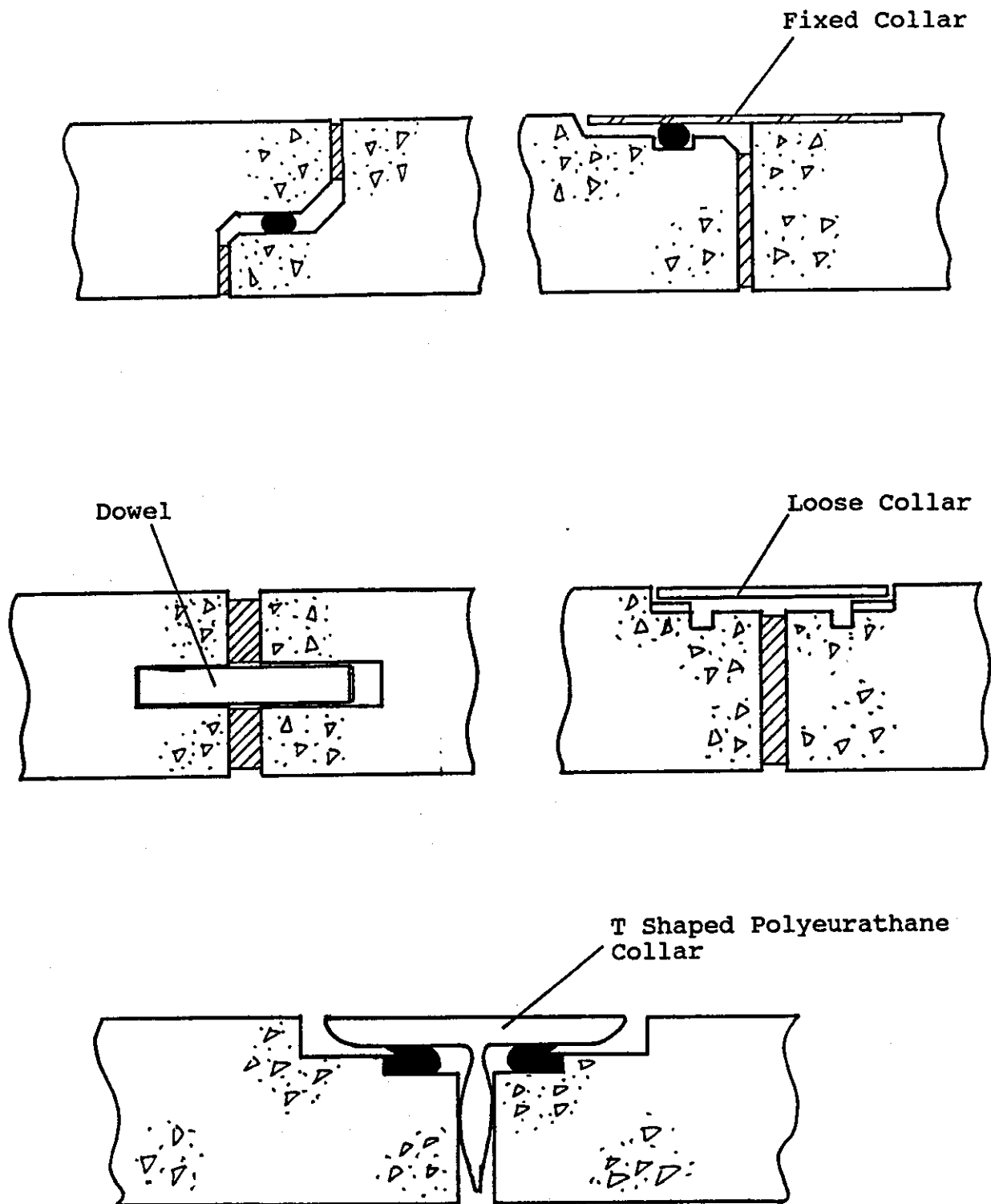


Figure 2.21b

Concrete Jacking Pipe Designs

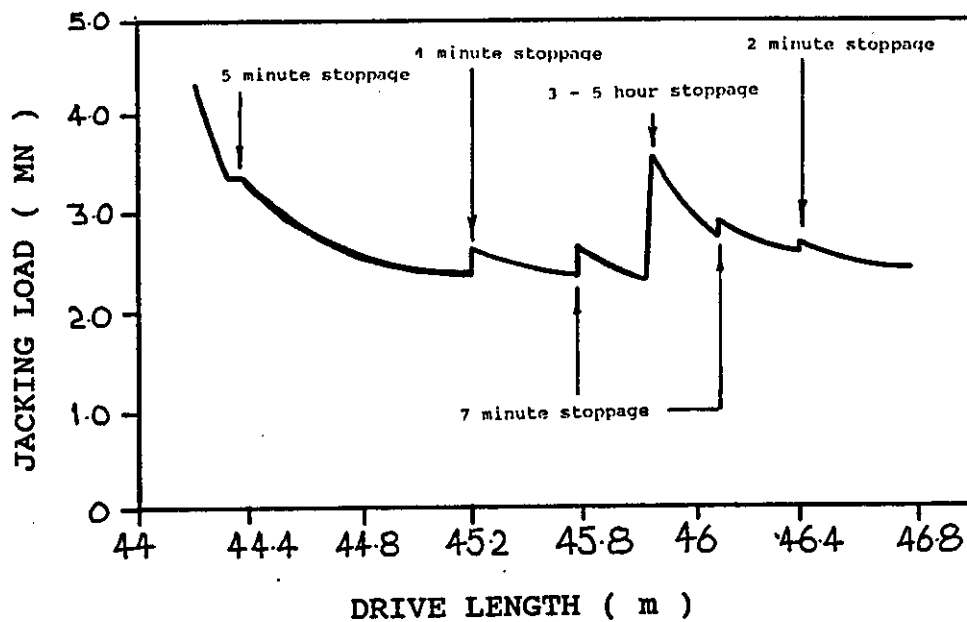


Figure 2.22 Graph showing increases in Jacking Loads with Stoppage Time (after Milligan and Norris 1992)

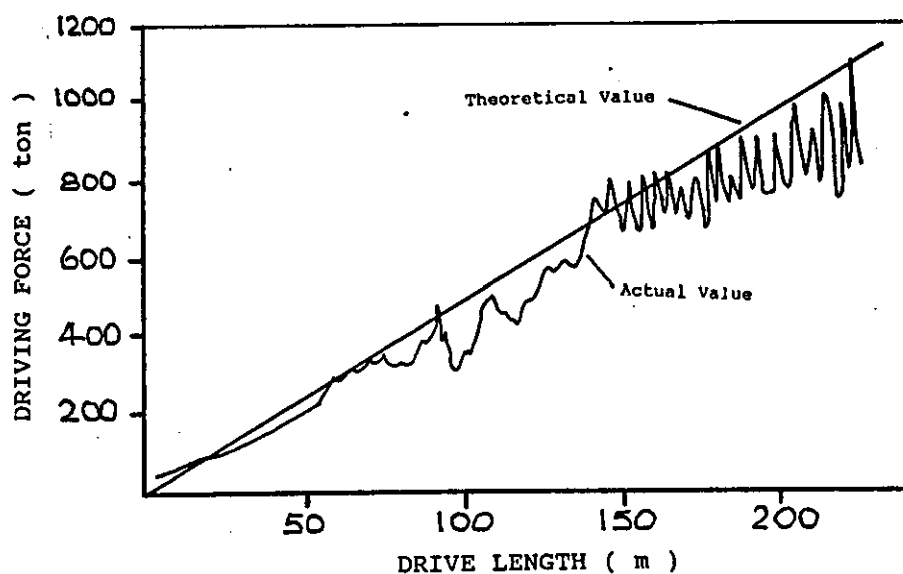
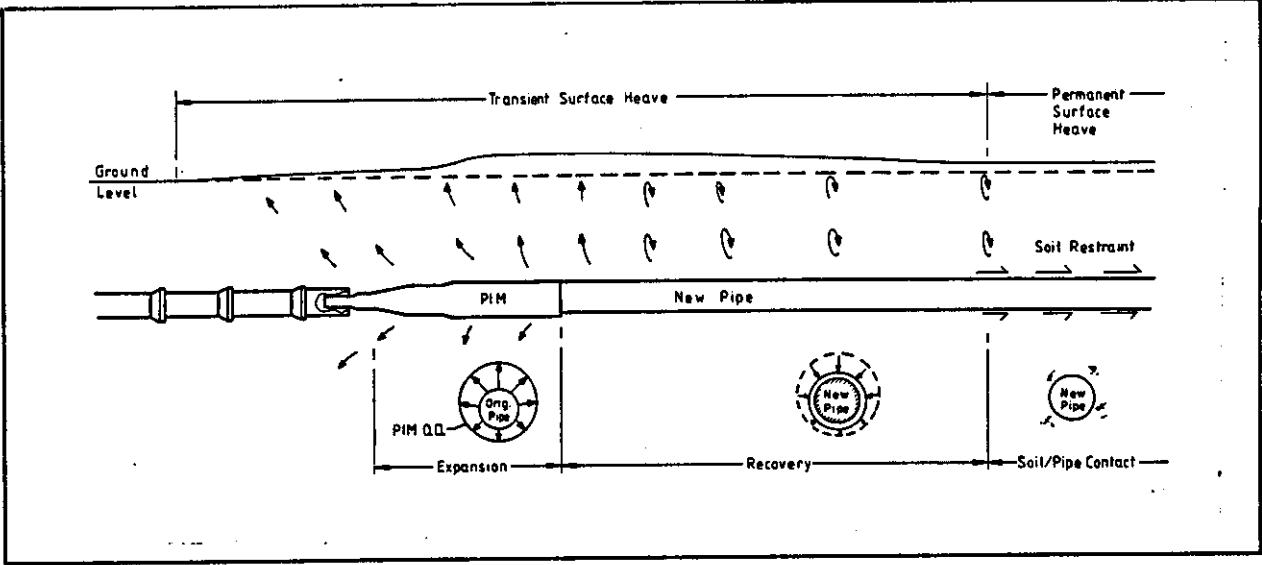
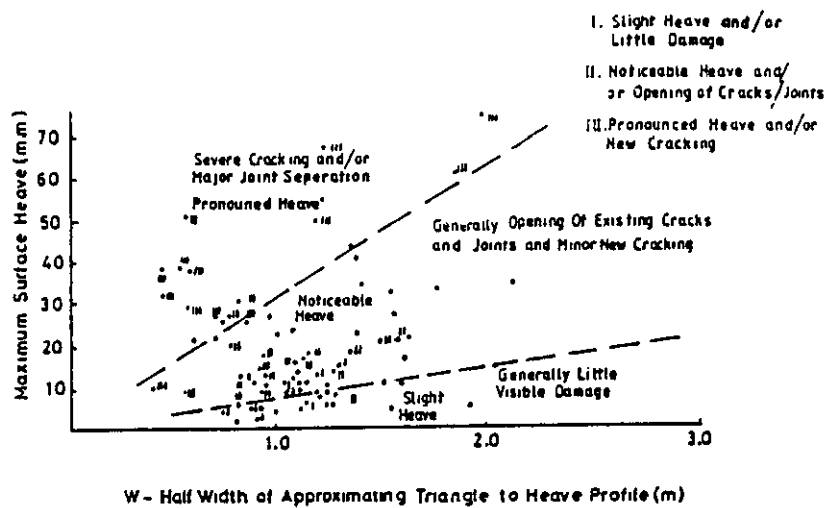


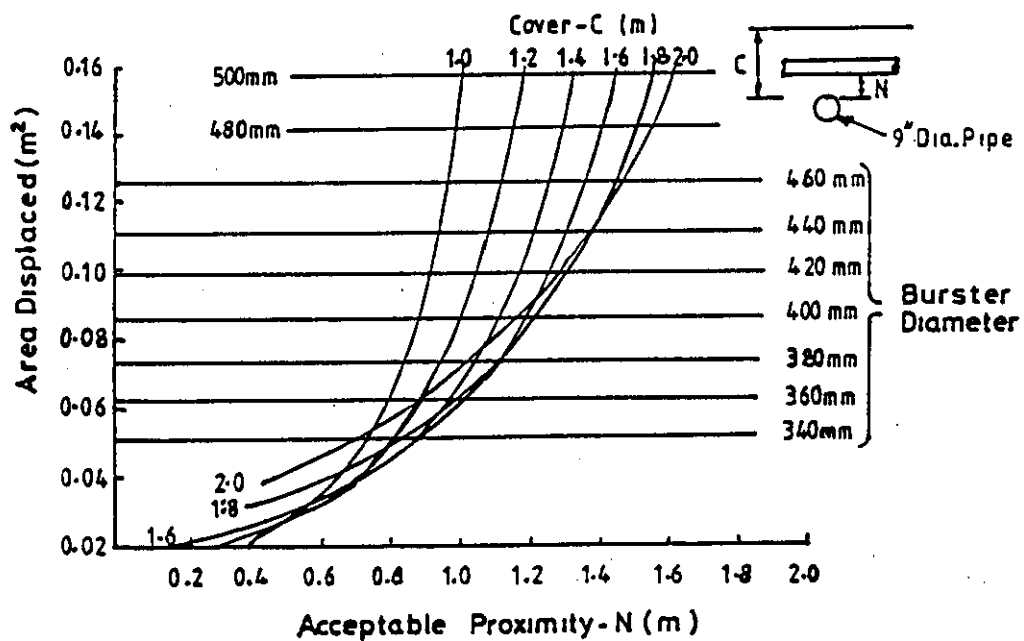
Figure 2.23 Comparisons of Theoretical and Actual Jacking Loads for Pipe Jacking (after Water Services 1981)



**Figure 2.24** Ground Movements caused by Pipe Bursting Operations  
(after Leach and Reed 1989)



(a) Surface damage chart



(b) Safe proximity chart

Figure 2.25

Pipe Bursting Surface Damage and Proximity Charts  
(after Leach and Reed 1989)

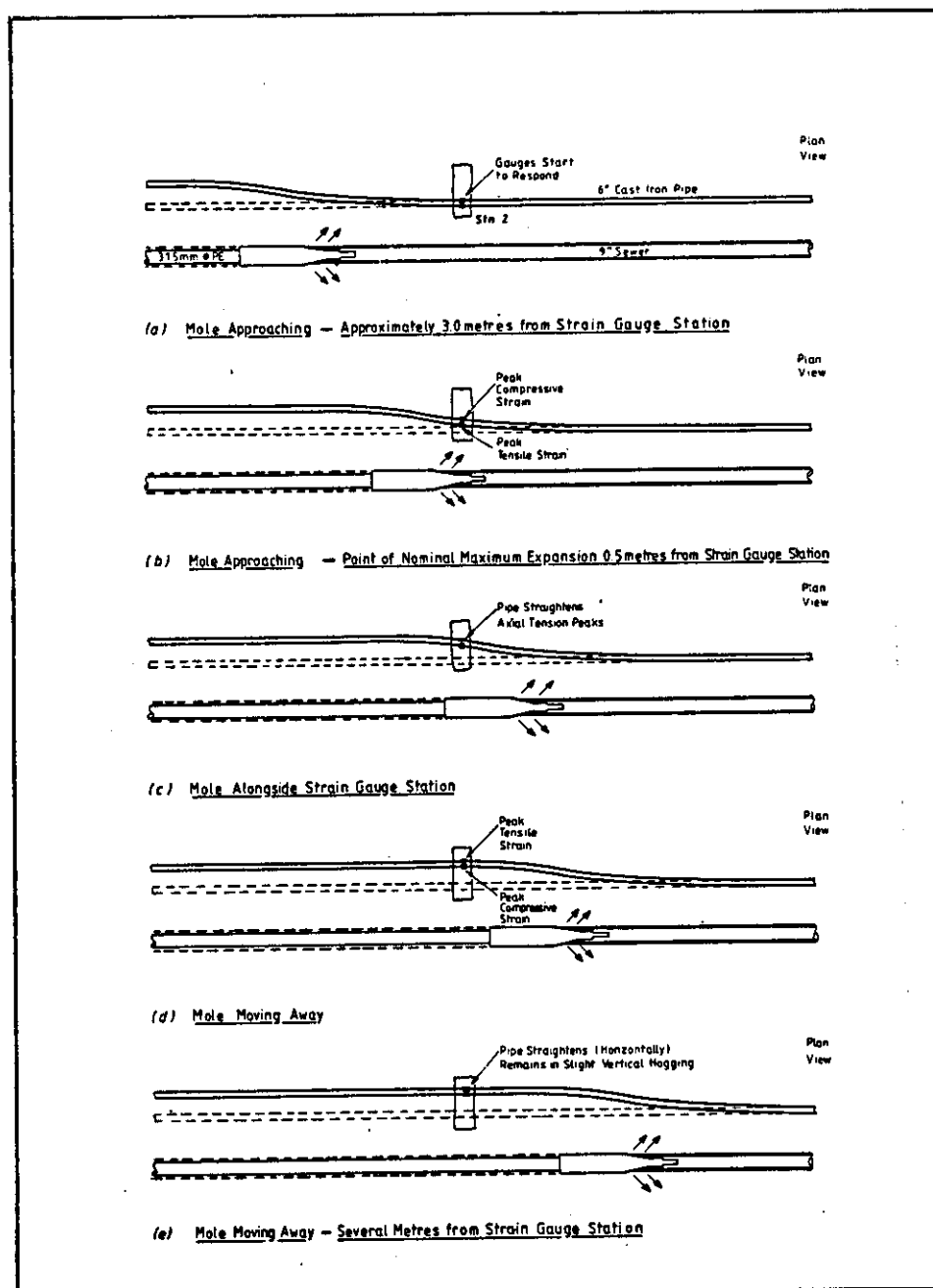
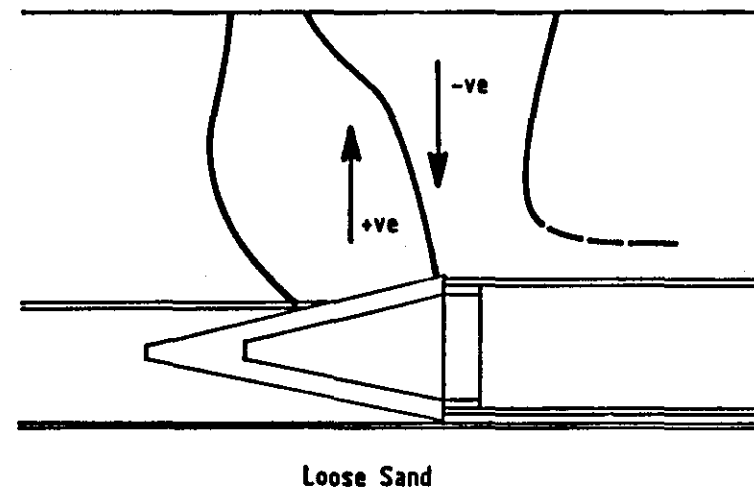
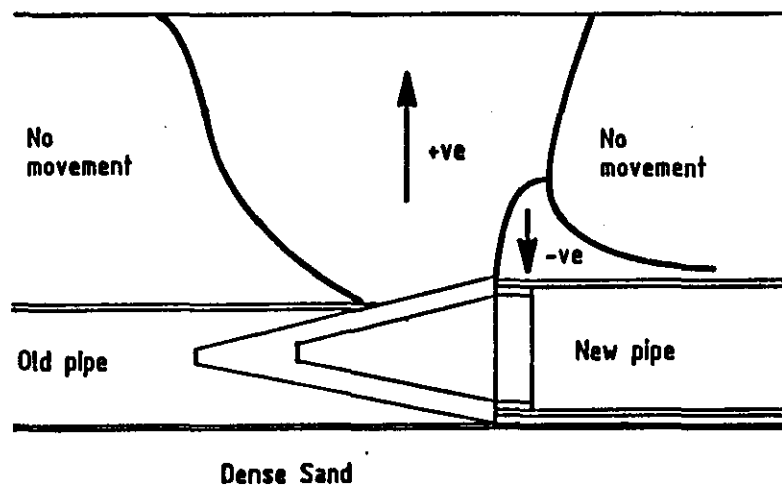
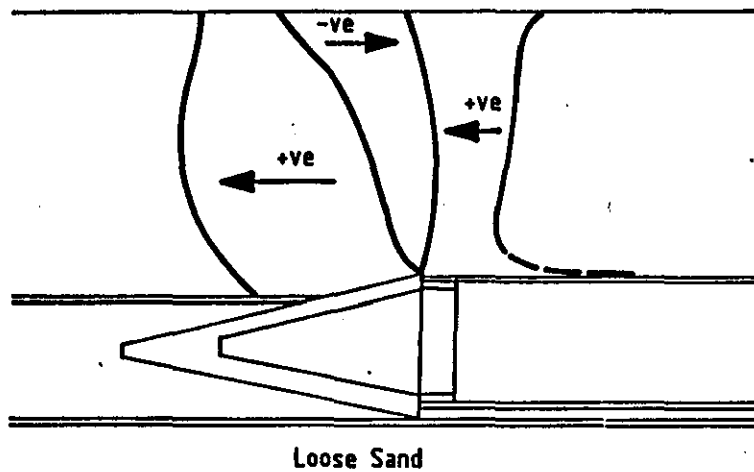
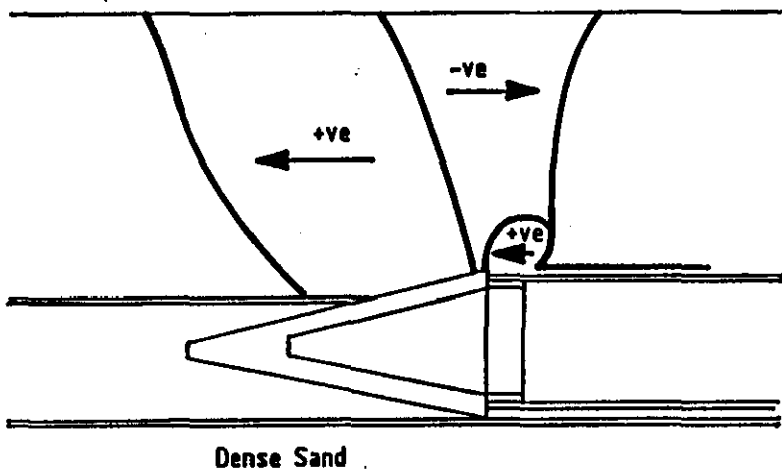


Figure 2.26

Strain Gauge Readings during a Pipe Bursting Operation  
(after Leach and Reed 1989)



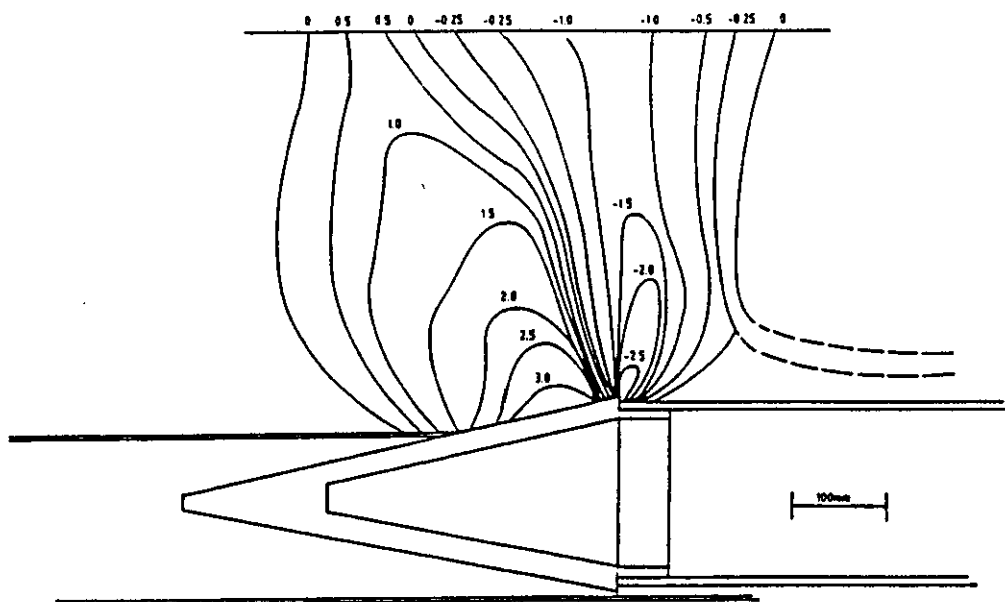
#### VERTICAL MOVEMENTS



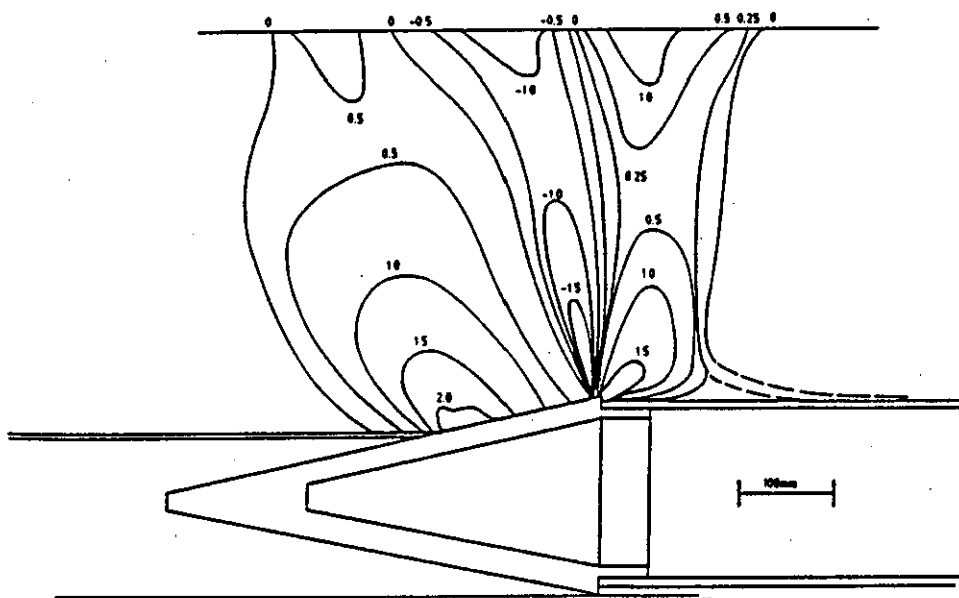
#### HORIZONTAL MOVEMENTS

Figure 2.27

Ground Movements associated with Pipe Bursting Tests  
(after Chanman 1992)



(a) VERTICAL MOVEMENTS  
+ UP - DOWN



(b) HORIZONTAL MOVEMENTS  
+ LEFT - RIGHT

Figure 2.28

Displacement Contour Plots for Pipe Bursting  
(after Chapman 1992)

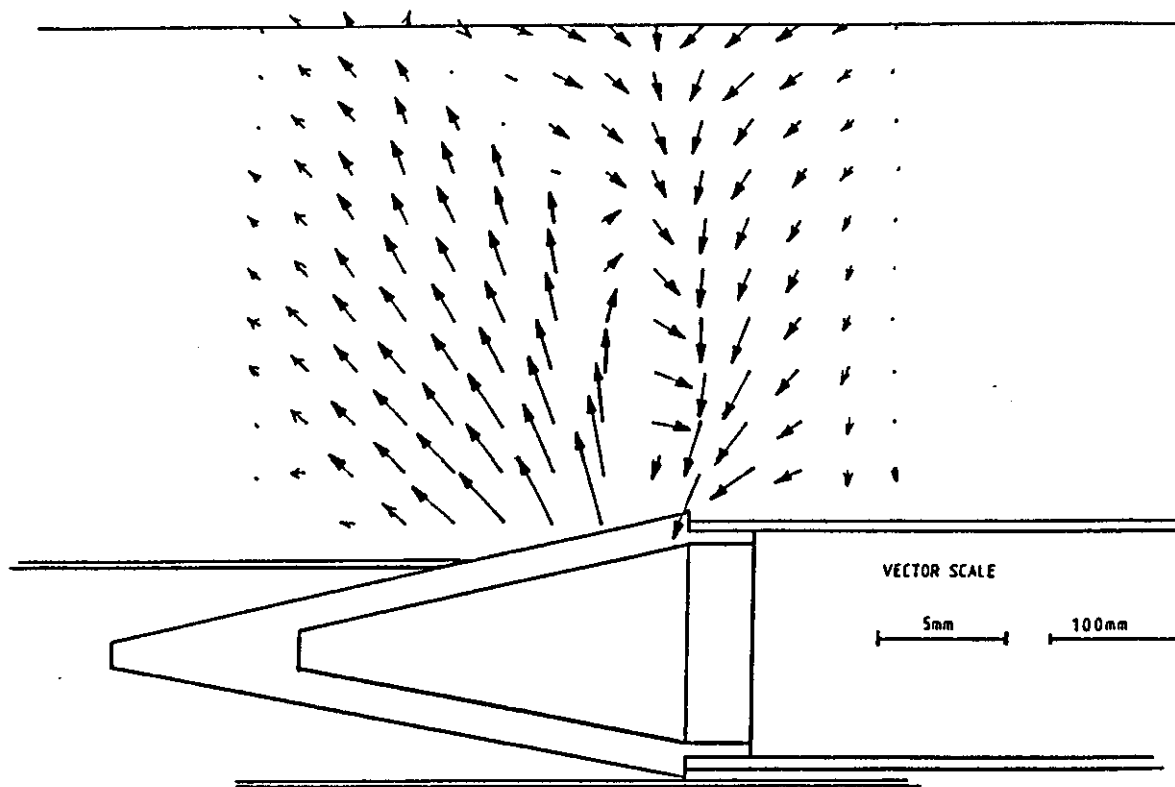


Figure 2.29

Total Displacement Vector Plot for Pipe Bursting  
(after Chapman 1992)

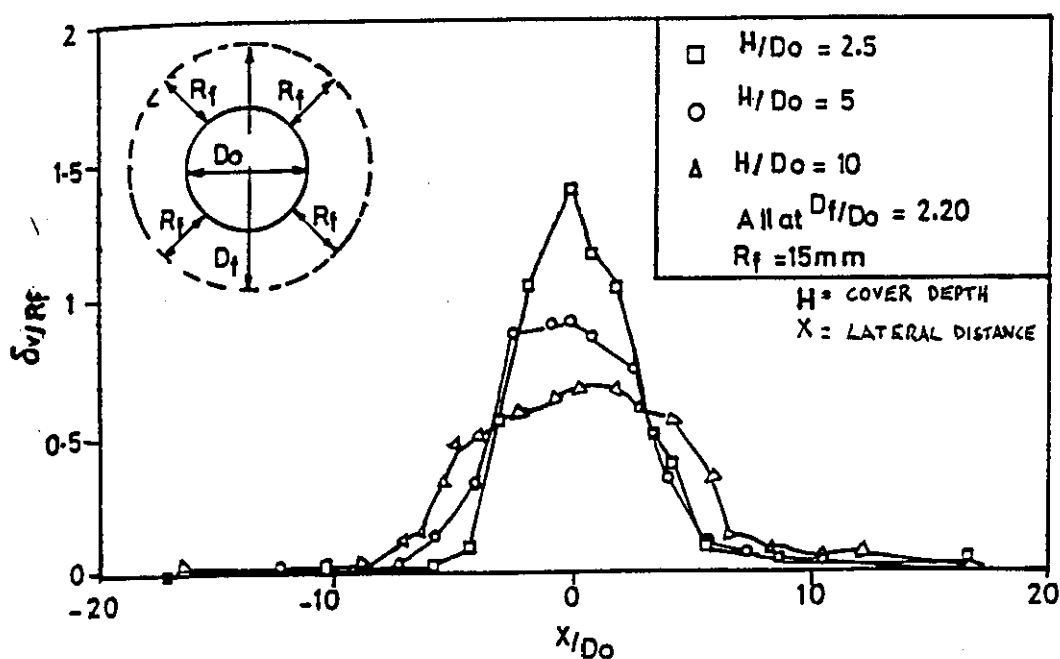


Figure 2.30

Surface Heave Profiles for Pipe Bursting Model Tests in Sand  
(after Swee and Milligan, 1990)

## **CHAPTER THREE**

### **RESEARCH PHILOSOPHY AND APPROACH**

### 3.1 INTRODUCTION

Water industry requirements and potential markets for ductile iron pipe products for use with trenchless technology were discussed in Section 2.6. This discussion established that the greatest potential market for a ductile iron trenchless pipe is in replacing ageing grey iron mains using pipe bursting techniques. The requirements perceived for such pipe are listed below:

- (i) the pipe should be suitable for installation from a small pit; individual pipes should thus be available in short lengths of 1.5 to 3.0 metres,
- (ii) the outside profile of the pipe system should be suitable for jacking or pulling through the ground behind the pipe bursting mole,
- (iii) the pipe should be designed to withstand high axial loads and to have a high working pressure rating,
- (iv) the external protective coating should remain intact during installation, and
- (v) the pipe should be available in the diameter range DN 150 to 400.

Having identified these requirements, the research was directed towards gaining a greater theoretical and practical knowledge of pipebursting techniques. The ultimate aim was then to develop a ductile iron trenchless pipelaying product to the above specification. The research was therefore carried out in three stages.

Stage One considered the fundamental mechanics of the pipe bursting process by considering pipe jacking loads and ground movements and these are described in Chapter 4. Stage Two was concerned with the design and testing of pipe products taking account of the perceived requirements and the predicted jacking loads established in Stage One. The work is described in Chapter 5. Having designed a suitable product, Stage Three tested its field performance by undertaking a full scale field trial in association with a major Water Company. This field trial is described in Chapter 6.

### 3.2 METHODS OF INVESTIGATION

The majority of research studies detailed in Chapter 2 used either laboratory tests or monitoring of full scale contracts in order to obtain data. Laboratory testing enables a relatively large number of tests to be performed and allows parameters to be altered as the test programme develops. However, these tests are often carried out using scale models and relating the results to full size work can be difficult.

The monitoring of full scale field contracts provides accurate data but is often expensive to undertake. For economic reasons the priority during such site trials is often to complete the work in the shortest possible time and this limits the amount of data which can be collected. There is also little scope for altering test conditions.

Throughout this research a number of testing techniques were used. A full-scale laboratory trial is described in Chapter 4. This utilised a unique test trench with a length of 13.4m, a width of 2.6m and a depth of 3.25m. This facility is able to withstand an end thrust of 9.47 MN and top load can be applied up to 214.5 kN/m<sup>2</sup>. The trench was prepared by laying two dummy pipelines each 8.7m in length in compacted sand. These pipelines were subsequently replaced by pipe bursting techniques. Such a trial combined the advantages of testing in the laboratory with those of monitoring full scale contracts. The data collected were of obvious value, but the research also benefitted from having gained an appreciation of how pipe bursting equipment is operated. The development of a jointing system for the ductile iron pipe bursting product is presented in Chapter 5. Detailed design work was followed by testing of prototypes using standard laboratory testing equipment such as tensile/compressive testing machines and hydraulic pressure testing equipment. Full-scale prototypes were used to avoid necessitating the use of scale modelling. The product was finally assessed by undertaking the full-scale field trial described in Chapter 6. During this trial the overall suitability of the product for use with pipebursting equipment was determined. Jacking forces and ground movements were measured during this trial in order to form a basis for assessing the overall capability of the system.

## **CHAPTER FOUR**

### **STUDY OF PIPE TO SOIL INTERACTION**

## 4.1 INTRODUCTION

Traditional designs of jacking pipe as described by Craig and Kirkland (1982) and Stevens (1989) have consisted of pipe with constant outside diameter. This has been achieved by either using an in-wall joint as used in sewer applications or by coating pipe in concrete, as described by Stevens (1989). Although such approaches provide a satisfactory technical solution, production of such designs on an industrial scale often incurs costs which counteract the advantages of using trenchless techniques. In Chapter 3 the requirements of the new pipe were presented. In this chapter the outside profile of the pipe system is discussed and the results of tests on alternative profiles are presented. The ground movements during installation are also discussed. This work expands on that reported by Robins et al, (1990).

Standard socket-spigot spun iron pipes are produced in 5.5m lengths and have the joint profile shown in Figure 4.1. Fundamental to the design of the new product is the length of each individual pipe. Since the direct production of short pipe lengths would involve heavy investment in new plant, the alternative of producing short pipes by cutting the standard product to the desired length was preferred, although this clearly has implications concerning the method of jointing pipes on site. Another important consideration is the outside profile of the jointed pipes, since traditional ductile iron pipes have an increase in diameter at the joint in order to house the sealing gasket (Figure 4.1). In pipe bursting applications, this increase in diameter may cause problems during installation due to increased jacking forces. Previous work on jacking pipes (Stevens 1989) has overcome this problem by coating the pipe barrel with concrete to give a constant outside diameter. For the diameters of pipe under consideration concrete coating is both expensive and time consuming and it would increase the weight of pipe making site handling difficult.

The overall suitability of ductile iron for use as replacement pipe was assessed by carrying out a full-scale pipe bursting trench trial. The main objective of this trial was to assess the increase in jacking load caused by the standard socket compared with constant outside diameter pipe. Ground movements during the trial were also recorded.

The trial was carried out in the test trench at Stanton Plc. The layout of the trench during the trial is shown in Figure 4.2. Two 8.7m lines of 229mm inside diameter grey iron drain pipe were laid and buried in sand that was compacted in 100-200mm layers. Compacted sand was used since this was considered to represent a 'worst case' condition, with the low cohesive strength causing the sand to collapse onto the pipes during the installation process. The pipes

were laid 1.2m above the trench floor with a depth of cover of 1.0m. The horizontal distance between the two lines was 1.0m. Plate 4.1 shows the test trench during preparation prior to covering the grey iron pipes with sand.

The grey iron pipes were expanded and burst using a pneumatic impact mole having an external diameter of 290mm. Ductile iron pipes were pulled into place directly behind the mole. The specification of these replacement pipes was as follows:

- |        |              |  |
|--------|--------------|--|
| Line 1 | (Figure 4.2) | DN 250 smooth profile pipes 1.5m length          |
| Line 2 | (Figure 4.2) | DN 200 socket-spigot pipes 1.5m effective length |

DN 250 smooth profile pipes were chosen in order to represent concrete coated DN 200 pipe.\* The sequence of operations used during the trial is shown in Figure 4.3. Each line was expanded and the respective replacement pipes installed by the mole (operations a and b, Figure 4.3). The newly installed pipes were then jacked through each line to provide a comparison of jacking loads (operations c and d, Figure 4.3).

#### 4.2 EFFECT OF JOINT PROFILE ON JACKING LOADS

The loads required to jack the respective pipes through lines 1 and 2 are shown in Figure 4.4. These results were recorded during trial stages c and d (refer to Figure 4.3), during which the respective pipes were jacked through the trench. The jacking length was a constant 8.7m during these two stages of the trial. Hence the horizontal axis of the graph on Figure 4.4 indicates the length of pipe installed and not the jacking distance.

The maximum, minimum and average loads for socket-spigot and smooth pipes are compared in Table 4.1. For the DN 250 smooth profile pipes jacking loads varied between 8 and 76 kN, the average load being 42 kN. For the DN 200 socket-spigot profile pipes the loads were generally higher and varied between 25 and 89 kN, with an average jacking load of 56 kN. Thus the socket profile caused average load to increase by 33% with the maximum recorded load increasing by 17%.

The jointing method used for smooth profile pipe allowed deflection at the joint and when this occurred there was potential for increased frictional resistance due to the increased bearing area associated with the opening of the joint. Similar circumstances occur when smooth profile concrete jacking pipe joints are deflected (Figure 2.21b). In contrast, the socket profile

\* It should be noted that the diameter of smooth pipe was chosen to represent current pipe jacking practice of coating ductile iron jacking pipe with concrete. Since DN 250 smooth profile pipes have the same external diameter as concrete coated DN 200 socket - spigot pipes, they were considered representative for this trial. The internal diameters of the pipes were not considered important since it was the jacking force which was under consideration and not the capacity of the main.



Plate 4.1: Test trench during preparation for laboratory trial

Table 4.1: Jacking load comparisons between socket-spigot and smooth profile pipes

LINE	MAXIMUM LOAD kN	AVERAGE LOAD kN	MINIMUM LOAD kN
1. DN 250 SMOOTH	76	42	8
2. DN 200 SOCKET-SPIGOT	89	56	25

Table 4.2: Comparison between contact stress values measured during the laboratory trial and those predicted by theoretical analysis for smooth profile pipe.

LOAD MEASURED DURING TRIAL kN/m <sup>2</sup>		AULDS PREDICTIONS kN/m <sup>2</sup>		JAPANESE PREDICTION FROM FORMULA 3 kN/m <sup>2</sup>
MAXIMUM	AVERAGE	METHOD 1	METHOD 2	
10.2	5.6	9.81	6.2	9.42

pipes were able to deflect with no increase in bearing area at the joint. Although this is worthy of note, the effect on overall jacking load is considered small for the size of pipe used.

In Chapter 2 the various methods of predicting pipe jacking forces were presented. A comparison of each of these methods with the results from the trial is given in Table 4.2. The maximum load recorded during the trial was 89 kN on line 2. This gives a contact stress, calculated by dividing the jacking load by the pipe surface area, of  $14.7 \text{ kN/m}^2$ . Auld (1982) quoted contact stress values for pipe jacking through wet and dry sand, these being 10-15 and 25-45  $\text{kN/m}^2$  respectively. Since the soil mechanics of pipe bursting are different from those of pipe jacking, Auld's figures are not directly comparable with the trial results. However, it is useful to note that the trial stress values are in the upper limits of the wet sand range.

Auld also presented two analytical methods of predicting jacking loads, each based on ground pressures, soil type and experience. Using these two methods, the load required to jack DN 250 smooth pipe was calculated and the results are compared with the trial results in Table 4.2.

The predicted stresses are seen to be comparable with those experienced during the trial. The stresses predicted by Method 1 give a good estimation of the maximum stress experienced during the trial whilst Method 2 correlates well with the average stress.

Further comparison of load values for each line reveals a number of interesting points. For the smooth profile pipes (line 1), Figure 4.4 shows that two distinct regions exist. In the first region, between 0 and 4.5m of pipe installed, the load is seen to fall initially before rising to its maximum at approximately 4.0m. In the second region, between 4.5 and 9.0m the jacking load falls at a steady rate.

With the socket-spigot profile pipes (line 2), similar behaviour is observed at the start of jacking with the load dropping before rising to a maximum at around 2.5m. However, after this the load does not drop with increased jacking but continues to vary between 30 and 89 kN. The average load therefore remained at a constant level (approximately 56 kN) throughout the jacking process. This observed behaviour indicates that after a certain amount of jacking, the smooth pipes establish a route of minimum resistance through the sand and once this occurs the jacking load decreases for this fixed drive length. For the socket-spigot profile however, the sockets continually disturb the surrounding sand causing the average load to remain at a

similar level throughout. The pipe sockets also cause a greater variation in load than that observed with the smooth pipes.

It is useful to use the load data recorded during the trial to predict loads which may be expected over longer jacking distances. The maximum contact stress of  $14.7 \text{ kN/m}^2$  converts to a jacking load of 520 kN over a drive of 50m for DN 200 pipes. For a ductile iron pipe, a 520 kN axial load equates to an axial pipe stress of  $166 \text{ MN/m}^2$ . Ductile iron components are generally designed to a compressive stress of  $300 \text{ MN/m}^2$ . It is well known that contact stresses are increased when a joint is fully deflected (Stevens (1989)). Hence during a 50m drive with a fully deflected joint the axial pipe stress may increase to the order of twice the uniform stress ( $2 \times 166 = 332 \text{ MN/m}^2$ ) if a small amount of local yielding is allowed.

### 4.3 GROUND MOVEMENTS DURING PIPE INSTALLATION

#### 4.3.1 Measurement Techniques

Leach and Reed (1989) and Chapman (1992) observed similar patterns of ground movement during pipebursting operations (Chapter 2). Users of pipebursting technology are primarily interested in the degree of vertical ground movement and the degree of influence on adjacent services likely to occur during their operation. Hence these two parameters were monitored during the trial. Vertical sand movements were monitored using a series of soil displacement rods positioned within the soil mass. These rods were placed at various levels above the pipe, at positions along the longitudinal axis of each line and offset from this axis. The positions of the rods with respect to the pipes are shown in Figure 4.5. The degree of influence on adjacent services was monitored by measuring the strain induced in line two whilst line one was expanded. Longitudinal and hoop strain gauges were placed at the crown and inner springings of the grey iron pipes at two separate locations as indicated in Figure 4.6.

#### 4.3.2 Vertical Sand Movement

The movements of the soil displacement rods during expansion of lines 1 and 2 are shown in Figures 4.7 and 4.8 respectively. In each figure, sand surface movements are shown at the top of the diagram and movements within the soil mass, at the various levels, are shown in the lower half of each diagram. Figures 4.7a and 4.8a show movements along the length of lines 1 and 2 respectively, whilst Figures 4.7b and 4.8b show movements across the trench. In each case, sand movements are presented for two burster positions, the first corresponds to the

burster passing under the displacement rods, whilst the second corresponds to the burster completing the drive.

Figures 4.7a and 4.8a show that in the region of up to 300mm above the pipe, vertical sand movement generally decreases with increasing distance from the pipe. However, at 300mm above the pipe (rods 15 and 16, Figures 4.7a and 4.8a) the sand movement is similar to that at the surface. The bursting action has therefore caused the sand in the immediate vicinity of the mole to be compressed. In addition the body of sand extending from the mole to the surface was lifted as a single mass. For expansion at four diameters cover, Leach and Reed (1989) predicted similar results and stated that soil compression zones are restricted to the vicinity of the pipe bursting mole and that movement of the soil mass is likely to extend to the surface.

The behaviour of the sand above the smooth profile pipes (line 1) can be compared with that above the socket-spigot pipes (line 2) by comparing Figures 4.7a and 4.8a and 4.7b with 4.8b. For smooth profile pipes the rods generally settled after their initial displacements (Figure 4.7b) and this behaviour continued throughout the jacking process.

For the socket-spigot pipes the rods were initially displaced and then settled during installation (Figure 4.8b). However, during the jacking process the rods were observed to rise and fall thereby indicating that the sand was repeatedly falling onto the barrel of the pipe and was forced outwards each time a socket passed by. Such behaviour gives further explanation for the difference in measured loads observed in section 4.2. The loads remained high on the socket-spigot pipes due to the increased disturbance of the sand and increased jacking resistance caused by the sockets. Comparison of Figures 4.7b and 4.8b also shows that movement of the surface during installation was greater for the socket-spigot pipes than for the smooth pipes.

The surface movements observed during the trial are compared with those presented by Tasker and Leach (1988) and Swee and Milligan (1990) in Figure 4.9. This uses a similar analysis to that used by Tasker and Leach by plotting a non-dimensional maximum surface heave  $\delta_v/RF$  versus  $H^2/D^3$ . In general the surface movements measured during the trial are greater than those observed in the field by Tasker and Leach. This is probably due to the difficulty in reproducing field conditions in the laboratory. The large variation in results observed by Tasker and Leach is another factor which restricts the direct comparison of results. Such variation is probably due to differing site conditions. Despite this, the results are within the total range of values reported by the above authors.

### 4.3.3 Induced Pipe Strains

Longitudinal and hoop strain gauges were used to monitor the strain induced in line two whilst line one was expanded. The strain gauges were installed at two separate locations along the length of the drive, as shown in Figure 4.6. At each location the gauges were placed at the crown and inner springings of the grey iron pipes. The strain gauge records are shown in Figure 4.10, 4.11 and 4.12. In general the strain values measured during the trial were low, with a maximum recorded strain of  $58 \mu\epsilon$  in the hoop direction at location 1. Allowable strain values for water distribution are in the order of 300 - 400  $\mu\epsilon$  whilst British Gas have quoted 100  $\mu\epsilon$  as the maximum allowable.

Figure 4.10 shows a record of the hoop strain as the mole passes location 1. It is interesting to note that the maximum hoop strain at the springings coincides with a minimum hoop strain at the crown. This indicates that the pipe was forced into an oval shape as shown in Figure 4.13a. Once the mole has passed by the strains are seen to return to equal levels at the crown and springings indicating a re-rounding effect.

Longitudinal strains are shown in Figures 4.11 and 4.12. The breaks in the graphs represent overnight stoppages. At each stoppage the strains are seen to decrease, thus showing the transient nature of the induced strain. This was also observed by Leach and Reed (1989). Longitudinal strains increased progressively as moling continued on each day, but then decreased once moling was completed. It is likely that the longitudinal strains are due to longitudinal bending as indicated in Figure 4.13b.

The behaviour observed is consistent with the work of Dorling (1984) and Leach and Reed (1989). Dorling recorded extremely low strains (maximum  $14 \mu\epsilon$ ) in a pipe running parallel to the pipebursting work but at a distance of 1.42m. The distance between the pipes during this trial was similar to that used by Leach and Reed and the strain values are noted to be comparable.

#### 4.4 CONCLUDING REMARKS

The tests described in Chapter 4 have shown that in sand, the use of ductile iron pipe with a conventional socket-spigot profile increases maximum jacking loads over those needed for smooth pipe by approximately 17%. Similarly, average loads are increased by around 33%. Under predicted maximum load conditions the axial pipe stress is well within the capabilities of ductile iron material and contact stresses at a deflected pipe joint may increase to twice this axial stress before significant yielding of the material occurs.

Movement of the sand mass (both vertical and horizontal) when installing socket-spigot profile pipes was greater than that recorded for smooth profile pipes. At a cover depth of 1 metre, compression of the sand mass was restricted to the immediate vicinity of the mole and vertical displacement of the sand *en bloc* extended to the surface. Surface movements recorded in the laboratory were generally greater than those recorded in the field by other authors.

The pipebursting operation had very little influence on an adjacent pipe at a distance of 1 metre. The strains recorded were within recognised acceptable limits and were consistent with those obtained by other authors carrying out similar trials.

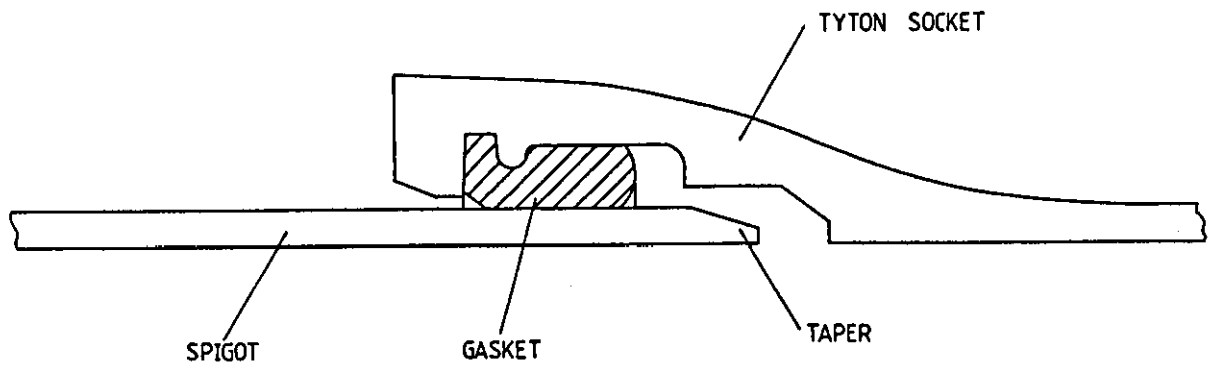


Figure 4.1

Standard Ductile Iron Socket

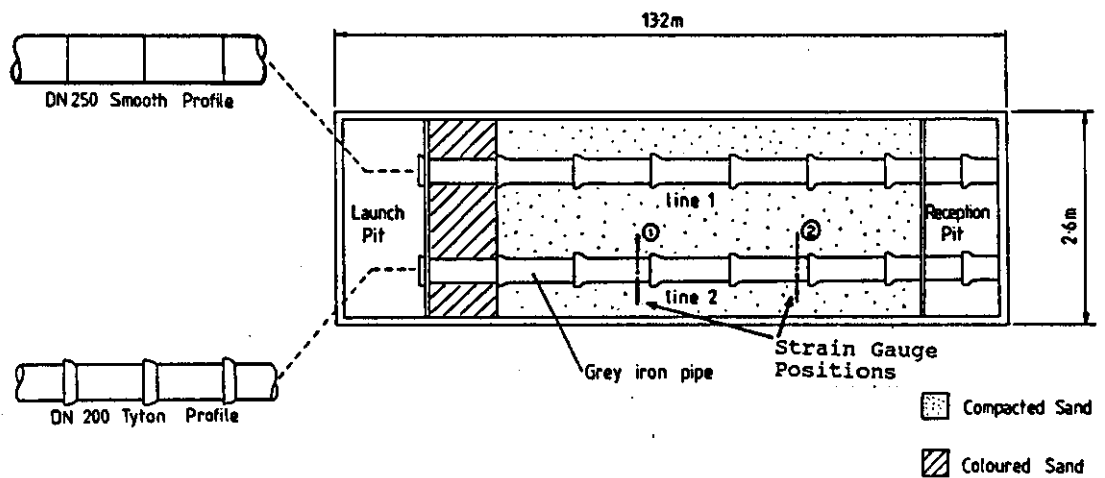


Figure 4.2

Layout of Test Trench during Laboratory Trials

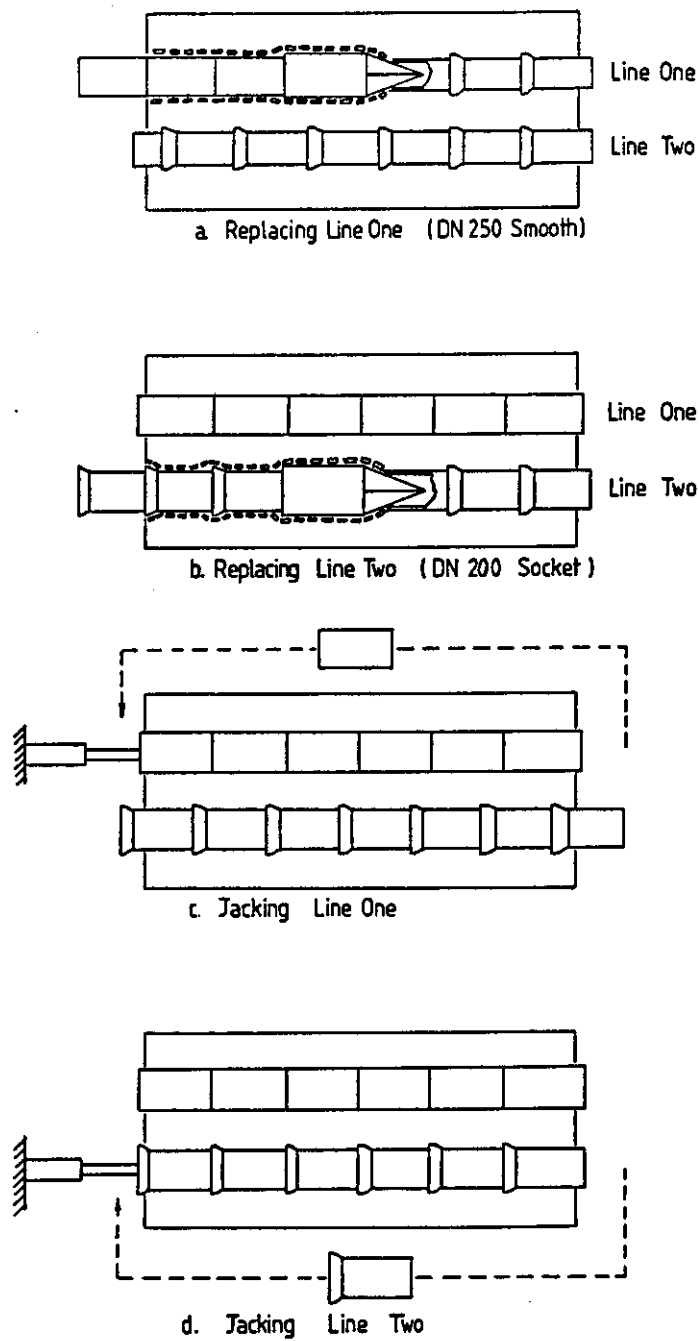


Figure 4.3

The Four Stages of the Laboratory Trial

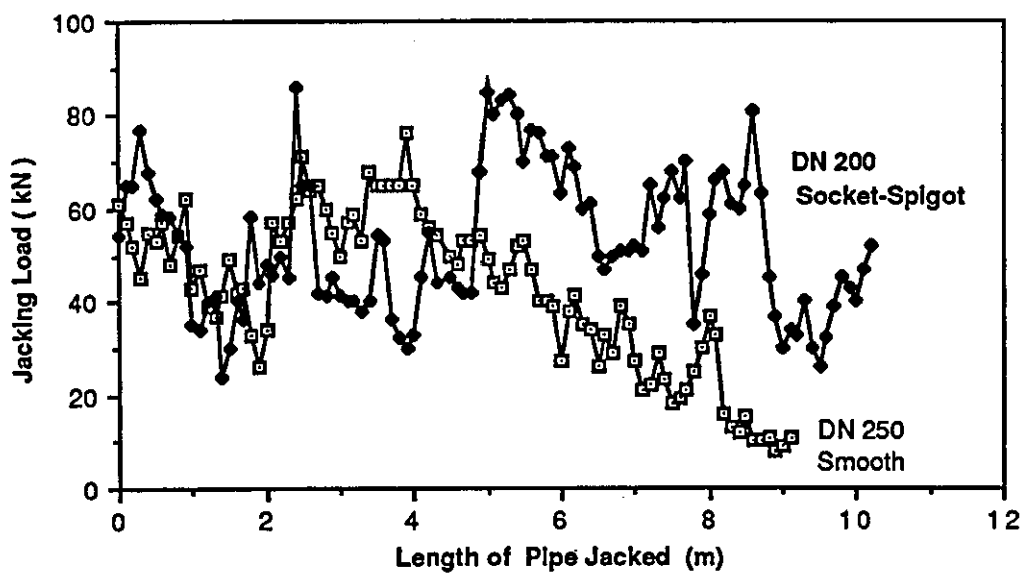


Figure 4.4 Jacking Loads for Lines 1 and 2 (laboratory trial stages c and d)

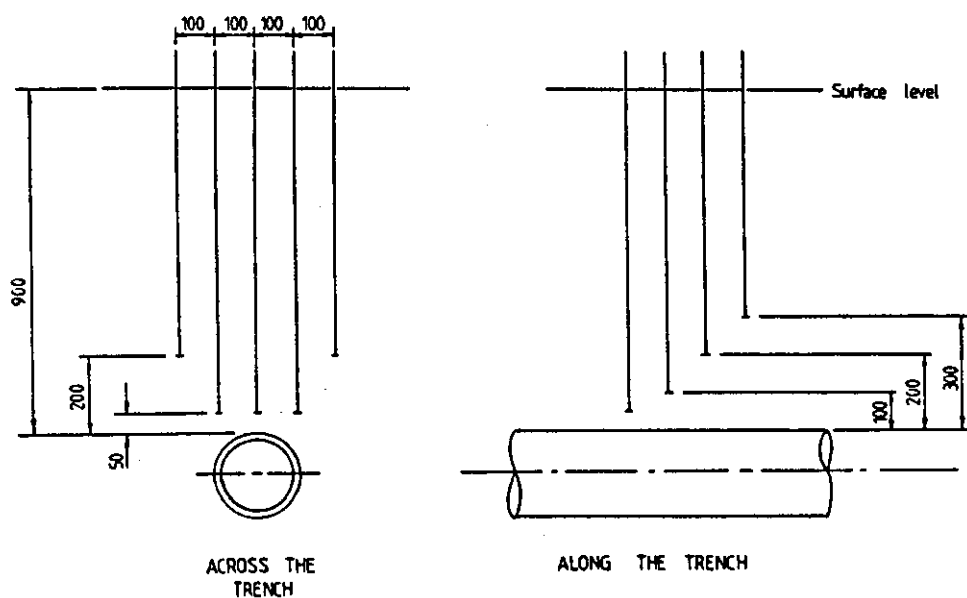


Figure 4.5 Positions of Soil Displacement Rods

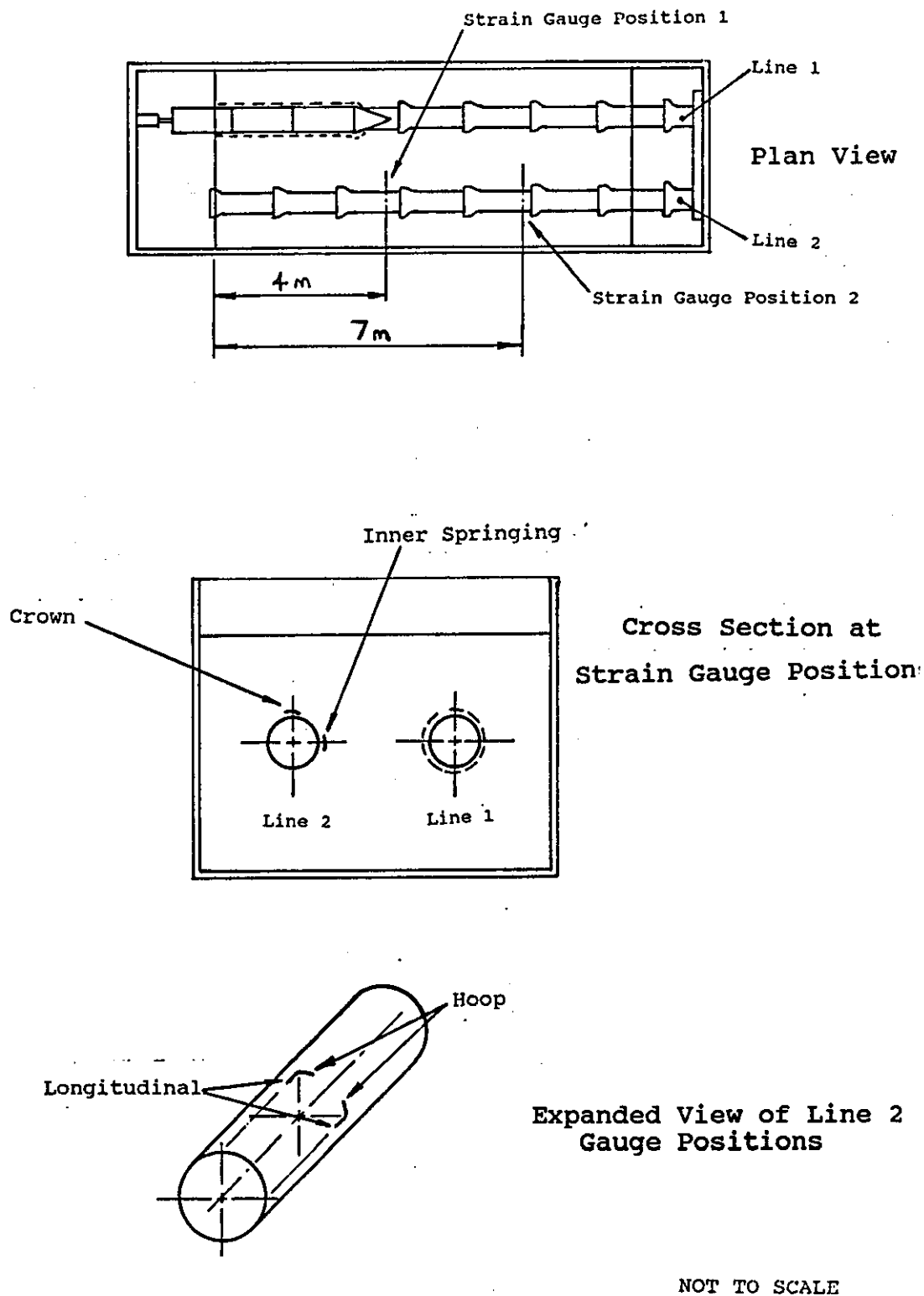
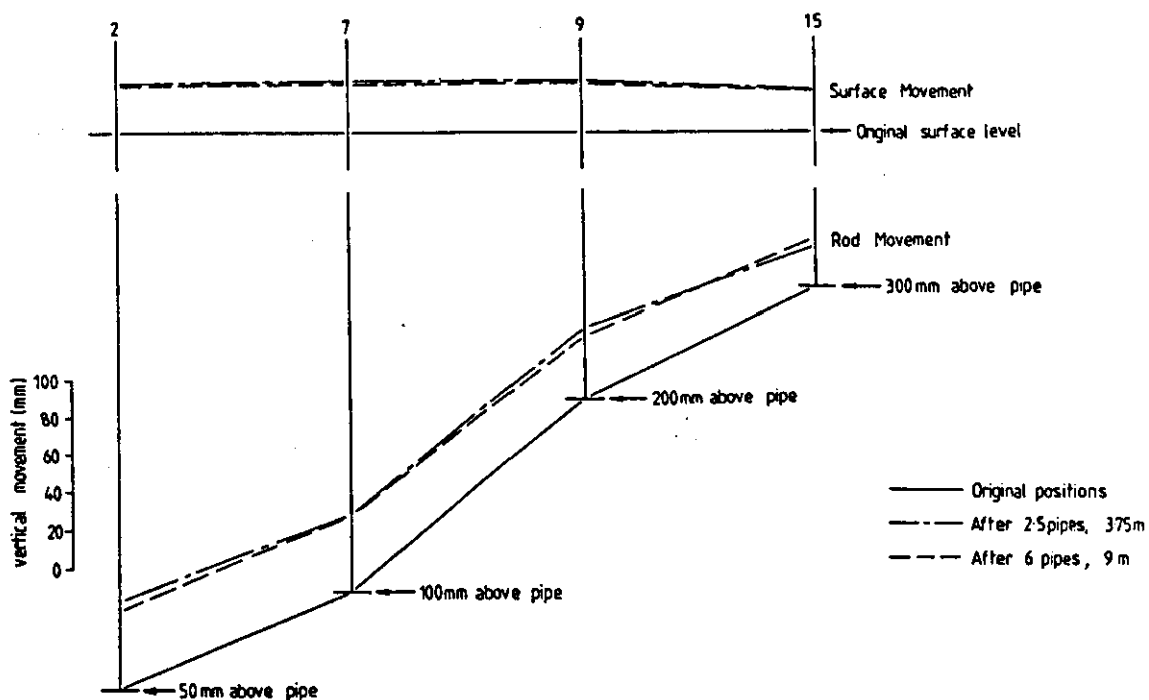
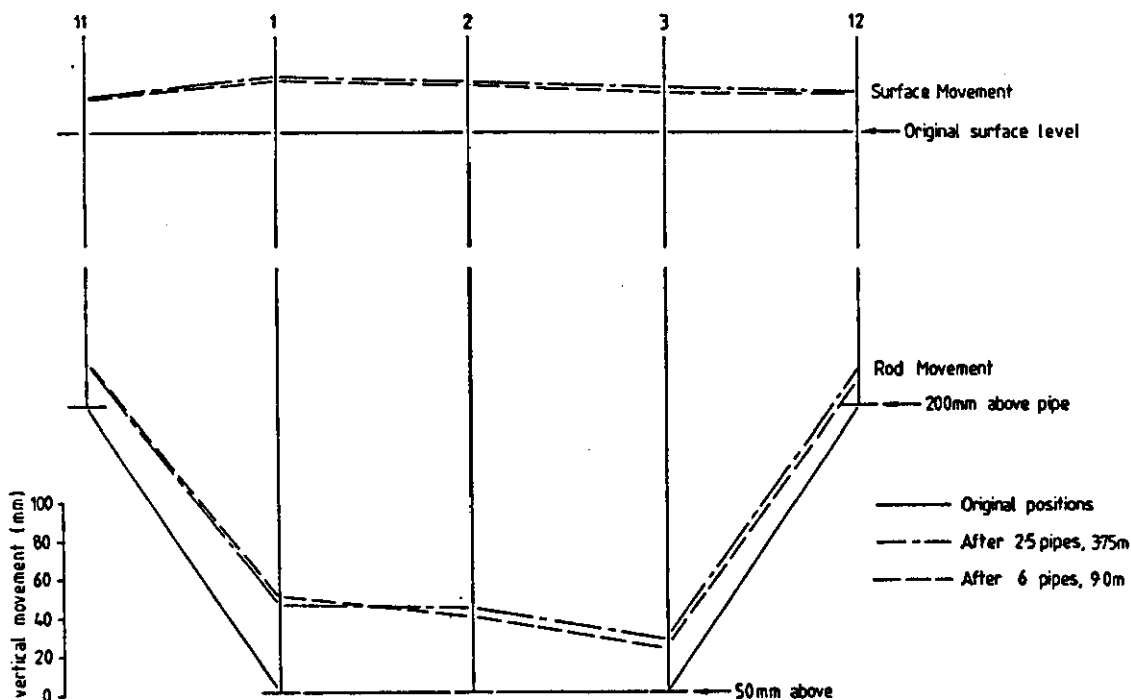


Figure 4.6

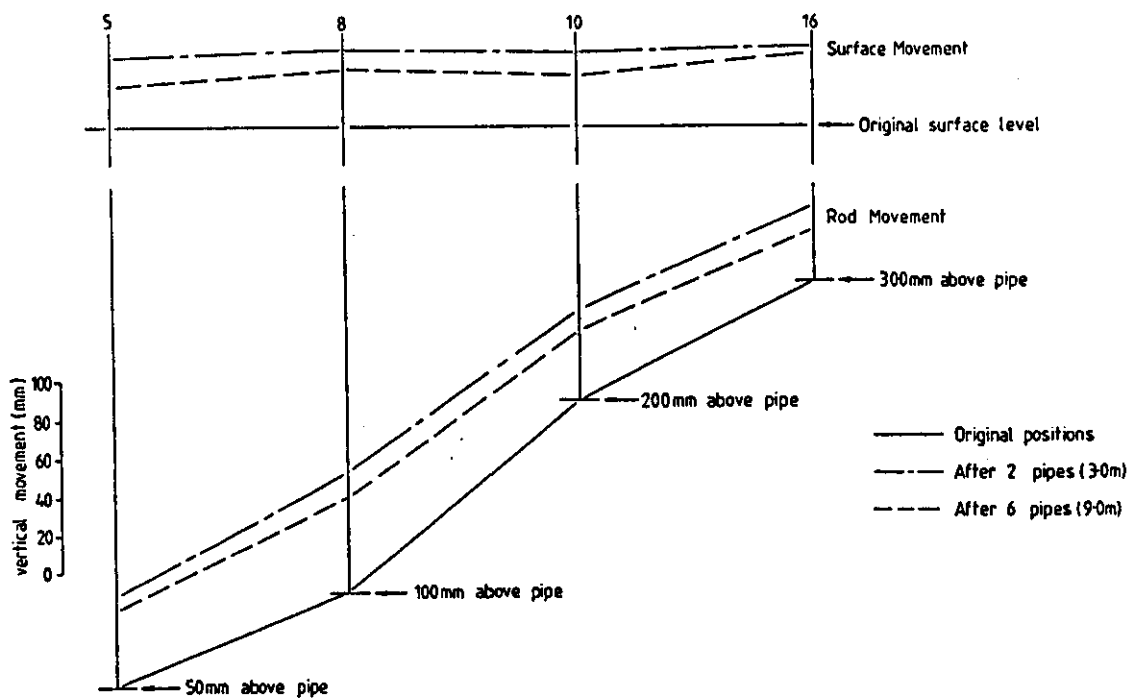
Locations of Strain Gauges



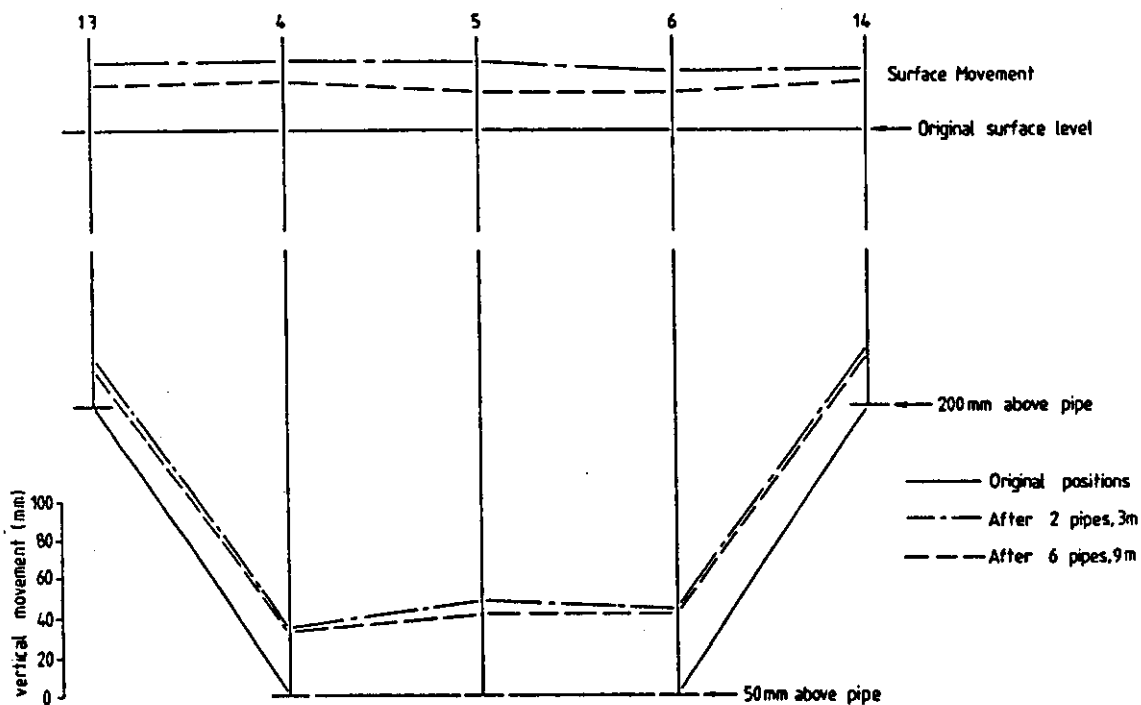
a. Sand movements along the pipe for Line 1



b. Sand movements across the trench for Line 1



a. Sand movements along the pipe for Line 2



b. Sand movements across the trench for Line 2

# Surface Heave

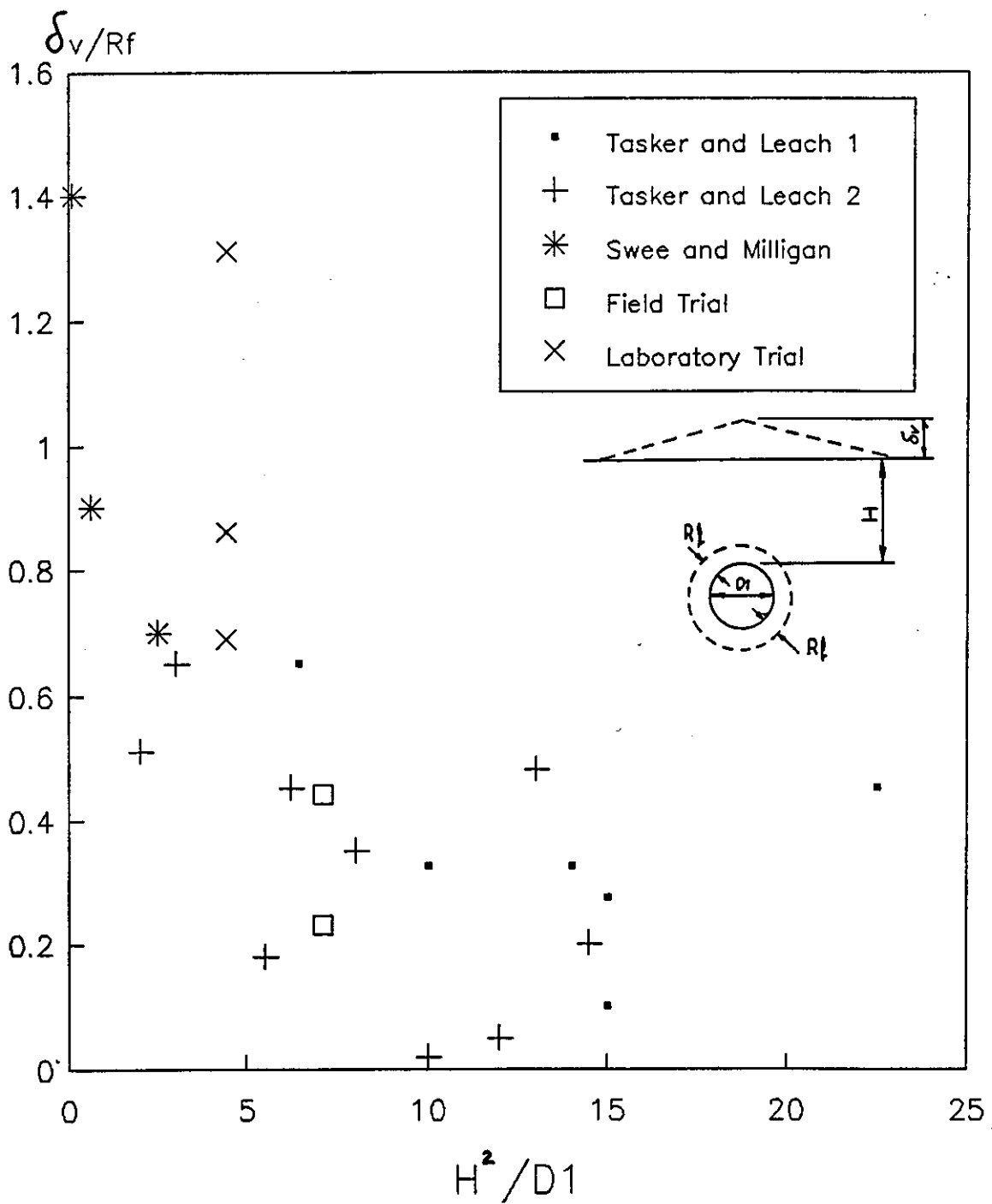


Figure 4.9

Comparisons of Surface Heave

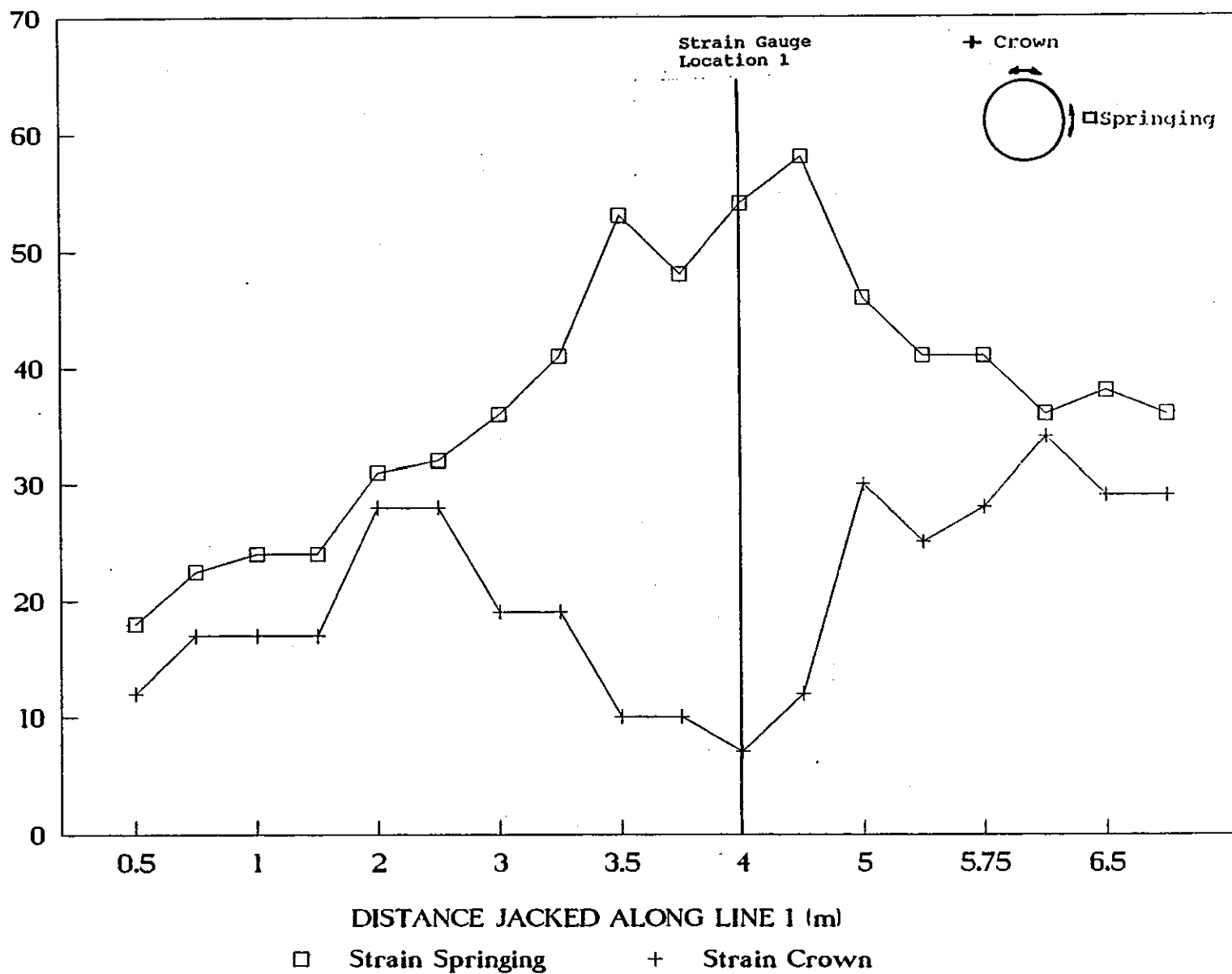
INDUCED HOOP STRAIN IN LINE 2 ( $\mu\epsilon$ )

Figure 4.10

Induced Hoop Strain in Line 2 due to Molding in Line 1

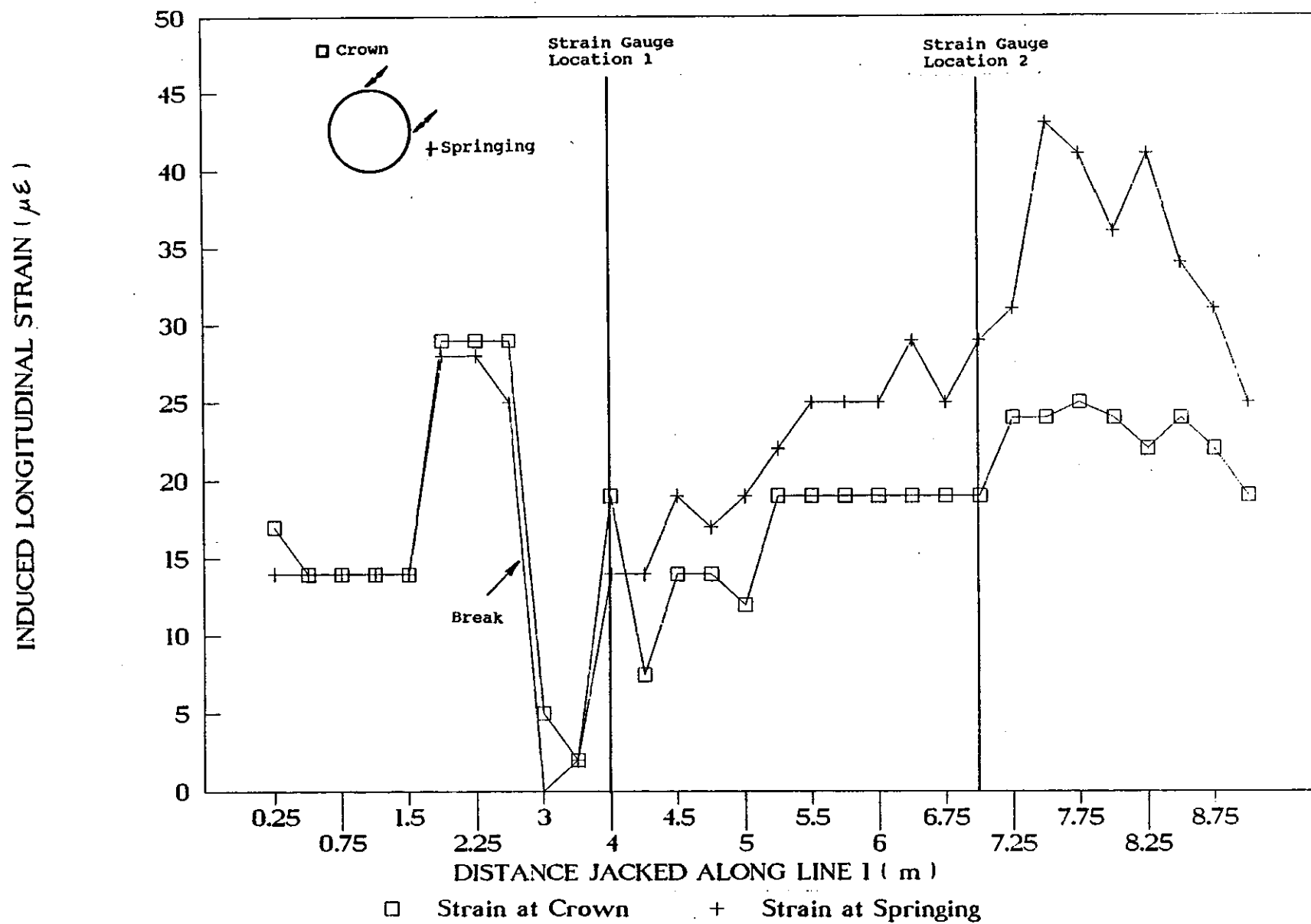


Figure 4.11

Longitudinal Strain in Pipe 3, Line 2

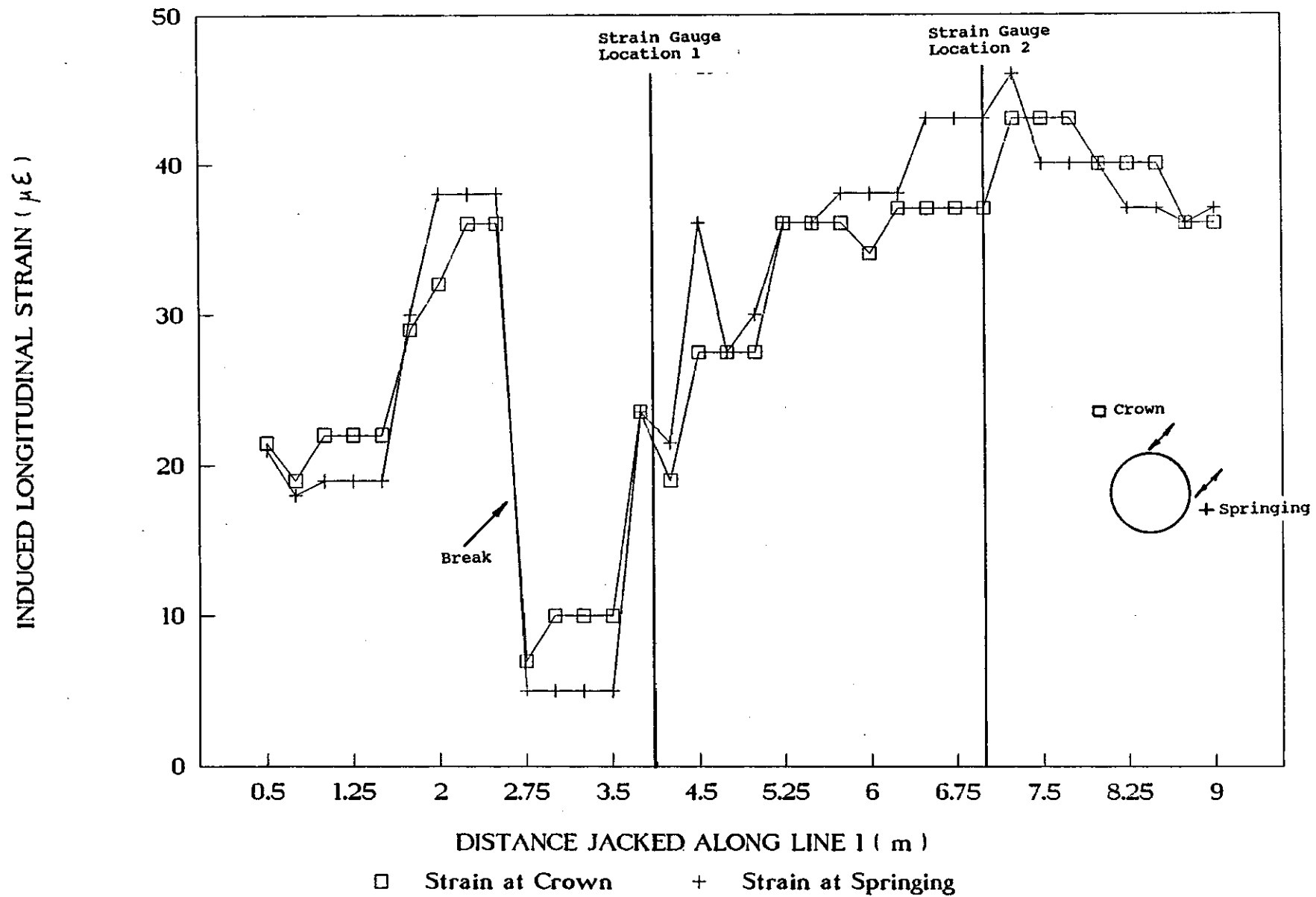


Figure 4.12

Longitudinal Strain in Pipe 5, Line 2

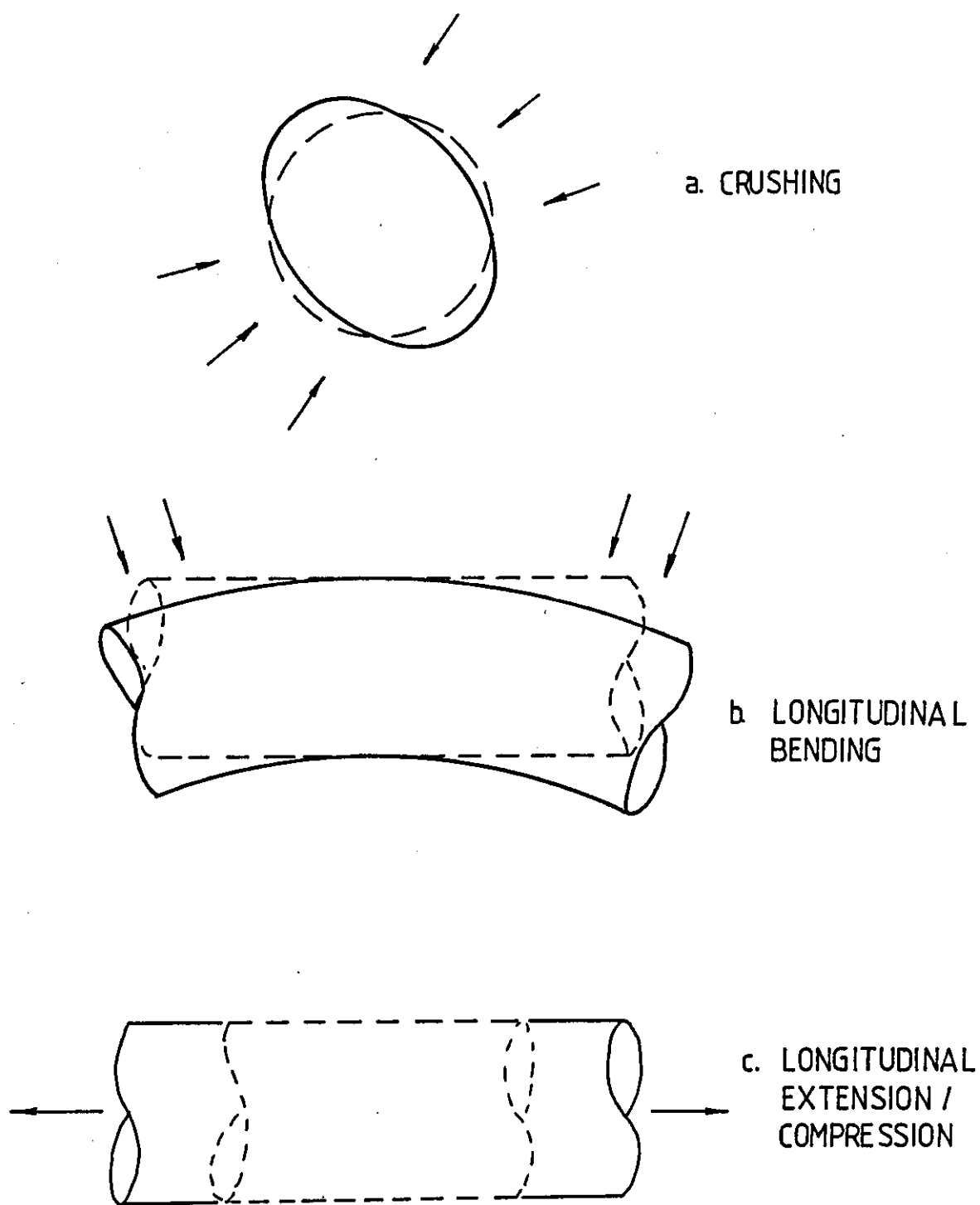


Figure 4.13

Modes of Pipe Deformation

## **CHAPTER FIVE**

### **MECHANICAL PIPE JOINT DESIGN**

## 5.1 INTRODUCTION

The requirements of a new ductile iron pipe for use with pipe bursting were discussed in Chapter 3. These requirements clearly have a controlling influence over the design of the new pipe. Of particular importance is the requirement that such pipe should be available in lengths of 1.5 to 3.0 metres. Standard production of ductile iron pipe, as detailed in Chapter 2, means that pipes are only available in 5.5m lengths. Taking account of production methods the most economical way of producing short length ductile iron pipe is to cut the standard product to the desired length and use a new independent jointing system to connect these short lengths. In order to better understand some of the fundamentals of the pipe bursting process, the trials described in Chapter 4 were carried out with particular emphasis on studying jacking loads and ground movements. In particular, the pipe-soil interaction was discussed and the effects of joint profile on jacking loads were studied. It was established that socket profile joints, as used with standard ductile iron pipe and shown in Figure 4.1, cause jacking loads to increase in comparison with smooth profile pipes. However, it was predicted that the inherent strength of ductile iron is able to withstand the stresses caused by increased load.

In this Chapter, the design and development of the new pipe joint is described. The reason for concentrating on the joint rather than the pipe and the joint, is that the pipe has a high inherent strength and it is only the end of the pipe which is in contact with the joint that requires attention in mechanical terms. The design takes full account of the requirements detailed in Chapter 3, with a profiled joint and newly developed gasket which minimise jacking loads and can be used with pipes of any length. Manufacturing and installation considerations are discussed and the load carrying capacity of the joint is studied in some detail with particular emphasis on contact strains at the joint. Finally, methods of protecting the pipe and joint from corrosion are presented with a number of coating systems described and evaluated.

## 5.2 JOINT DESIGN AND EVALUATION

### 5.2.1 Joint Design

It has already been established that the joint must be capable of forming a watertight coupling between two lengths of ductile iron pipe cut from standard 5.5 metre length pipes. Individual pipe lengths of 2.5m were envisaged for most applications but longer and shorter lengths are considered possible depending upon individual site conditions. In order to ensure that jacking

loads are kept to a minimum, the protrusion of the joint above the bore of the pipe was to be kept to a minimum and the joint was to be suitably profiled to ensure smooth passage through the ground.

The jointing system developed is shown in Figure 5.1 and has the following characteristics:

- (a) ability to be installed from small jacking pits;
- (b) an external profile suitable for jacking through the void created by a pipebursting mole;
- (c) ability to withstand large axial loads, high internal pressures and large external surcharge loadings;
- (d) an external protective coating that will remain intact during installation and during subsequent use;
- (e) a range of diameters from DN 150 to 400; and
- (f) remains sealed at pressures of up to 65 bar under adverse tolerance conditions.

These characteristics match the requirements listed in Chapter 3. Detailed drawings of the joint and gasket are shown in Figures 5.2 and 5.3 respectively. Plates 5.1 and 5.2 show the prototype jointing system.

The internal profile of the collar joint was designed with machining tolerances (Figure 5.2). Such tight tolerances enable the maximum and minimum annulus between the pipe and the joint to be controlled within closer limits than those with as-cast surfaces normally associated with standard ductile iron joints. Consequently, the range of gasket compressions required to achieve a leaktight joint can be obtained by using a smaller gasket. This smaller gasket can then be housed in a joint socket which has a reduced outside diameter by comparison with standard ductile iron sockets. This means that the protrusion at the joint (dimension A on Figure 5.1) is reduced by up to 42% when compared with standard socket-spigot pipes. A short computer program was written to calculate gasket sizes for DN 200 and DN 250 pipe, and this is detailed in Appendix A.

The leading edge of the joint is tapered to ensure that the soil is pushed in a radial direction as the pipes are pushed through the ground. The reduced diameter and the taper both act to reduce the jacking resistance. Axial thrust is transferred via a downstand which also serves to locate the collar. The joint was designed to allow 3° deflection between the collar and each spigot, resulting in a total of 6° between spigots. The maximum working pressure is 60 bar, the design being based on the full range of gasket compression. The minimum

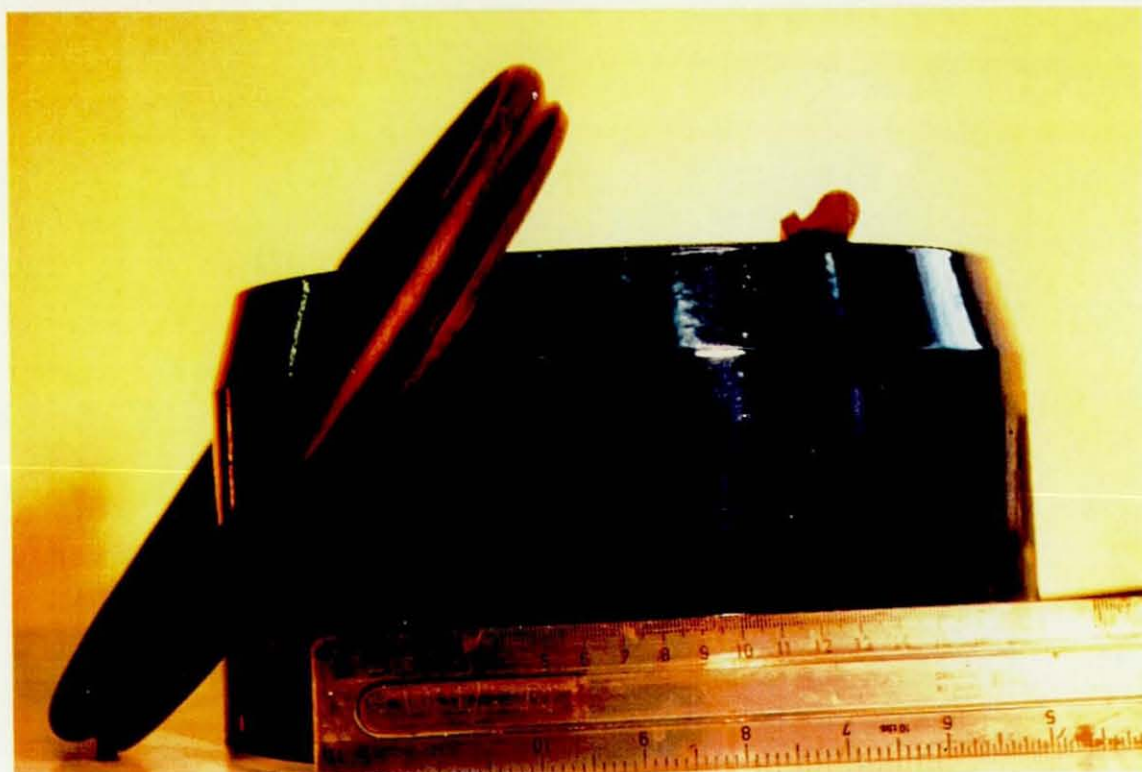


Plate 5.1: Detail of prototype jointing system

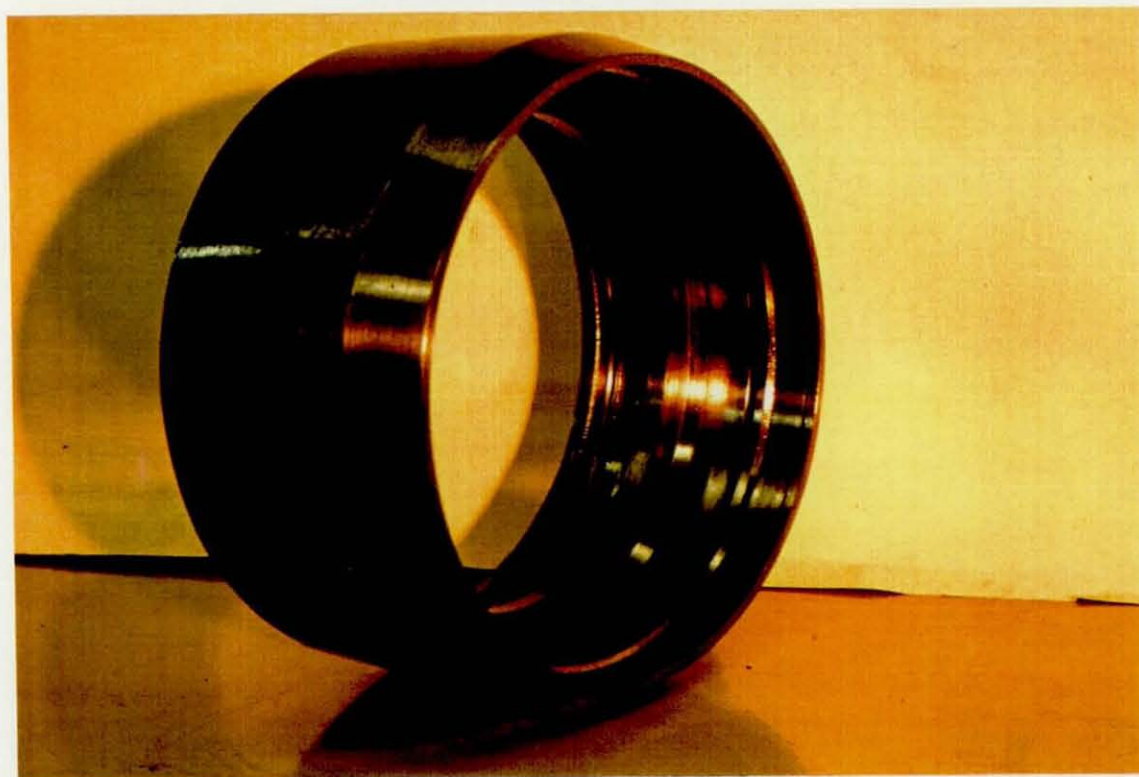


Plate 5.2: Detail of prototype jointing system

gasket compression is based on that required to seal at the maximum pressure whilst the maximum is based on that required to ensure that excessive tensile stresses are not introduced in the rubber which could limit gasket life.

### 5.2.2 Gasket and Jointing Tests

In order to determine the behaviour of the gasket whilst making a joint at the extremes of gasket compression, gasket trials were carried out in the model rig shown in Figure 5.4 and Plate 5.3. A steel plate was machined to the profile of the collar socket and the jointing force was measured and gasket behaviour observed as the spigot plate was introduced into the model rig (Figure 5.4). As shown in Figure 5.5, the jointing load was found to increase until full gasket compression occurred, this being the point at which the spigot taper had travelled past the gasket bulb (Point A, Figure 5.4). Once past this point the jointing load steadily decreased.

Jointing loads were found to be 147% higher for maximum gasket compression than those for minimum gasket compression. Maximum gasket compression conditions were defined as those where a maximum diameter spigot is introduced into a minimum diameter socket with a maximum diameter gasket bulb (all values relating to component manufacturing tolerances). Correspondingly, minimum gasket compression occurs when a minimum diameter spigot is introduced into a maximum diameter socket with minimum diameter gasket bulb. Jointing loads for the model rig are quoted in terms of Newtons per linear millimetre of gasket length (N/mm). This enabled the model loads to be converted into predicted jointing loads for the prototype joint by multiplying model loads by the circumference of the gasket. During testing of the prototype joint, the jointing loads were measured and the results are shown in Figure 5.6. A comparison between the loads predicted by the model and those measured during prototype testing is given in Table 5.1. The results were found to be comparable with those of the model predicting slightly higher loads than those measured on the prototype.

The behaviour of the gasket during jointing was observed during the model rig tests and typical results are shown in Plates 5.4. It can be seen that the heel of the gasket rotates excessively during jointing. This was further confirmed during prototype jointing trials, the results of which are detailed in Table 5.2. It was found that the gasket dislodged when the joint was made under minimum gasket compression conditions. As shown in Plates 5.3 the gasket heel was rotating as the spigot was introduced into the socket. Under maximum compression conditions the annulus between the spigot and socket was small so restricting gasket heel rotation. However, under minimum compression conditions the annulus was sufficiently large to allow

Table 5.1: Comparison between predicted and actual jointing forces

GASKET COMPRESSION	MODEL LOAD N/mm	PREDICTED LOAD FOR DN 200 JOINT kN	ACTUAL LOAD FOR DN 200 PROTOTYPE kN
MAXIMUM 44%	4.7	3.7	3.1
MINIMUM 4%	1.9	1.5	0.8

Table 5.2: Results of prototype jointing trial

TEST NUMBER	SPIGOT DIAMETER (mm)	COLLAR DIAMETER (mm)	ANNULUS MAX / MIN	GASKET DIAMETER (mm)	JOINT * LUBRICATION REFERENCE	RESULT * REFERENCE
J1	223	234.5	MIN	248.3	1	N
J2	223	234.5	MIN	248.3	2	N
J3	223	235		248.3	2	Y
J4	223	234.5	MIN	251.5	2	Y
J5	219	234.5		251.5	2	N
J6	219	235	MAX	251.5	3	Y
J7	219	234.5		251.5	3	N
J8	223	235		252.5	2	Y
J9	219	235	MAX	252.5	2	Y
J10	223	234.5	MAX	252.5	2	Y
J11	219	234.5		252.5	2	N
J12	223	234.5	MIN	250.5	2	Y
J13	219	234.5		250.5	2	N
J14	223	235		250.5	2	Y
J15	219	234.5	MAX	250.5	2	Y
J16	223	235		250.5	1	Y
J17	219	235	MIN	250.5	1	Y
J18	223	235		250.5	4	Y
J19	219	235	MIN	250.5	4	Y
J20	223	235		240	2	Y
J21	219	235	MIN	240	2	Y
J22	223	235		240	1	Y
J23	219	235	MIN	240	1	Y
J24	219	235	MIN	241.5	1	Y
J25	223	235		241.5	1	Y
J26	223	235		241.5	5	Y
J27	220.76	235		241.5	2	Y
J28	220.76	234.5		251.5	6	N

\* KEY TO REFERENCES

- 1 LUBRICATION IN GASKET HEEL
- 2 PROPER LUBRICATION AS RECOMMENDED
- 3 LUBRICATION ON SPIGOT ONLY
- 4 LUBRICATION IN WET CONDITIONS
- 5 NO LUBRICATION
- 6 PROPER LUBRICATION WITH COATED SPIGOTS

Y SUCCESSFUL JOINTING  
N UNSUCCESSFUL JOINTING

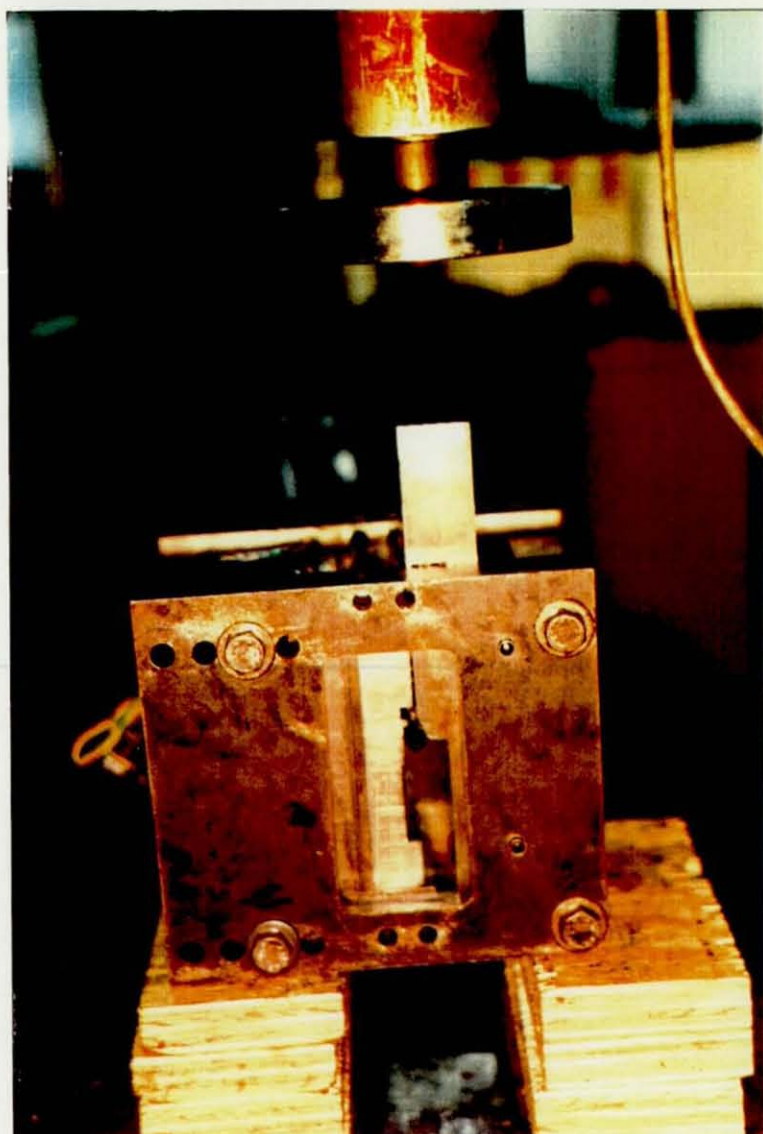
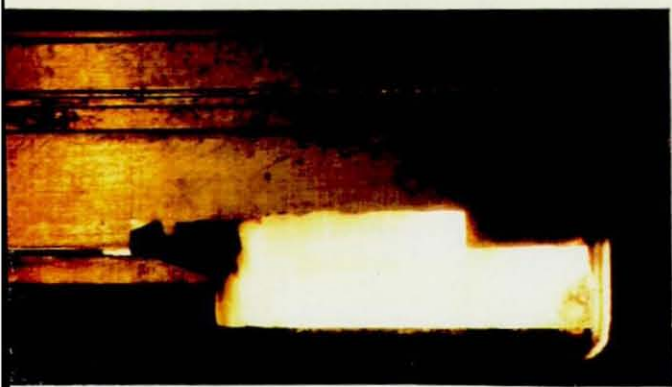
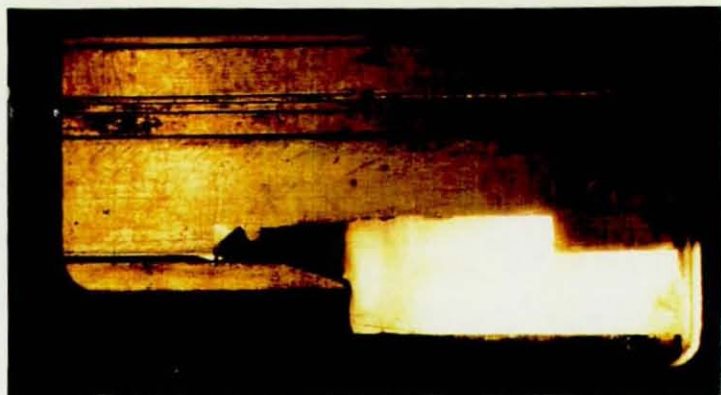


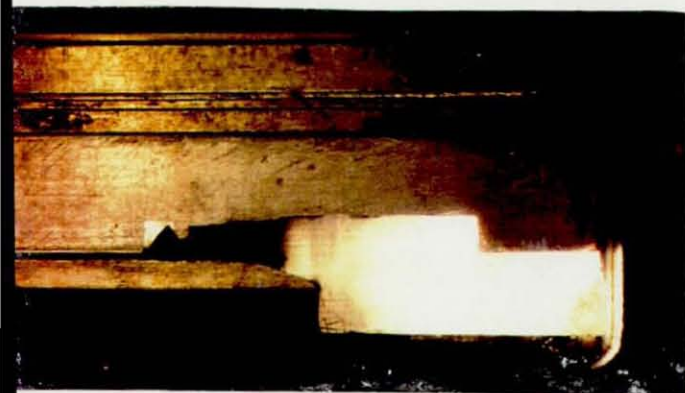
Plate 5.3: Model gasket jointing rig



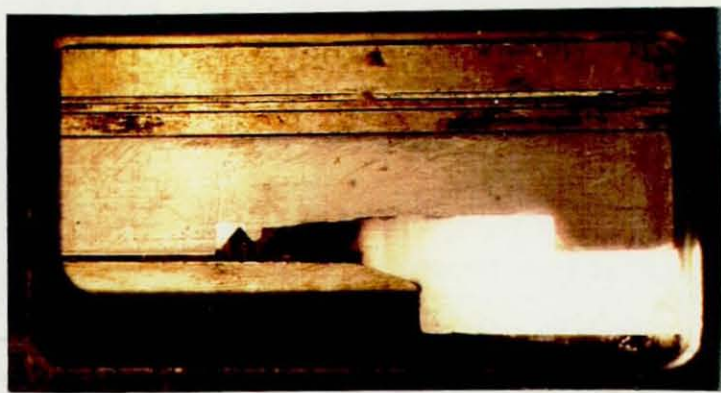
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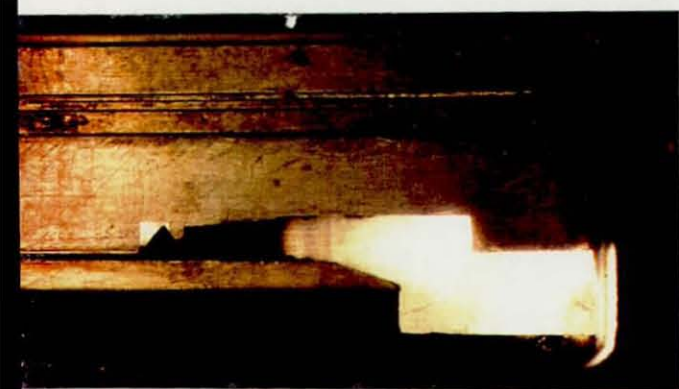
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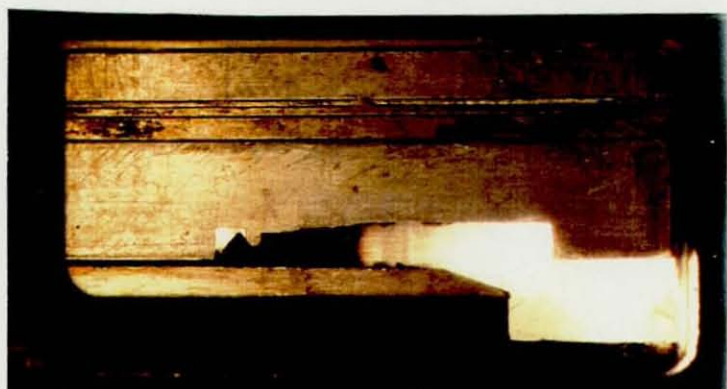
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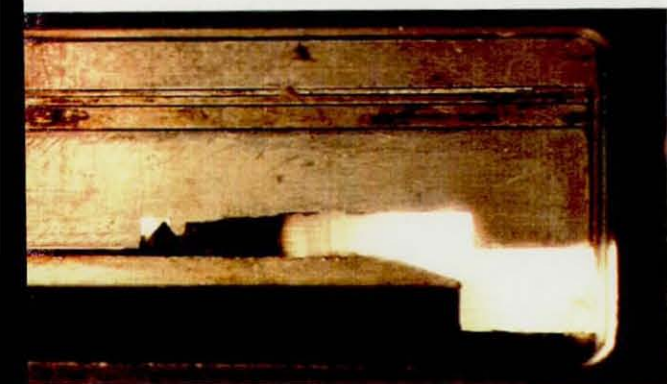
4



5



6



7



8

Plate 5.4: Model gasket trials showing behaviour of gasket during jointing

the heel to rotate out of its groove and hence dislodge the gasket. Consequently, the size and shape of the gasket heel were modified to reduce the amount of heel rotation.

### 5.2.3 Pressure Tests

Hydraulic pressure tests were carried out under maximum and minimum gasket compression conditions. Tests were performed in the aligned, deflected and fully eccentric conditions as shown in Figure 5.7. Hydraulic pressure was applied in three stages. The pipe was filled with water and mains pressure of 4 to 5 bar was applied. The pressure was then increased to 40 bar and held for approximately 20 minutes. Provided that the pressure remained above 40 bar it was then increased to the final test pressure of 65 bar and held for at least three hours. Plates 5.5a and 5.5b show the joint under pressure test in the fully eccentric condition. It was found that at the minimum design gasket compression, leaks sometimes occurred on applying mains pressure. If greater pressure was then applied, the servo or self-sealing effect of the gasket made a satisfactory seal but having leaked at low pressure this is classed as a failure. This problem was overcome by increasing the minimum compression to 7.6%. During pressure testing it was also noted that the heel of the gasket was visible when the spigot was fully eccentric. Figure 5.8 shows the position of the gasket in this condition. Although no leakage occurred during a further series of long-term pressure tests, further testing is required to ensure full long-term stability of the gasket in this condition.

## 5.3 JACKING LOAD TRANSFER

### 5.3.1 Model Joint Axial Load Tests

The ability of the joint to transfer jacking load under deflected conditions was assessed by performing deflected load tests using the arrangement shown in Figure 5.9 and Plate 5.6. The aim of these tests was to determine the degree of spigot yielding at high jacking loads and to this end, the pipes were strain gauged in order to record the increase in strain as the load, and hence stress in the pipe wall, increased. Some local yielding of the pipes was expected since under deflected conditions the load is transferred via a point contact. In order to determine the load carrying capability of the joint, the spigots were machined to a minimum thickness of 4.9mm, the point at which large-scale yielding occurs being of particular interest during these trials. Other important factors included the level of strain away from the contact area, the potential damage to the lining of the pipe, together with the strain level and hence the load at which this damage occurs. Three strain gauges were installed at the contact point, between



A GENERAL VIEW



B DETAILED VIEW

Plate 5.5: Prototype joint under pressure test



Plate 5.6: Deflected load test

the deflected spigot and the collar, in order to measure longitudinal, hoop and 45 degree strain. In addition, longitudinal gauges were installed at 50mm and 170mm from the contact point in order to measure the level of strain away from the contact point. The positions of the strain gauges are shown in Figure 5.10. A total of three tests were performed, during each of which the load was gradually increased and strain readings recorded. Table 5.3 gives a summary of the strains recorded at the maximum load in each of these tests together with the corresponding principal strains and the residual strain once the load was removed.

Figure 5.11 shows the increase in strain with load during Axial Load Test 1 (AL1) and this reveals a number of interesting points. Firstly, the direction of strain is as expected, with the longitudinal gauge at the contact point reading compressive strain and the corresponding hoop gauge reading tensile strain. These strains were seen to increase gradually with load in their respective directions. Secondly, there is little distribution of stress (and hence load) away from the contact point with the longitudinal gauges CH7 and CH9 recording very low strain values. Analysis of the recorded strains at the maximum load of 22.3 kN (refer to Table 5.3) shows that the maximum principal strain at the contact point was  $-1705 \mu\epsilon$  which is the level of strain at which yielding can be expected to start (refer to Table 5.3, Note 2). On removing the load, small residual strains remained in the material confirming that yielding had commenced. The axial load during the second test (AL2) was slightly higher and the maximum principal strain increased to  $-2134 \mu\epsilon$ . Higher residual strains were observed indicating that significantly more yielding of the material occurred, although the values remain low in absolute terms. The results of tests AL1 and AL2 confirm that at a jacking load of approximately 23 kN, yielding of the spigot commences if the joint is deflected to the design limit.

Figure 5.12 shows the increase in strain with load during the third axial load test, (AL3). The strain values at the contact point were seen to increase in their respective directions until the load reached 260 kN. At this load a redistribution of strain at the contact area was observed and the strains then decreased with further increase in load. Also at loads in excess of 260 kN the longitudinal strain at 50mm from the contact point increases sharply with load (Figure 5.12), proving that the yielding at the contact point causes a general spreading of material and consequent redistribution of strain.

The high strains recorded during test AL3 indicate that large deformation of the spigot end had occurred at a load of 260 kN, and this may lead to a decrease in gasket compression which is clearly unacceptable. Further tests were performed with the aim of increasing the load

TABLE 5.3: Strains recorded at contact point during axial load tests

	STRAIN RECORDED AT MAXIMUM AXIAL LOAD		
	MICROSTRAIN ( $\mu\epsilon$ )		
	TEST AL1 22.3kN	TEST AL2 23.2kN	TEST AL3 400kN
MAXIMUM STRAIN CH1, $\epsilon_x$	- 1661	- 2047	- 17538
MAXIMUM STRAIN CH2, $\epsilon_y$	950	1362	3014
MAXIMUM STRAIN CH3, $\epsilon_{45}$	- 698	- 894	- 7603
MAXIMUM PRINCIPAL STRAIN	- 1705	- 2134	- 17544
MAXIMUM PRINCIPAL STRAIN	994	1449	3020
RESIDUAL STRAIN CH1, $\epsilon_x$	- 26	- 177	-
RESIDUAL STRAIN CH2, $\epsilon_y$	21	101	-
RESIDUAL STRAIN CH3, $\epsilon_{45}$	- 28	- 95	-

**NOTES:** 1.  $\epsilon_x$  = Longitudinal strain  
 $\epsilon_y$  = Hoop strain  
 $\epsilon_{45}$  = 45 degree strain

2. for the grade of ductile iron used, yielding is expected to commence at approximately 1700  $\mu\epsilon$ , as detailed below:-

Grade of Ductile Iron Pipe Material: BS 2789 420/12

Predicted yield stress of Pipe Material  $\sigma_y = 289 \text{ MN/m}^2$

Youngs Modulus of Pipe Material  $E = 169 \text{ GN/m}^2$

$$\begin{aligned}
 \text{Predicted yield strain } \epsilon_y &= \frac{\sigma_y}{E} \\
 &= \frac{289 \times 10^6}{169 \times 10^9} \\
 \epsilon_y &= 1710 \mu\epsilon
 \end{aligned}$$

carrying capability of the joint. This was achieved by reducing the length of the spigot taper and so increasing the load carrying area at the end of the spigot. The tests were directed at optimising the length of taper to give satisfactory jointing and axial load carrying capability. By reducing the taper length from 10mm to 6mm, the potential load carrying area was increased by some 79%, whilst the jointing capability was maintained. Consequently, the above tests were repeated with a 6mm spigot taper and it was found that the yielding of the spigot end when loaded up to 400 kN was reduced to acceptable levels. Although at this load there was some cracking of the cement mortar lining, the damage was considered acceptable. Further increases in jacking load carrying capability are predicted by using thicker K12 pipes and this has the added advantage of avoiding potential damage to the cement mortar lining. The performance of K12 thickness pipes is assessed and discussed in the following section.

### 5.3.2 Full Length Axial Load Tests

Having studied the behaviour of the joint under axial load (as described in the previous section), the ability of full length pipes to withstand axial load was investigated by performing axial load tests on 2.5 metre long pipes in conjunction with the new joint. The test layout is shown in Figure 5.13 and the positions of strain gauges are indicated in Figure 5.14.

Considering the high levels of strain recorded during the model joint tests, thicker K12 class pipe was used for this series of trials. As shown in Figure 5.15, this has a nominal ductile iron thickness of 8.4mm which is 31% thicker than the K9 class pipe used in the model joint tests. A total of six tests were performed as detailed in Table 5.4; tests FAL 1 to FAL 4 studied the strain induced in the pipe barrel of pipe 2 and the levels of strains recorded at the joint under increasing axial loads; test FAL 5 was a pressure test to determine the effect of axial load on the sealing capability of the joint and test FAL 6 was a final axial load test to determine the failure load.

Figures 5.16, 5.17 and 5.18 show the increase in strain whilst loading to 100 kN, 300 kN and 450 kN respectively (Tests FAL 1, FAL 2 and FAL 3). In all cases the direction of strain in Pipe 2 was consistent with that recorded during the model joint tests with the longitudinal strain at the contact point showing the highest compressive values and this being associated with tensile hoop strain on the inside of the pipe wall.

In general, when loading to 300 kN the increase in strain with pipe end load is linear (Figure 5.17) and the slight irregularities in the graph in Figure 5.16 are due to a settling effect during this initial loading. The level of longitudinal strain at the contact point is significantly less than

TABLE 5.4

FULL LENGTH AXIAL LOAD TESTS

TEST NUMBER	MAXIMUM AXIAL LOAD (kN)	PURPOSE
FAL 1	100	Monitor pipe and joint strains at low loads
FAL 2	300	Monitor pipe and joint strains at increased loading
FAL 3	450	Monitor pipe and joint strains at high loads and assess risk of buckling
FAL 4	400	Cyclic load test
FAL 5	-	Pressure test to assess joint leaktightness after axial load tests
FAL 6	TO FAIL	Determine failure load

that recorded during the model axial load tests due to the increased pipe thickness. This is confirmed when calculating the principal strains at the contact point which shows that at 150 kN the maximum principal strain was - 960  $\mu\epsilon$  compared with - 4608  $\mu\epsilon$  during the model joint tests.

During test FAL 3 there is evidence of yielding of the spigot at a load of 400 kN and longitudinal strain of - 1845  $\mu\epsilon$  (gauge 101) since the graph deviates from the linear relationship of pipe end load versus strain. Furthermore, at a load of 450 kN the maximum principal strain was - 1936  $\mu\epsilon$  which compares with the predicted a yield strain of approximately - 1700  $\mu\epsilon$  and this gives further indication that yielding had commenced.

The hoop strains recorded on each side of the joint (gauges CH 113 and CH 115, Figure 5.14) were of similar values but were of opposite sense, indicating different behaviour on each side of the joint. The positive hoop strain in gauge CH 113 was probably due to the spigot of Pipe 2 (Figure 5.13) being forced oval due to the action of Pipe 1 and the reaction against the support. The strains recorded in the barrel of Pipe 2 confirm that the average longitudinal strain can be converted into axial load by calculating the stress using Youngs Modulus and Poissons Ratio then axial load by multiplying by the pipe area. Table 5.5 compares calculated loads with measured loads with good agreement between the two. The strain records show that Pipe 2 was subjected to longitudinal bending since the strain recorded in gauge CH 106 was tensile whilst that recorded in CH 102 was compressive. Gauge CH 106 continued to record tensile strain until the load reached 350 kN in test FAL 3, after which compressive stress was recorded. Comparison of the average longitudinal strain in the barrel of Pipe 2 with the longitudinal strain at the contact point, show the strain concentration at the pipe joint is in the order of 3 to 4. This emphasises the importance of the design of load transfer at the joint.

The results of the cyclic load test FAL 4 are presented in Table 5.6. These results show there is little change in longitudinal strain due to cyclic loading up to 400 kN. This indicates that at this load there is little yielding of the spigot ends at the contact point and this is further confirmed by the maximum principal strain values which remain constant at approximately - 1800  $\mu\epsilon$  for each of the four cyclic load tests.

Having completed the cyclic axial load tests the pipes and joints were pressure tested to 20 bar. The aim of this test was to determine whether any yielding at the spigot had caused sufficient damage to sacrifice the sealing capability of the joint. At 20 bar both joints were found to be leaktight.

Table 5.5 Comparison of axial loads calculated from the measured strain and those recorded during full length axial load tests

RECORDED LOAD (kN)	AVERAGE LONGITUDINAL STRAIN ( $\mu\epsilon$ )	AVERAGE HOOP STRAIN ( $\mu\epsilon$ )	LONGITUDINAL STRESS (MN/m <sup>2</sup> ) <sup>*</sup>	CALCULATED LOAD (kN)
25	-30	25	-4.23	19
50	-67.5	33.5	-10.6	47
100	-139	51.5	-22.8	101
150	-211	68.5	-35.1	155
196	-282	83.5	-47.4	209
258	-357	101	-60.2	266
298	-410	117	-69	305

\* Formula for Longitudinal Stress

$$\sigma_L = \frac{E}{1 - \nu^2} (\epsilon_L + \nu \epsilon_H)$$

Where  
 $E$  = Youngs Modulus (  $E = 169 \text{ GN/m}^2$  )  
 $\nu$  = Poissons Ratio (  $\nu = 0.275$  )  
 $\epsilon_L$  = Longitudinal Strain  
 $\epsilon_H$  = Hoop Strain  
 $\sigma_L$  = Longitudinal Stress (MN/m<sup>2</sup> )

Table 5.6: Strains recorded during cyclic full length axial load test (FAL4)

LOAD (kN)	MICRO STRAIN							
	FIRST CYCLE		SECOND CYCLE		THIRD CYCLE		FORTH CYCLE	
	Average Longitudinal	Longitudinal at Contact	Average Longitudinal	Longitudinal at Contact	Average Longitudinal	Longitudinal at Contact	Average Longitudinal	Longitudinal at Contact
100	-169	-772	-149	-739	-170	-819	-165	-796
200	-300	-1221	-297	-1225	-299	-1232	-309	-1258
300	-425	-1552	-437	-1559	-443	-1561	-454	-1580
400	-561	-1805	-573	-1800	-534	-1768	-572	-1771

After the pressure test, a final axial load test was applied to determine the failure load of the joint and the increase in strain with load is shown in Figure 5.19. The level of strain and deformation of the spigot end were found to be acceptable at loads up to 500 kN with the maximum principal strain being  $-2008 \mu\epsilon$  at this load. At a load of 600 kN it was not possible to sustain the load at a constant level indicating yielding of the spigot ends. The maximum principal strain at this load was  $-2206 \mu\epsilon$  at the contact point and the longitudinal strain was  $-2110 \mu\epsilon$ . From the above results the maximum jacking load for the joint was set at 500 kN and it was concluded that the maximum principal strain at the contact point should not exceed 0.2% ( $2000 \mu\epsilon$ ) in order to avoid significant spigot yielding.

The effect of increasing the pipe thickness on the maximum allowable jacking load can be seen by comparing the model joint axial load test results with those recorded during the full length axial load test. The maximum allowable jacking load increased from 400 kN with 6mm taper K9 thickness pipe, to 500 kN with 10mm taper K12 thickness pipe. On first assessment this increase in allowable load may not appear significant but the pipe thickness at the end of the taper was only increased from 3mm to 3.65mm between the two types of pipe. Hence an increase in thickness of only 0.65mm gave an increase in jacking capacity of 100 kN. Consequently, higher allowable jacking loads, potentially in excess of 600 kN, are predicted if a 6mm taper were used with K12 thickness pipe.

## 5.4 PROTECTION AGAINST CORROSION

The inherently corrosive nature of ductile iron necessitates a corrosion protection system which will remain effective after the pipes are installed. Standard ductile iron pipes are provided with a metallised zinc spray coating which acts as a sacrificial barrier should the external bitumen coating and polyethylene sleeving become damaged. This standard coating provides a highly effective protection system for the majority of soil conditions in the United Kingdom when laid by traditional open cut methods. However, installation by pipe bursting was shown to remove the polyethylene sleeving and severely damage the bitumen and zinc layers. Consequently, alternative protection methods were studied during the trial described in Chapter 4. A total of six coatings were tested: standard bitumen, bitumen-based epoxy paint, epoxy paint, polyethylene tape wrap, polyethylene sheet wrap and polyurethane. All of these coatings were applied in addition to the standard active protection of metallised zinc. Damage to the socket region was severe in comparison with that of the pipe barrels, suggesting that the sockets gave a degree of protection to the barrel of the pipe. However, this protection was over limited length and the pipes still require adequate protection of their whole length in order to ensure that no corrosion occurs.

The polyurethane coating gave full protection to the pipe barrel, any damage being minor since the scratches did not penetrate the coating fully. On the joint/socket damage was more serious and further shielding is required in order to fully protect this region. The extent of damage to the barrel and socket are shown in Plates 5.7 and 5.8 respectively.

The polyethylene tape wrap performed reasonably well during the trial and although large areas of the coating were ripped, the damage did not penetrate through to the pipe barrel surface. However, the tape wrap on the socket area was completely removed and further protection is required in this area. Plates 5.9 and 5.10 show the damage to this coating.

Epoxy paints are generally considered to be wear resistant. However, these coatings did not perform as well as expected, with large areas of coating being removed. This damage was considered to be due to inappropriate surface preparation prior to applying the coating. The polyethylene sheet wrap was completely removed from the pipe and was therefore considered inappropriate.

Although the polyurethane coating performed well, the cost of this material and the cost of its application to the pipe made the material less attractive. Consequently, alternative materials,



Plate 5.7: Damage to polyurethane coating on barrel

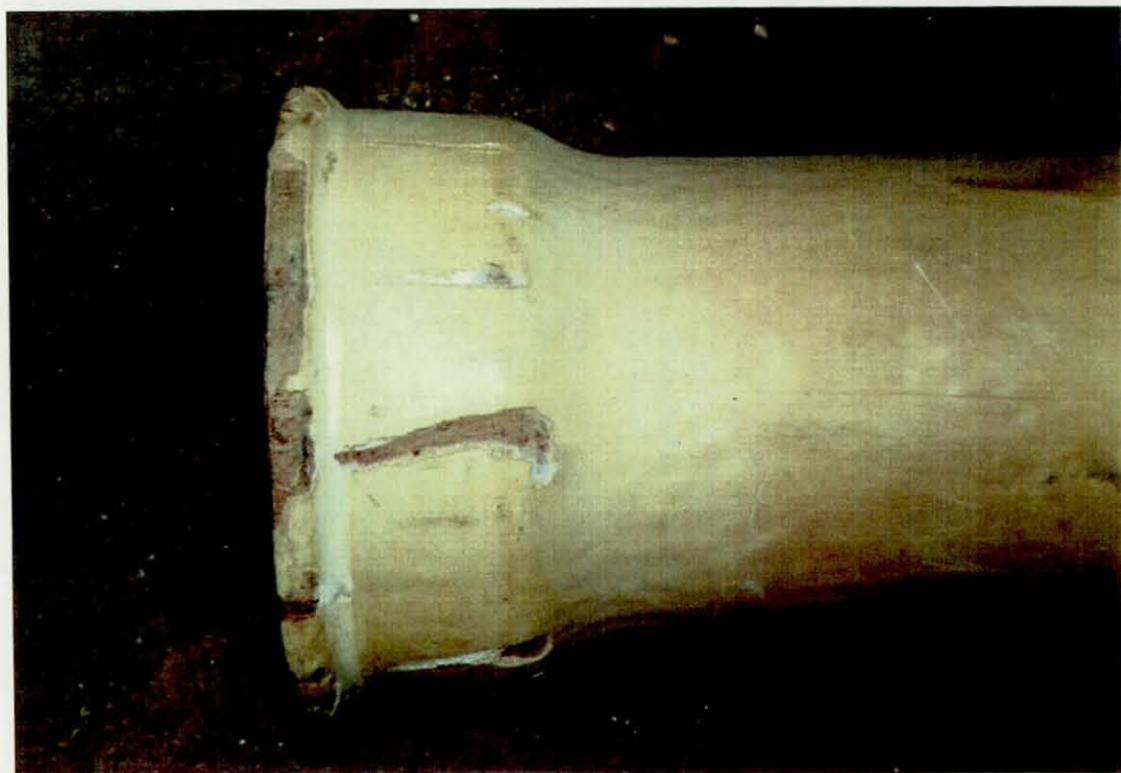


Plate 5.8: Damage to polyurethane coating on socket



Plate 5.9: Damage to tape wrap coating on barrel

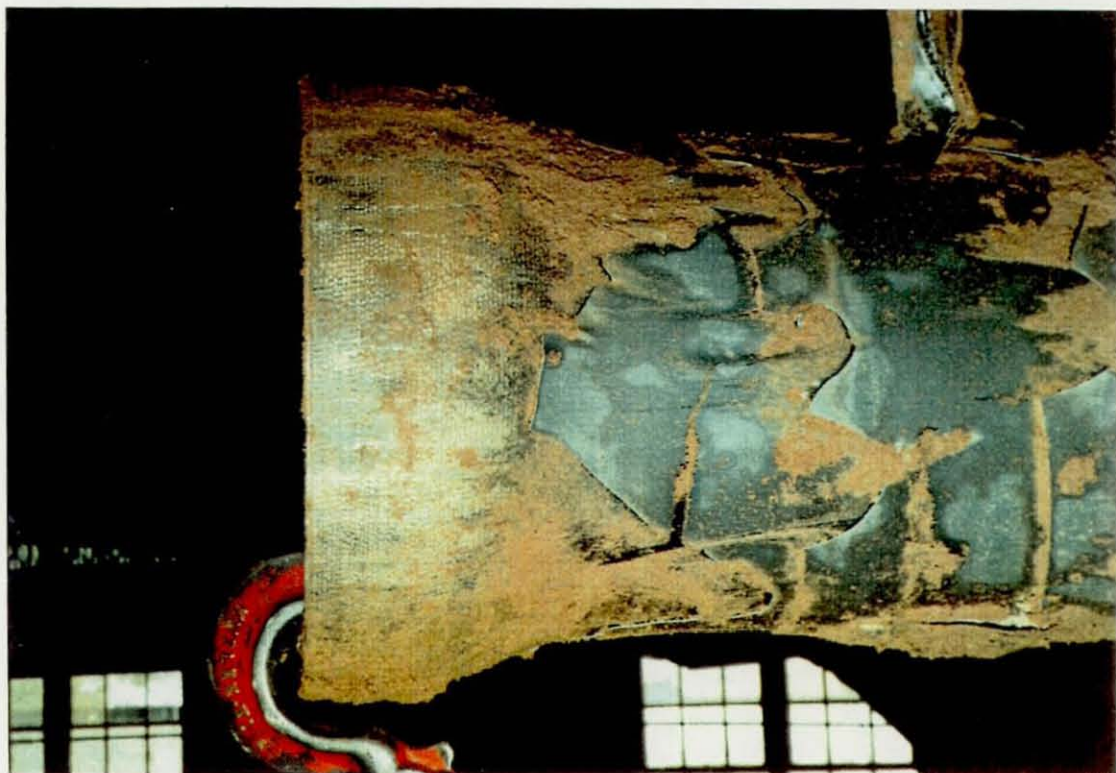


Plate 5.10: Damage to tape wrap coating on socket

consisting of various types of glass reinforced plastic material were investigated. A description of these materials is given in section 2.3.2. Four types of coating using these materials were developed, these being:

- a) polyester resin
- b) polyester resin and sand
- c) polyester resin with one layer of glass fibre rovings (with and without sand)
- d) polyester resin with four layers of glass fibre rovings (with and without sand)

The performance of each of these coatings was compared with that of the polyurethane coating using a specially devised scratch test, details of which are shown in Figure 5.20. This involved pulling the pipe over a scratch tip under varying levels of load. In this test the coating was assessed in terms of the width of scratch obtained at each level of load. The results are shown in Figures 5.21 to 5.24. Each graph shows the width of scratch at the beginning and end of the test, with the load on the scratch tip gradually increasing throughout the test.

The performance of the polyurethane coating is shown in Figure 5.21. The scratch width increases progressively as the load increases, reaching a width of 17mm at a load of 1000 N. The polyester resin coating performance is shown in Figure 5.22, and for this test half of the pipe was coated with polyester resin, impregnated with sand whilst the remainder was coated with polyester resin only. The sand clearly had a detrimental effect on the scratch resistance since the width of scratch without sand was significantly less than that with sand. At a load of 1000 N the scratch width was only 12mm without sand compared to 24mm with sand.

Figures 5.23 and 5.24 show the effect of adding layers of glass fibre rovings to the polyester resin. Again the effect of adding sand was detrimental to the overall performance of this coating. In general, the fibre rovings do not improve the performance of the coating and this was due to the fibre strands catching on the scratch tip and ripping the coating. With both levels of glass fibre rovings, the scratch width was approximately 18mm at 1000 N load.

The collar joint was designed to have a fusion-bonded epoxy coating for resisting internal and external corrosion. This type of coating is used for valves and fittings in the water industry and has a proven record in resisting corrosion. In order to protect the epoxy coating on the collar joint a uPVC shield was developed as shown in Figure 5.25. This was produced by the dip moulding process and was designed to slip over the collar with a small interference fit, thereby ensuring that it remains in place during installation. Another important function of the shield is

to prevent soil ingress into the gasket chamber. The effectiveness of this shield is discussed in Chapter 6.

## 5.5 CONCLUSIONS

A new pipe joint and gasket system has been developed. This new joint is essentially a collar connector into which the two pipe spigots are inserted. The design is such that pipes of any individual length can be used and this length can be made to suit specific site conditions. The collar design also allows a small gasket to be used and, on a DN 200 pipe, this in turn leads to a 42% reduction in protrusion at the joints when compared with the conventional socket/spigot arrangement. The collar has a tapered leading edge to reduce jacking resistance and an external plastic shield to prevent soil infiltration at the joint and reduce the risk of corrosion.

Under the most adverse conditions of tolerance, extensive testing has been carried out to ensure that the gasket will not become dislodged as the joint is assembled. Pressure tests were performed under adverse tolerance conditions and with maximum design deflection and maximum spigot eccentricity. These tests showed that a minimum gasket compression of 7.6% was required in order to achieve a leaktight seal at all pressures up to 65 bar. This is greater than the minimum compression for standard ductile iron joints, the difference being due to the smaller gasket used in the new joint.

The mechanics of load transfer at the joint have been studied in some detail. Under deflected conditions there was little distribution of load away from the contact point and this caused high strain levels at the contact point. With minimum thickness K9 pipe and standard spigot end profile the load carrying capability of the joint was limited to 260 kN. This limits the jacking length to approximately 26m assuming a contact stress of  $14.7 \text{ kN/m}^2$ , which was the maximum recorded during the trial described in Chapter 4. Small changes to the spigot taper were found to significantly increase the load carrying capacity to 400 kN, giving a total jacking length of approximately 40m. However at this load there was some yielding of the spigot end causing damage to the cement mortar lining of the pipe.

The use of thicker K12 pipe greatly reduced the level of strain at the contact point and at a load of 450 kN yielding of the spigot end had only just commenced. At this load there was little damage to the cement mortar lining and the pipes were subsequently pressure tested. Further testing up to 600 kN load showed that the maximum allowable jacking load was 500 kN for the K12 thickness pipe. This gives a jacking distance of 50 - 55m which is sufficient to cross the

majority of obstacles such as motorways or dual carriageways. It was predicted that higher jacking loads would be sustained if the length of the spigot taper were to be reduced from 10mm to 6mm.

A total of seven types of pipe coatings have been assessed. These consisted of polyurethane, polyethylene, epoxy paints and polyester resin, with and without glass reinforcement. The polyurethane, polyethylene tape wrap and unreinforced polyester resin were found to give the most effective protection to the pipe during installation.

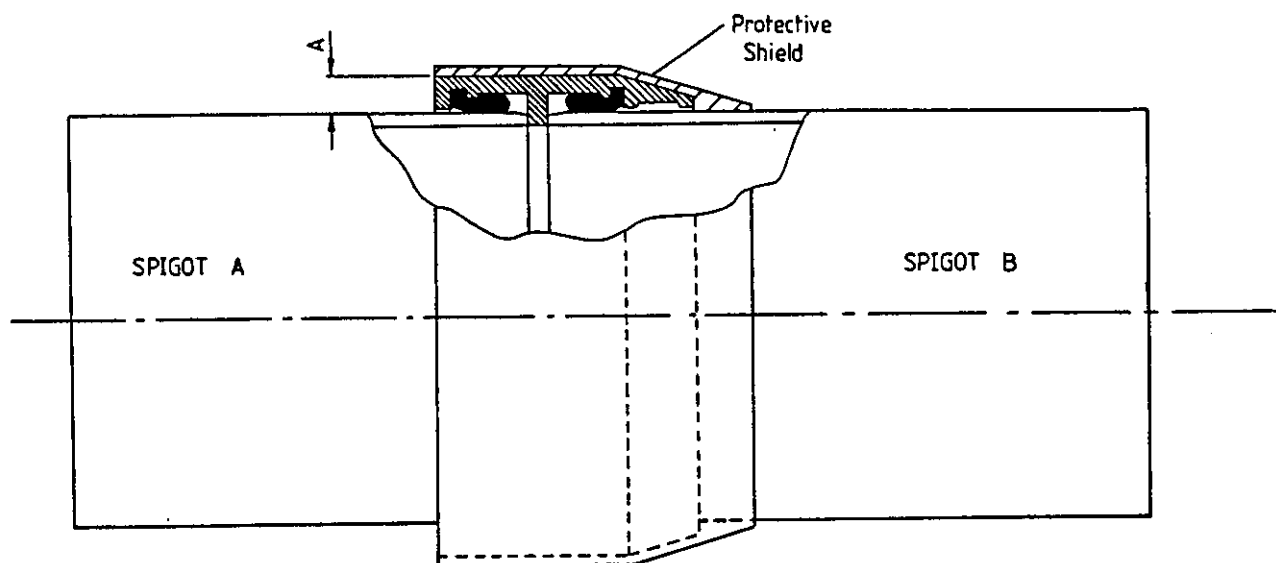


Figure 5.1

Detail of Ductile Iron Pipe Joint for Trenchless Methods

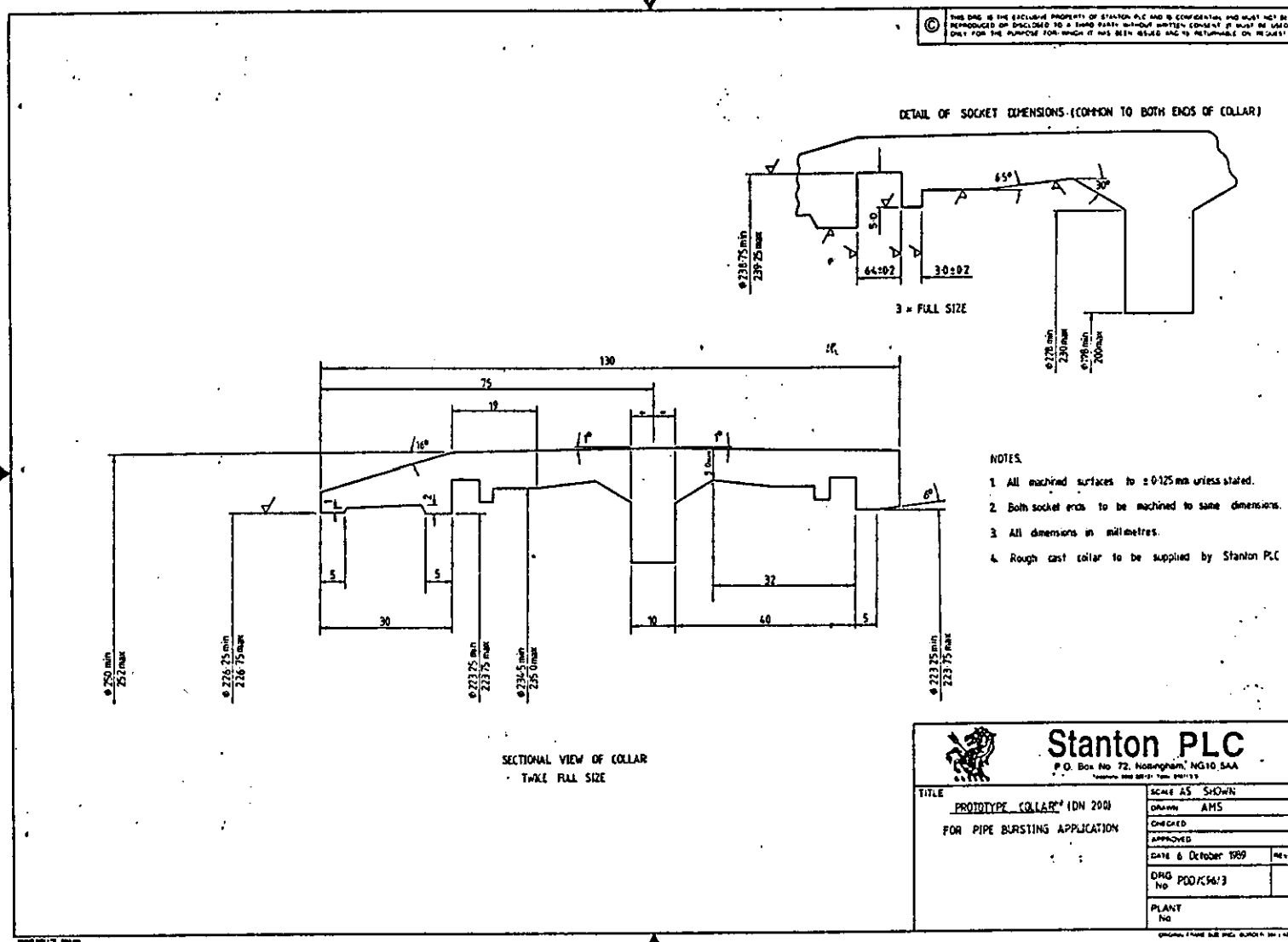


Figure 5.2

Detailed Drawing of the New Joint

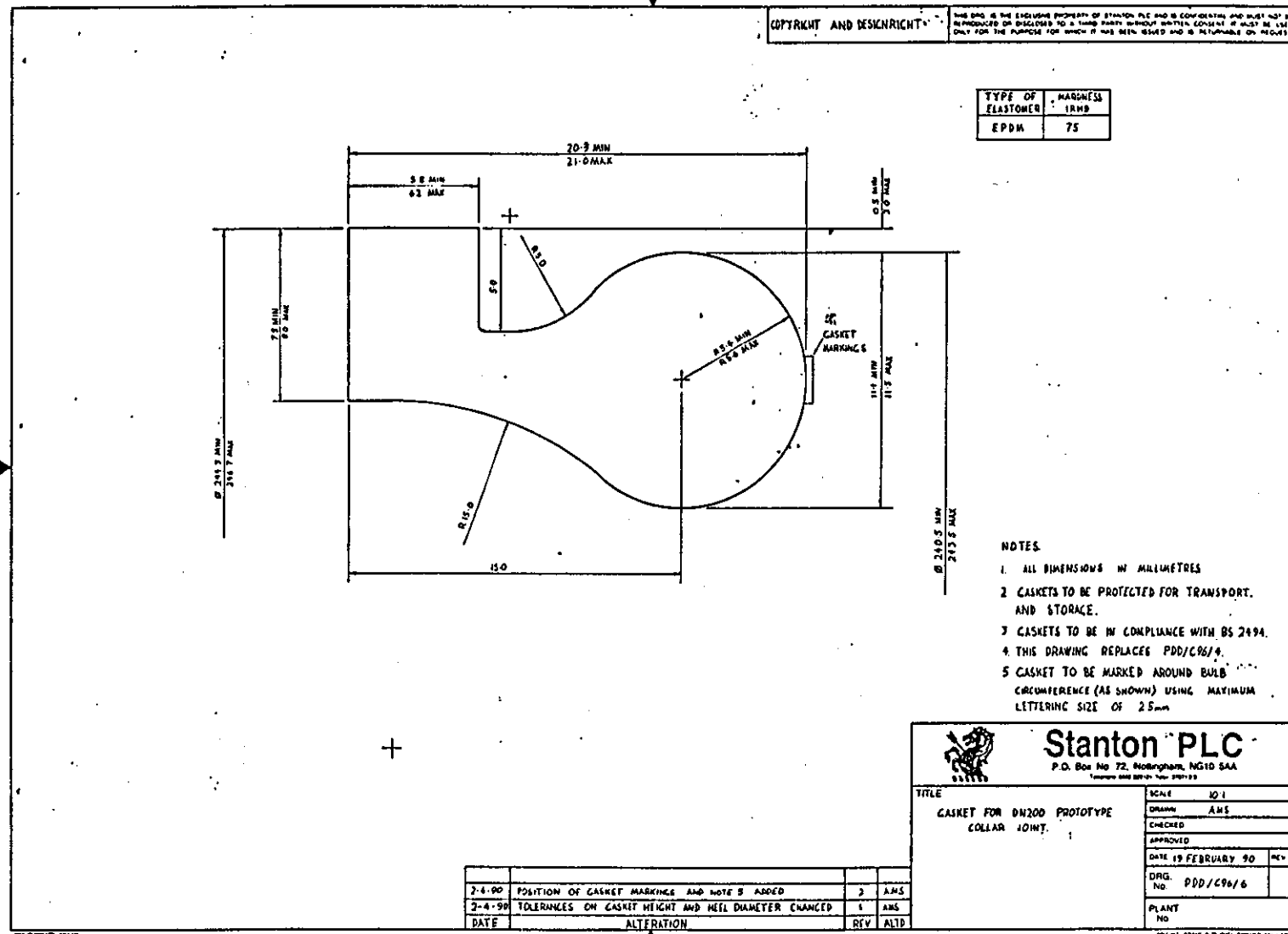


Figure 5.3

Detailed Drawing of the New Gasket

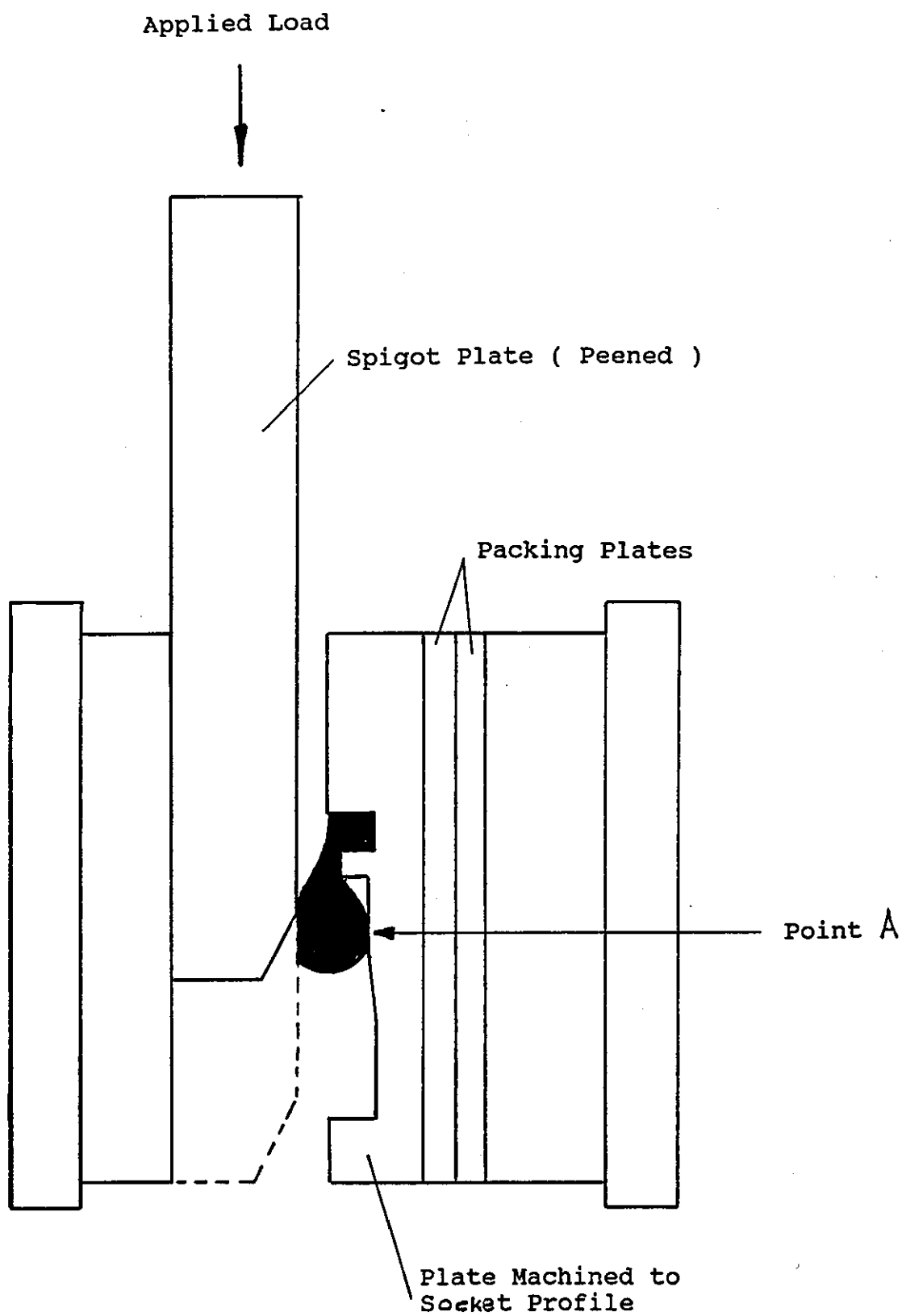


Figure 5.4

Gasket Test Rig

# Jointing Load

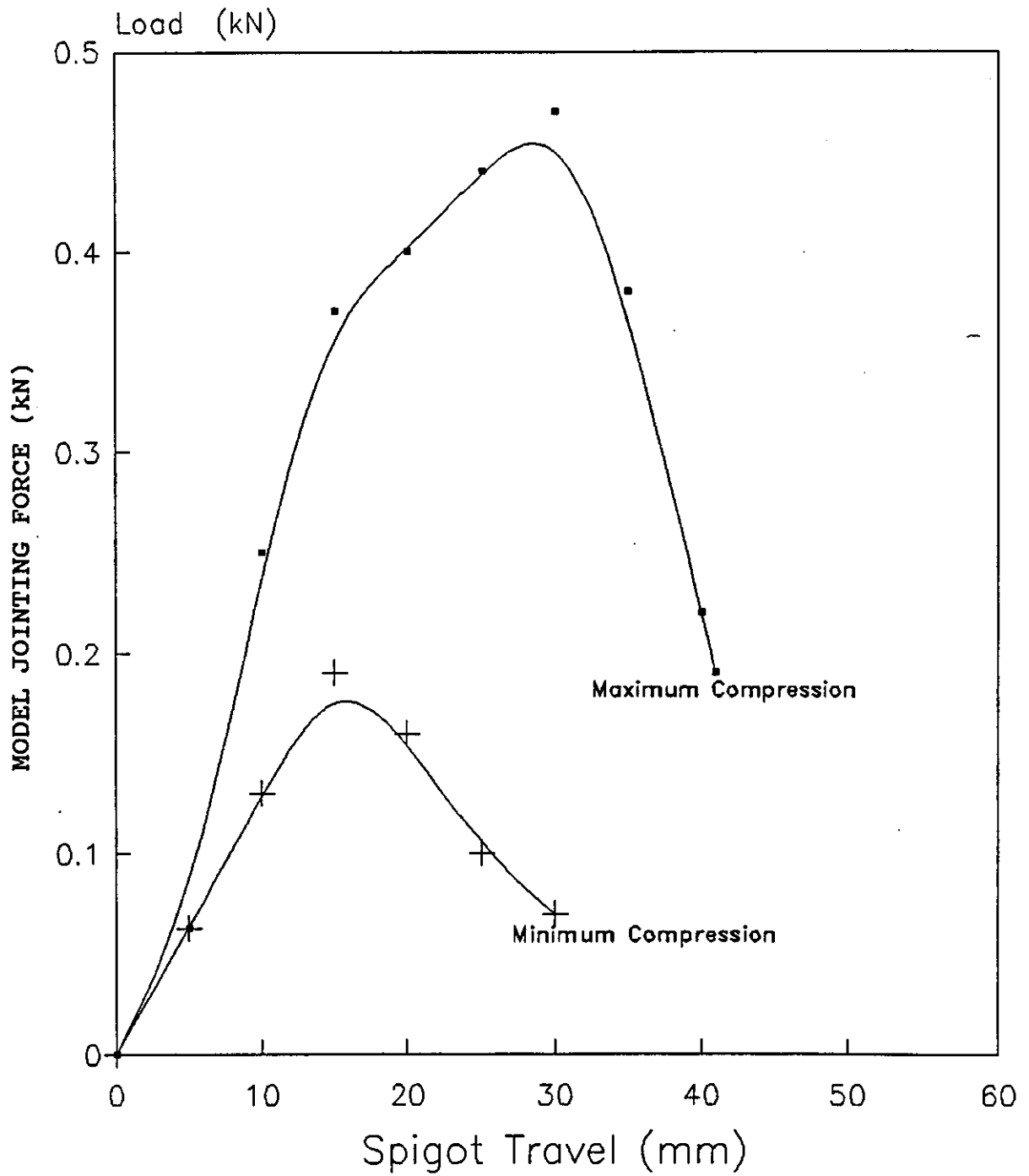


Figure 5.5

Graph of Model Jointing Force Versus Spigot Travel

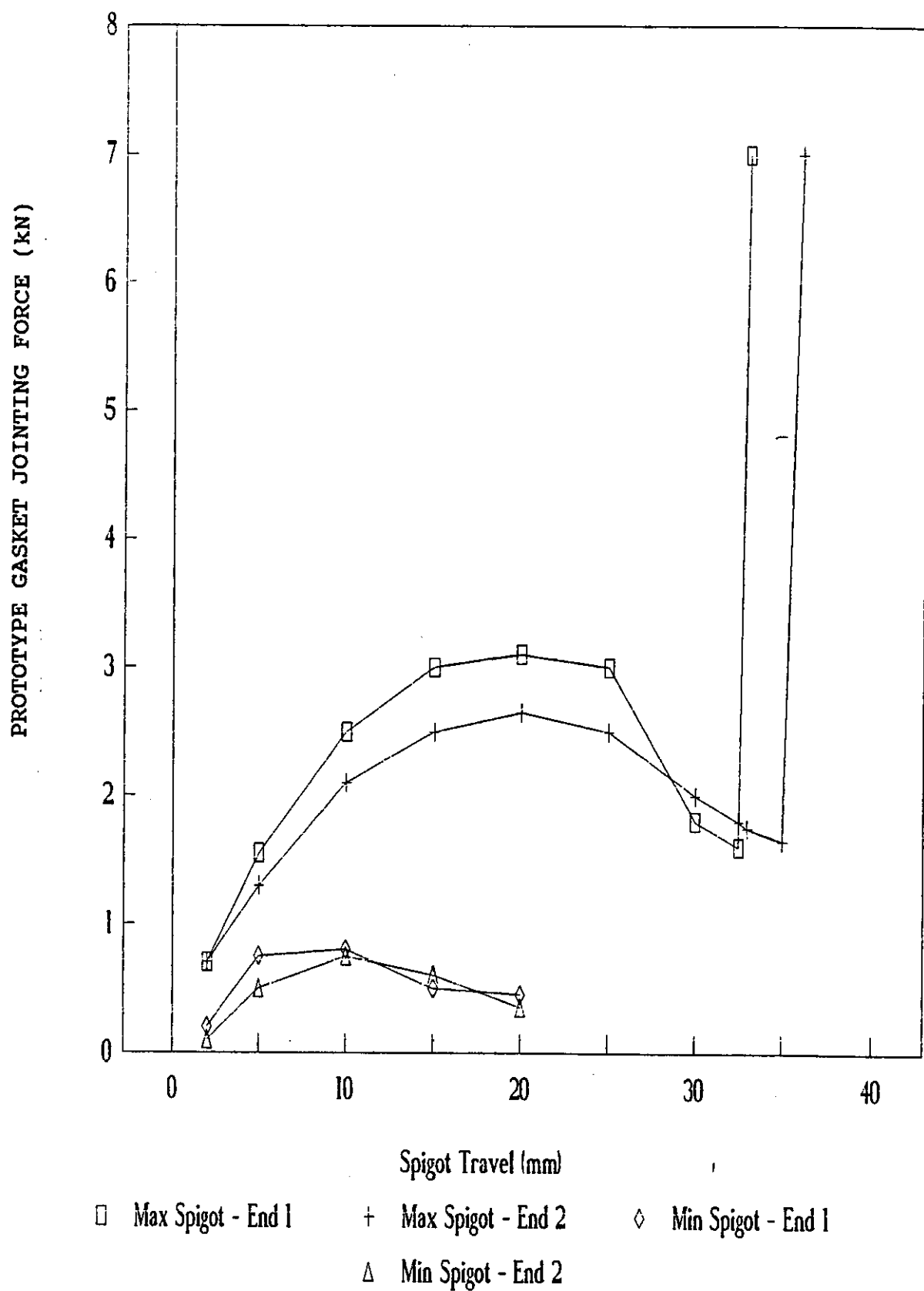
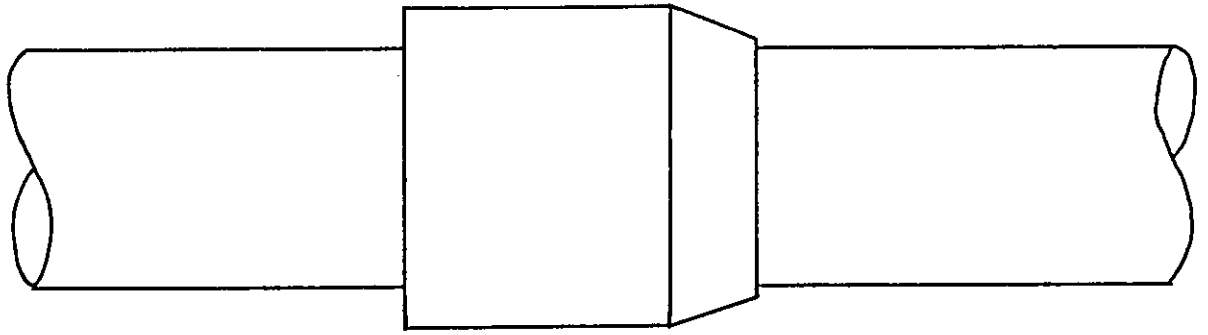
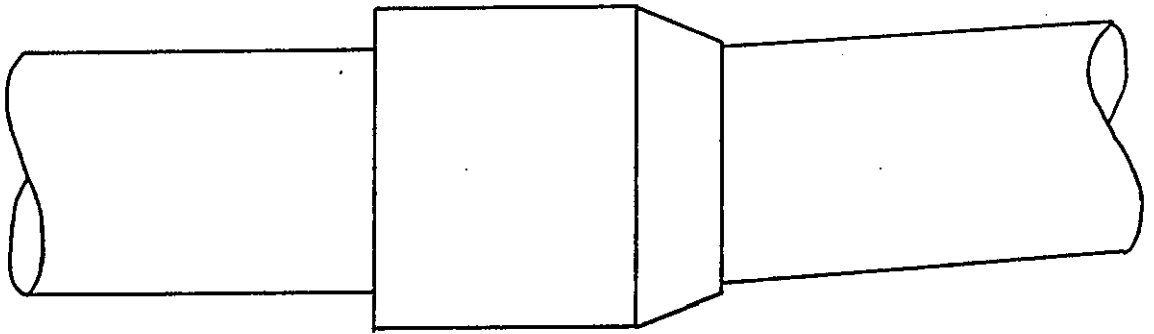


Figure 5.6

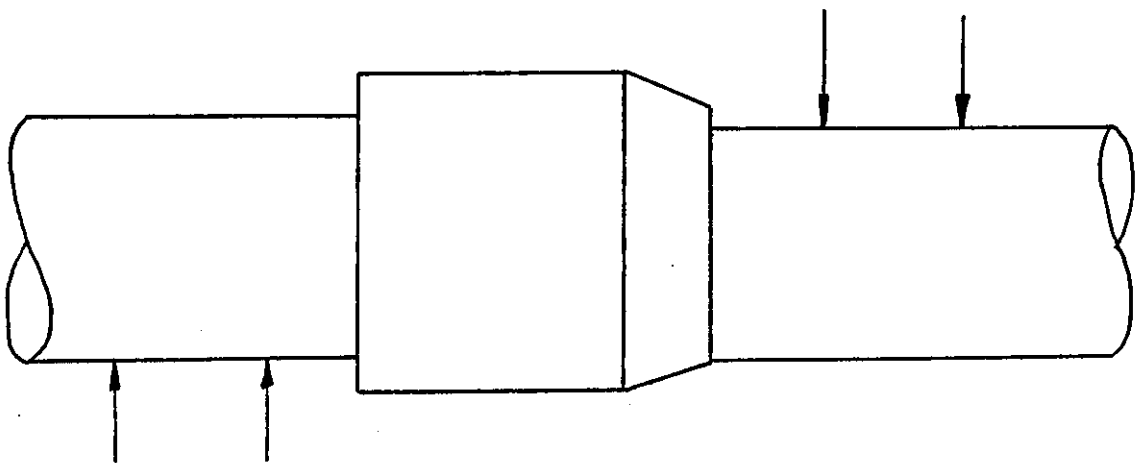
Jointing Force for Prototype Joint



1. SPIGOTS ALIGNED.



2.  $3^{\circ}$  DEFLECTION.



3. MAXIMUM SHEAR.

Figure 5.7

Pressure Test Conditions

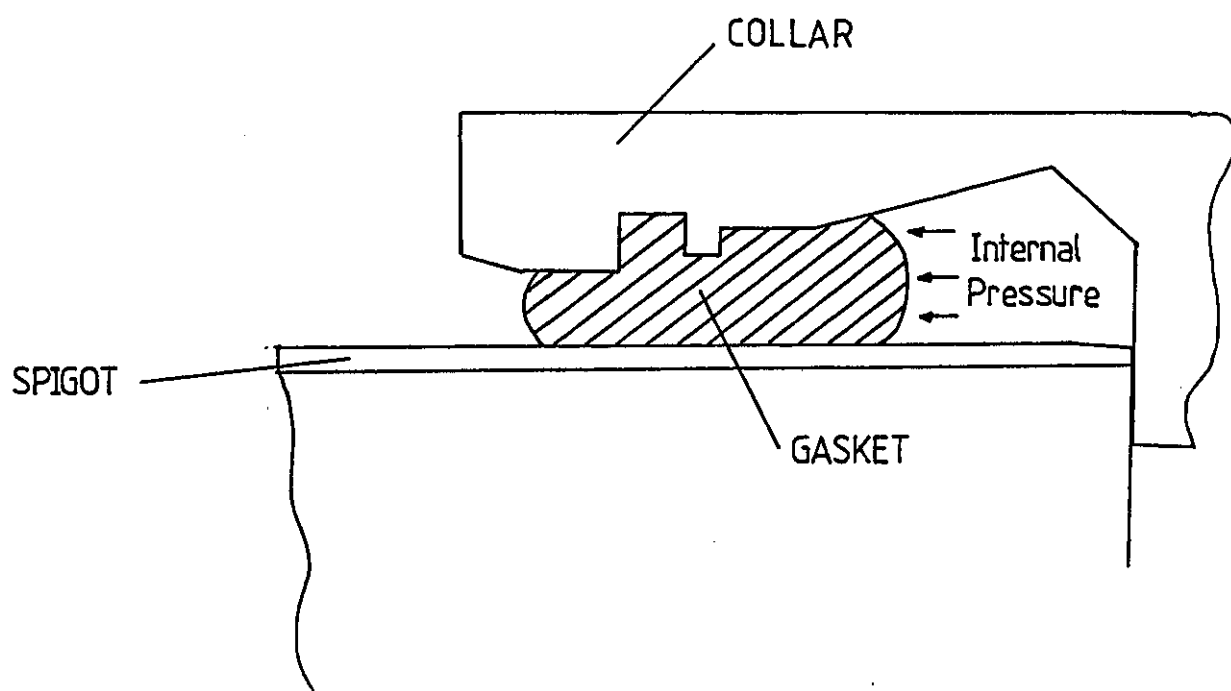


Figure 5.8

Gasket Position during Eccentric Pressure Tests

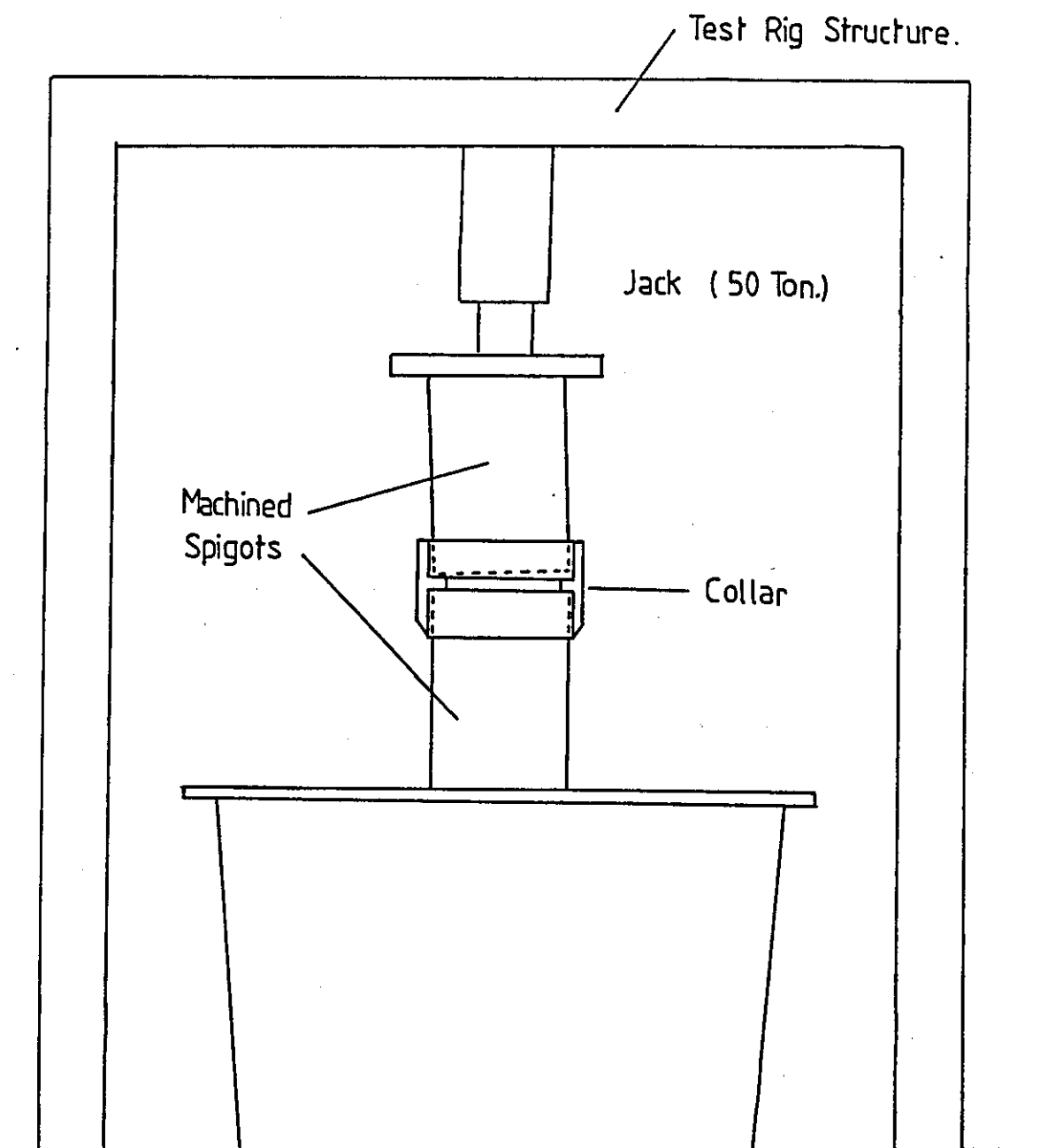
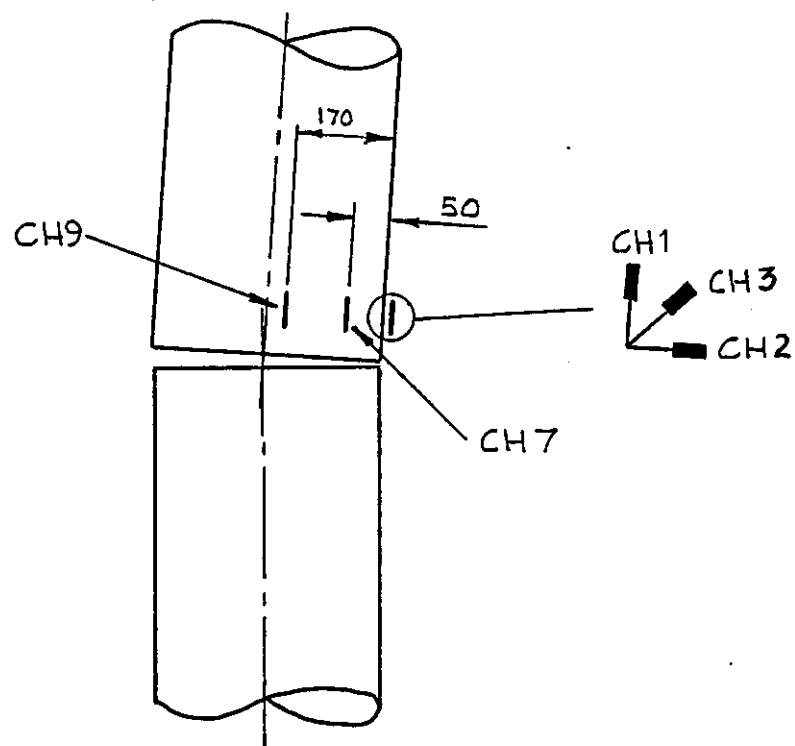


Figure 5.9

Arrangement during Model Joint Axial Load Tests



- CH1 - Longitudinal Strain at Contact Point
- CH2 - Hoop Strain at Contact Point
- CH3 - 45 Degree Strain at Contact Point
- CH7 - Longitudinal Strain 50mm from Contact Point
- CH9 - Longitudinal Strain 170 mm from contact Point

Figure 5.10      Positions of Strain Gauges during Model Axial Load Tests

# TEST AL1 : Strain versus Load

LOADING TO 22.3 kN

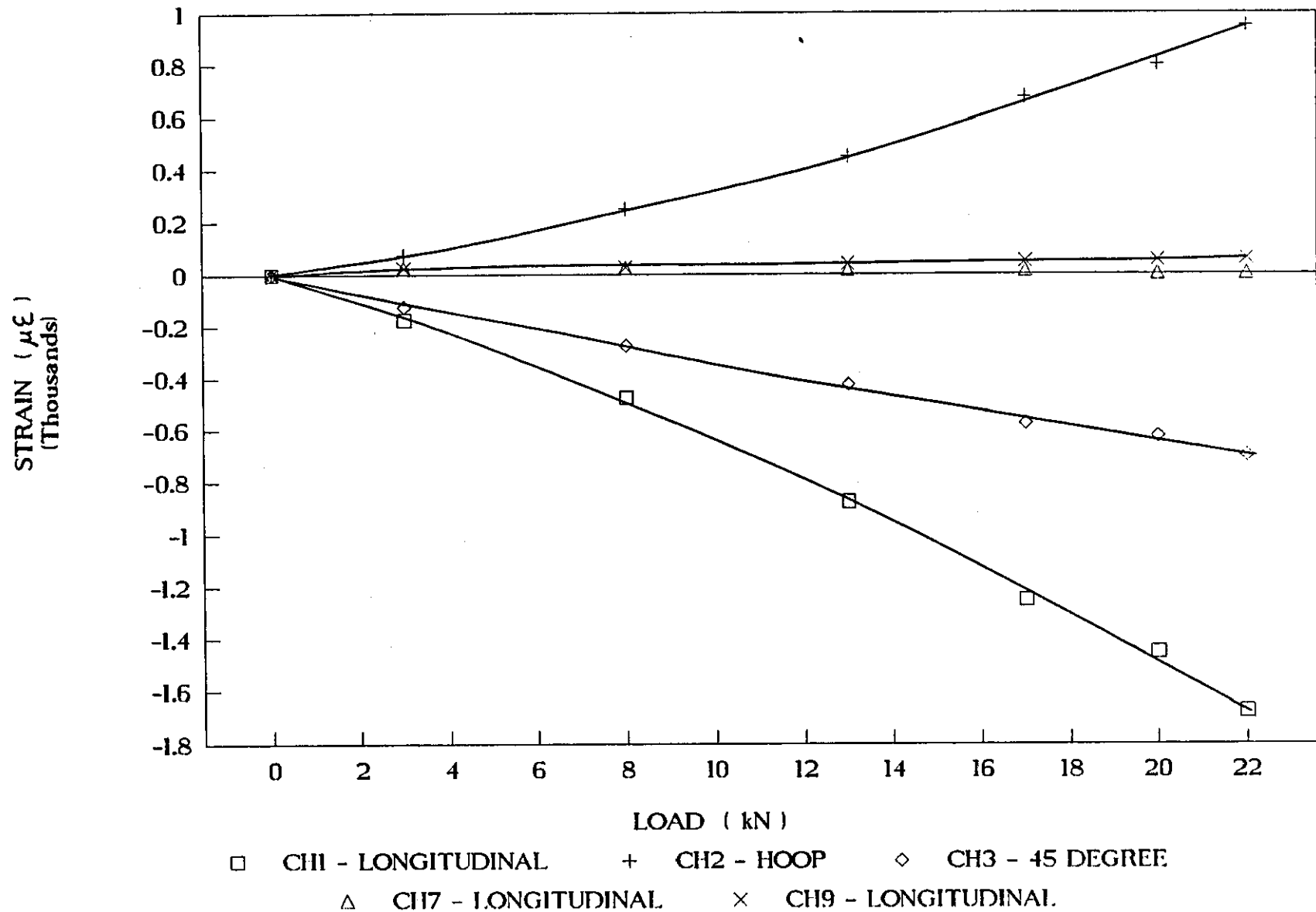


Figure 5.11

Strain Gauge Readings during Axial Load Test 1 (AL1)

# TEST AL3 : Strain versus Load

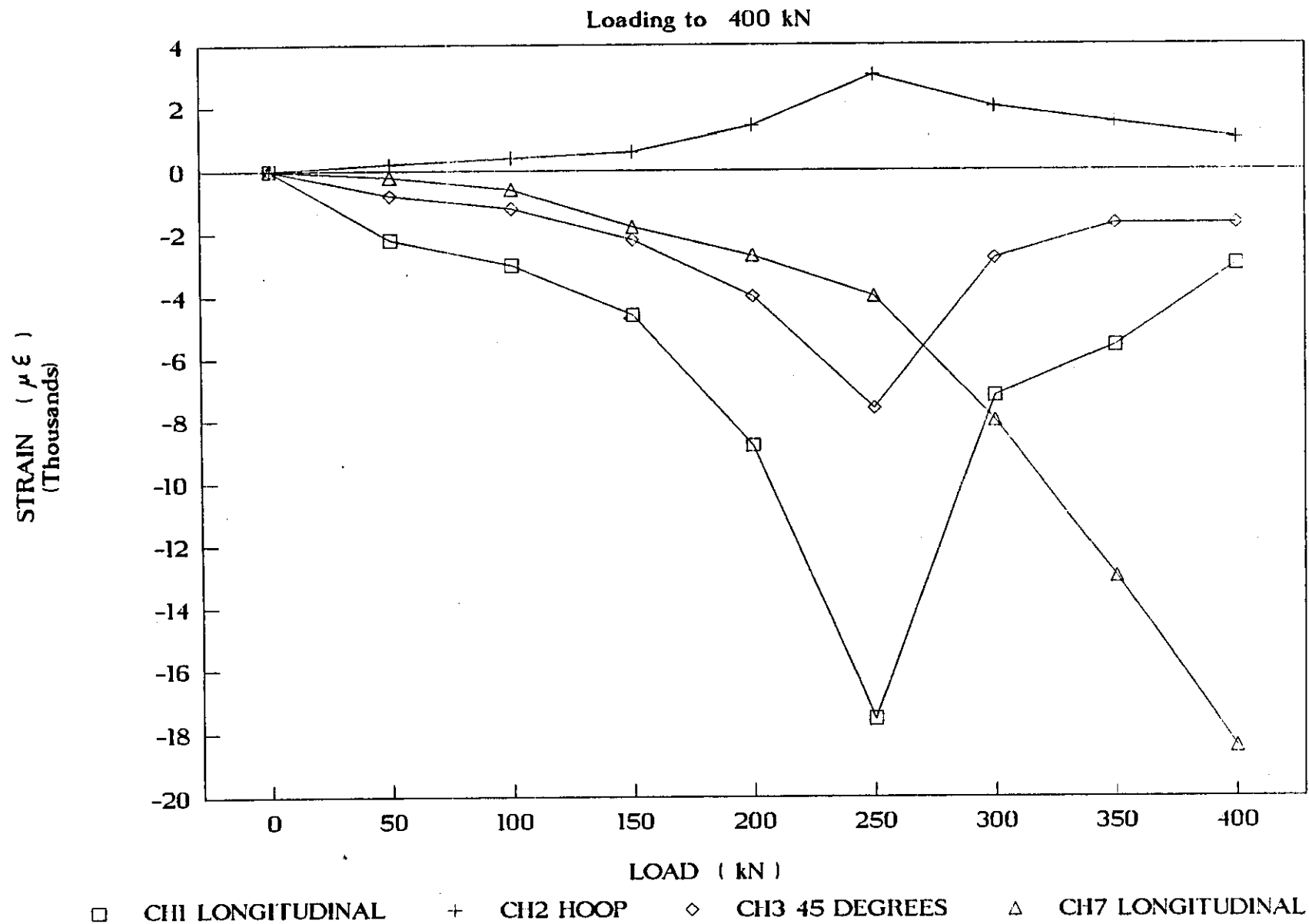


Figure 5.12

Strain Gauge Readings during Axial Load Test 3 (AL3)

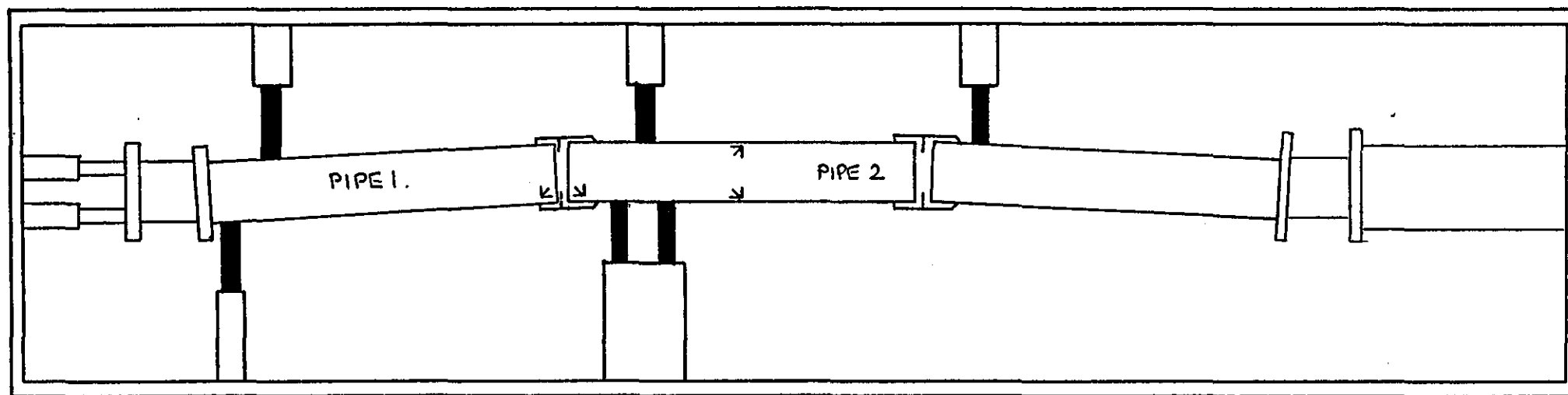


Figure 5.13

Layout of Trench for Full Length Axial Load Tests

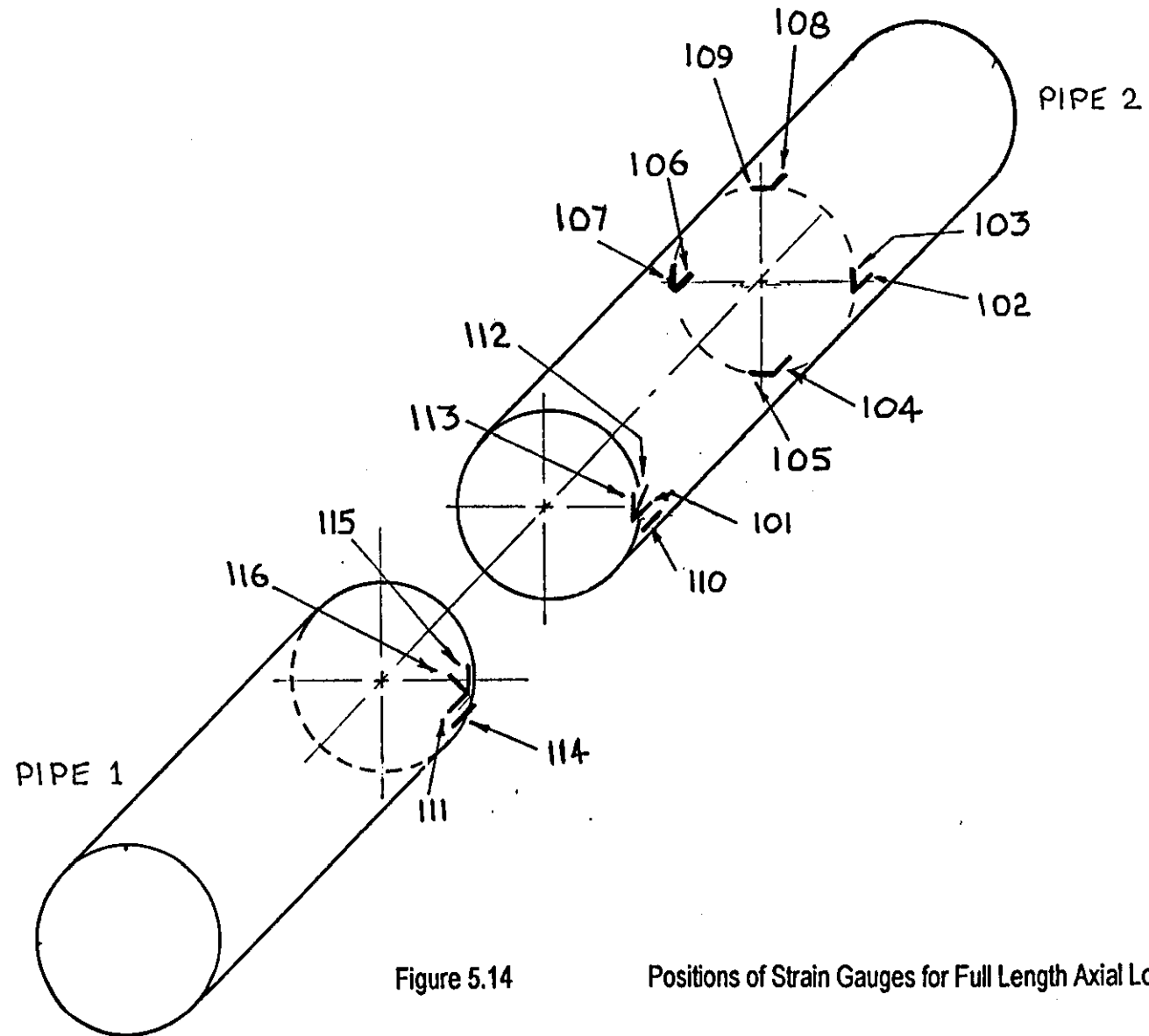


Figure 5.14

Positions of Strain Gauges for Full Length Axial Load Tests



# TEST FAL1 : Strain versus Load

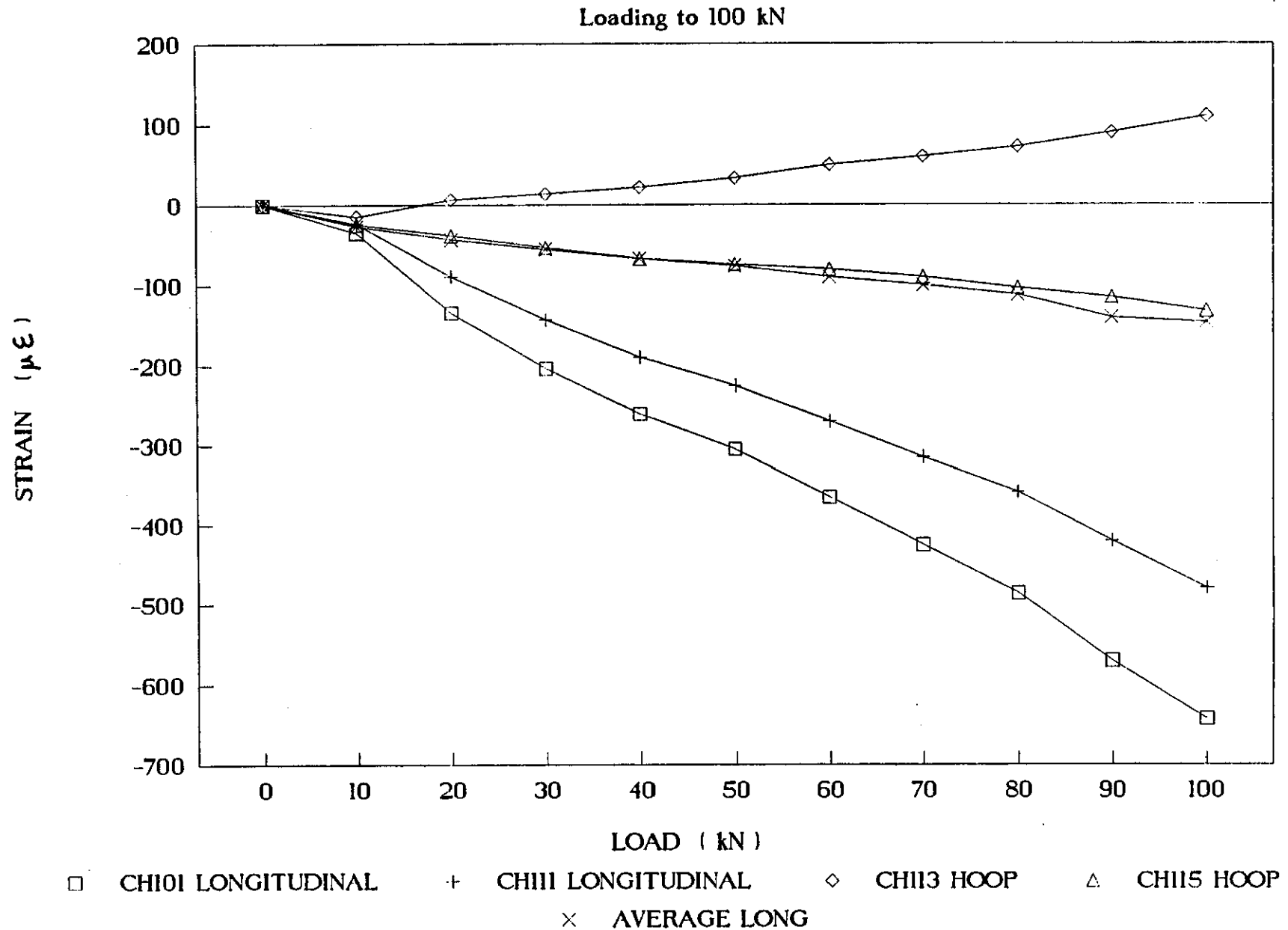


Figure 5.16

Strain Gauge Readings during Full Length Axial Load Test (FAL1)

# TEST FAL2: Strain versus Load

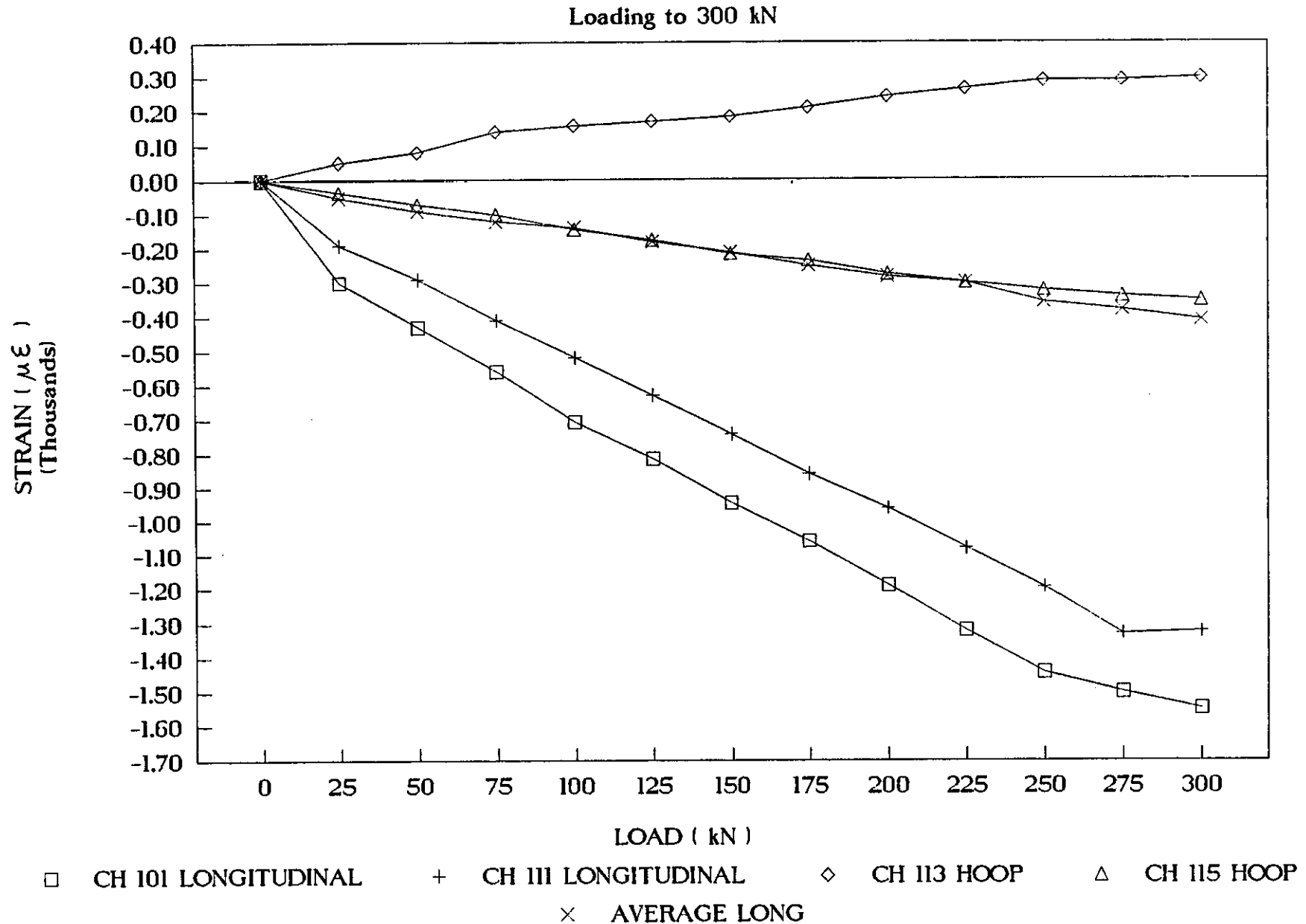


Figure 5.17

Strain Gauge Readings during Full Length Axial Load Test (FAL2)

# TEST FAL3: Strain versus Load

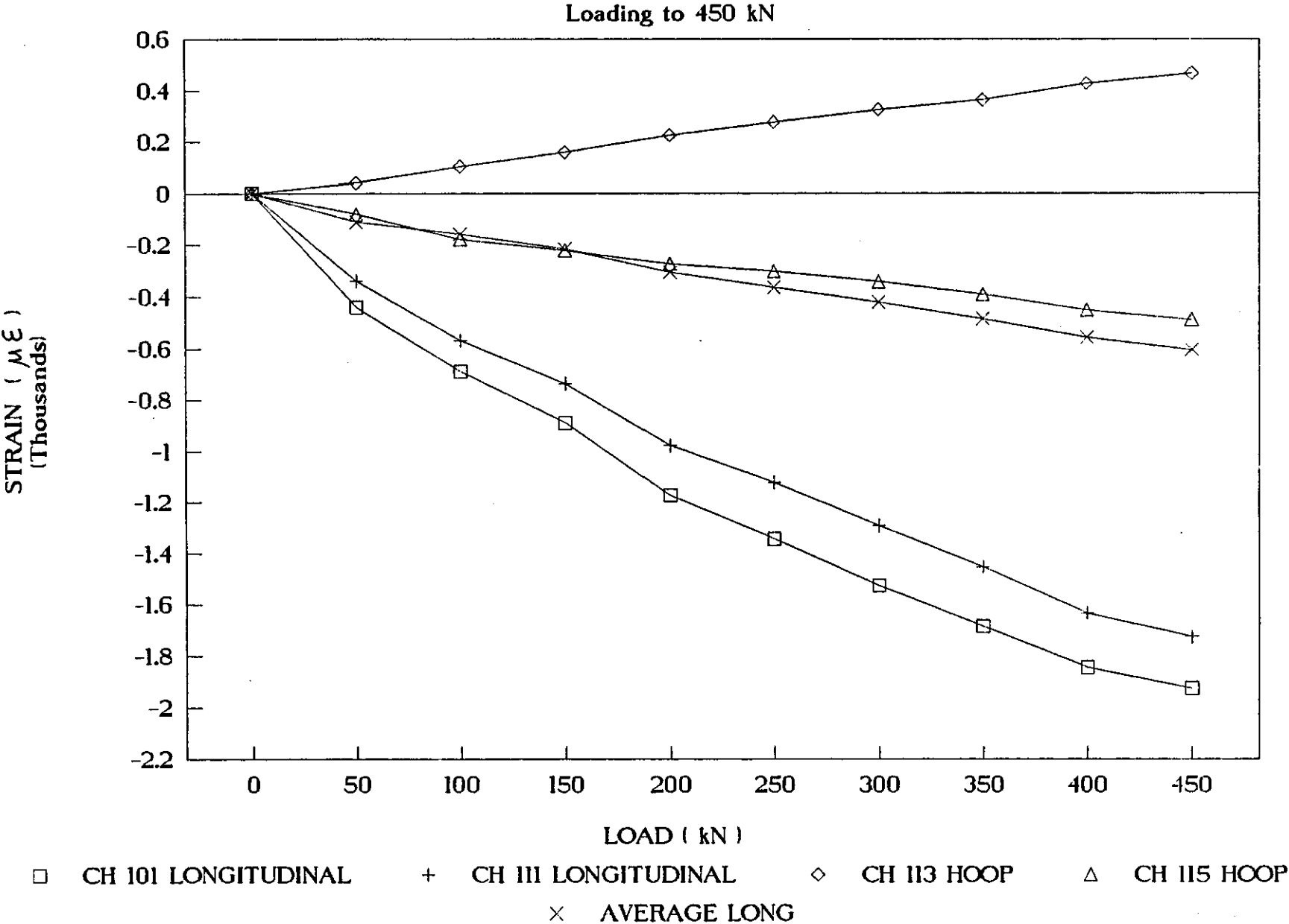


Figure 5.18

Strain Gauge Readings during Full Length Axial Load Test (FAL3)

# TEST FAL6 : Strain versus Load

Loading to 600 kN

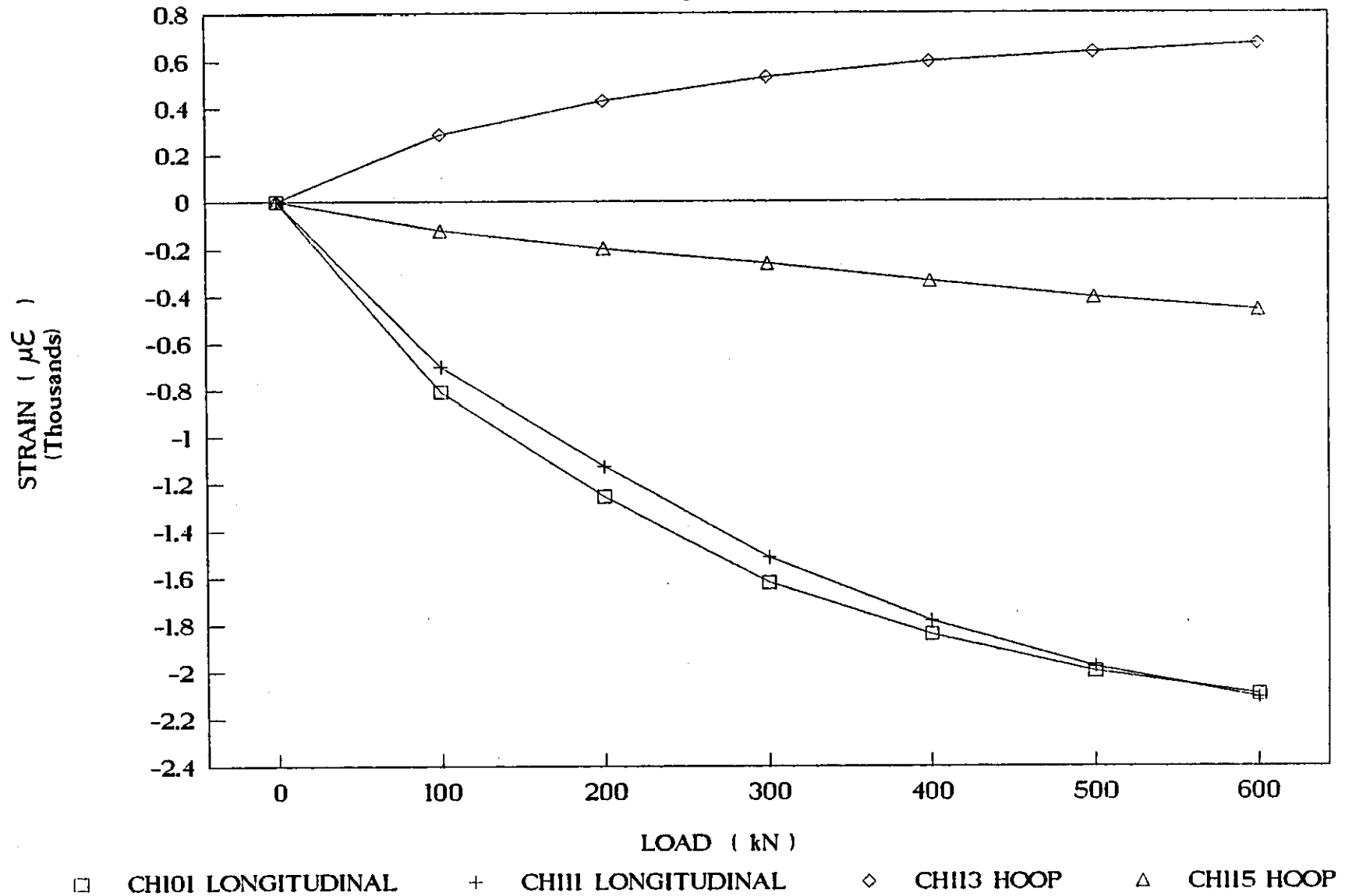
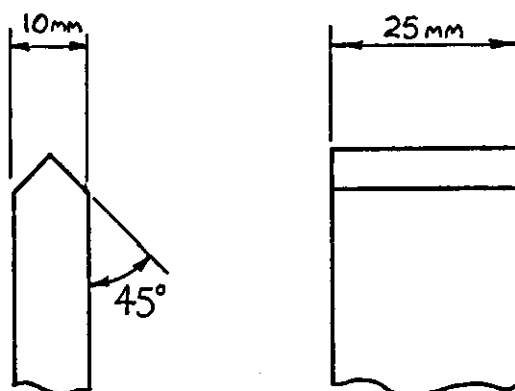
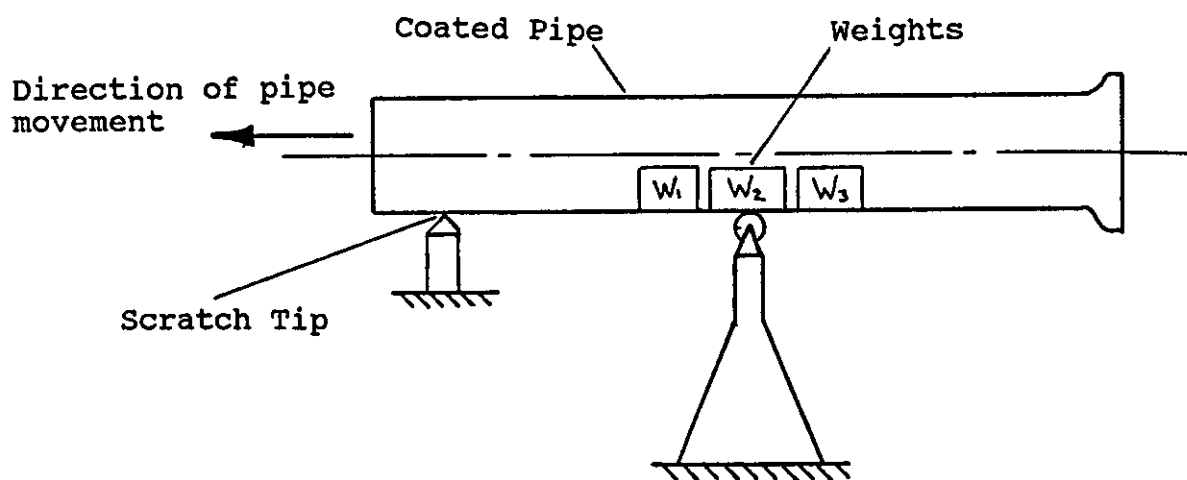


Figure 5.19

Strain Gauge Readings during Full Length Axial Load Test (FAL 6)



Details of Scratch Tip

Figure 5.20

Details of Coating Scratch Test

# POLYURETHANE COATING

## SCRATCH TEST

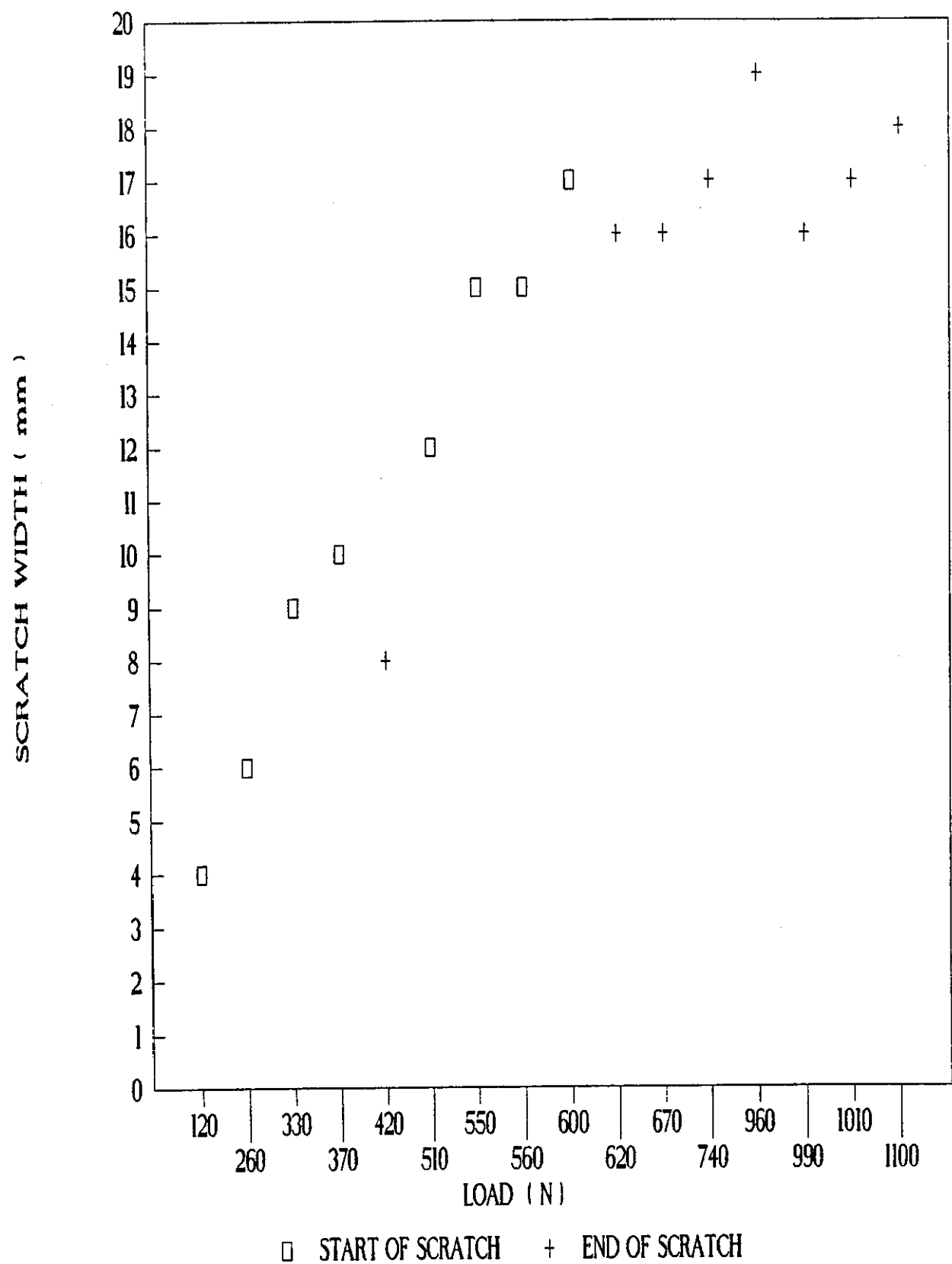


Figure 5.21

Polyurethane Coating Scratch Test Results

# POLYESTER RESIN COATING

## SCRATCH TEST

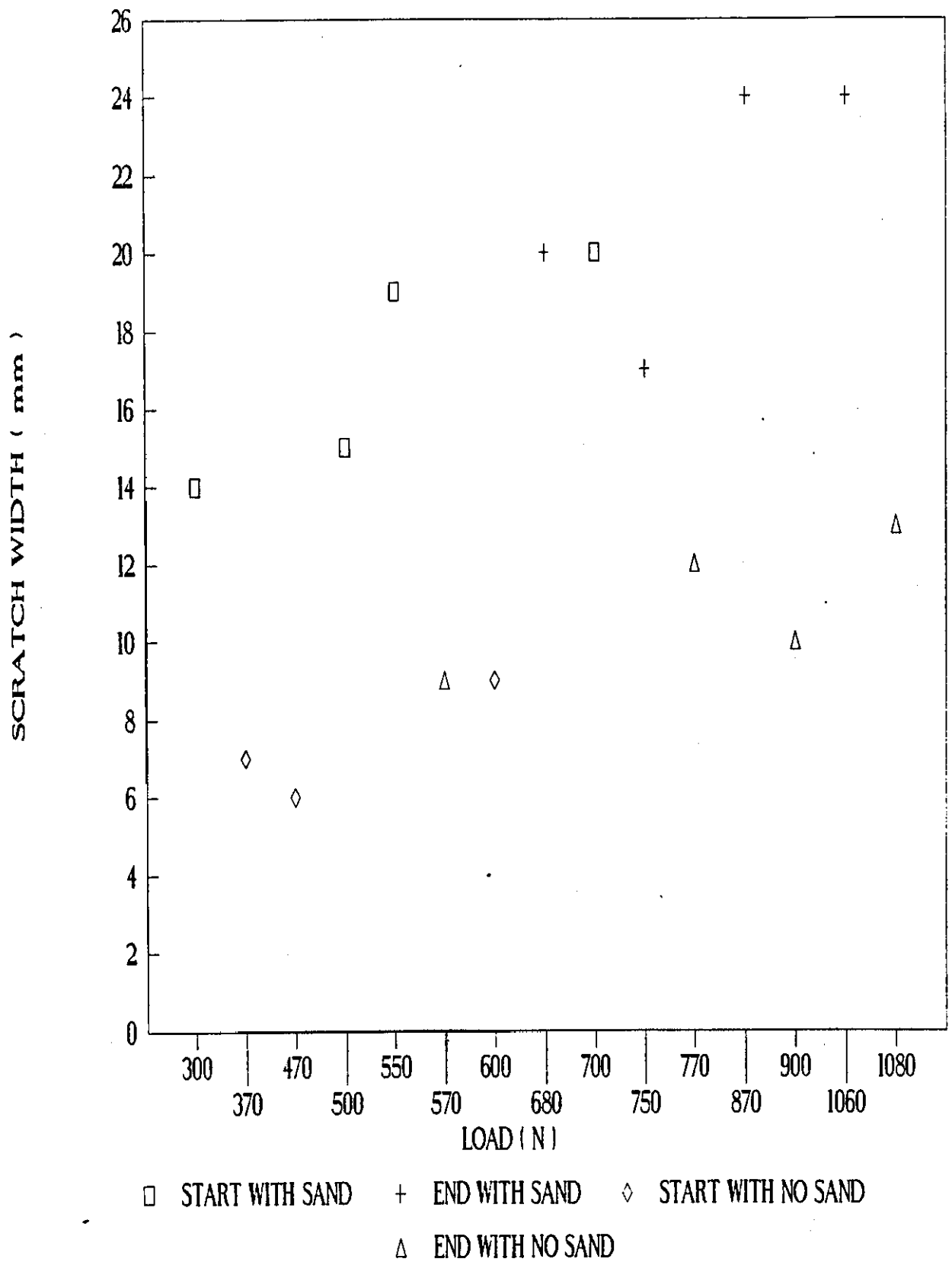


Figure 5.22

Polyester Resin Coating Scratch Test Results

# POLYESTER RESIN AND 1 LAYER OF FIBRE

## SCRATCH TEST

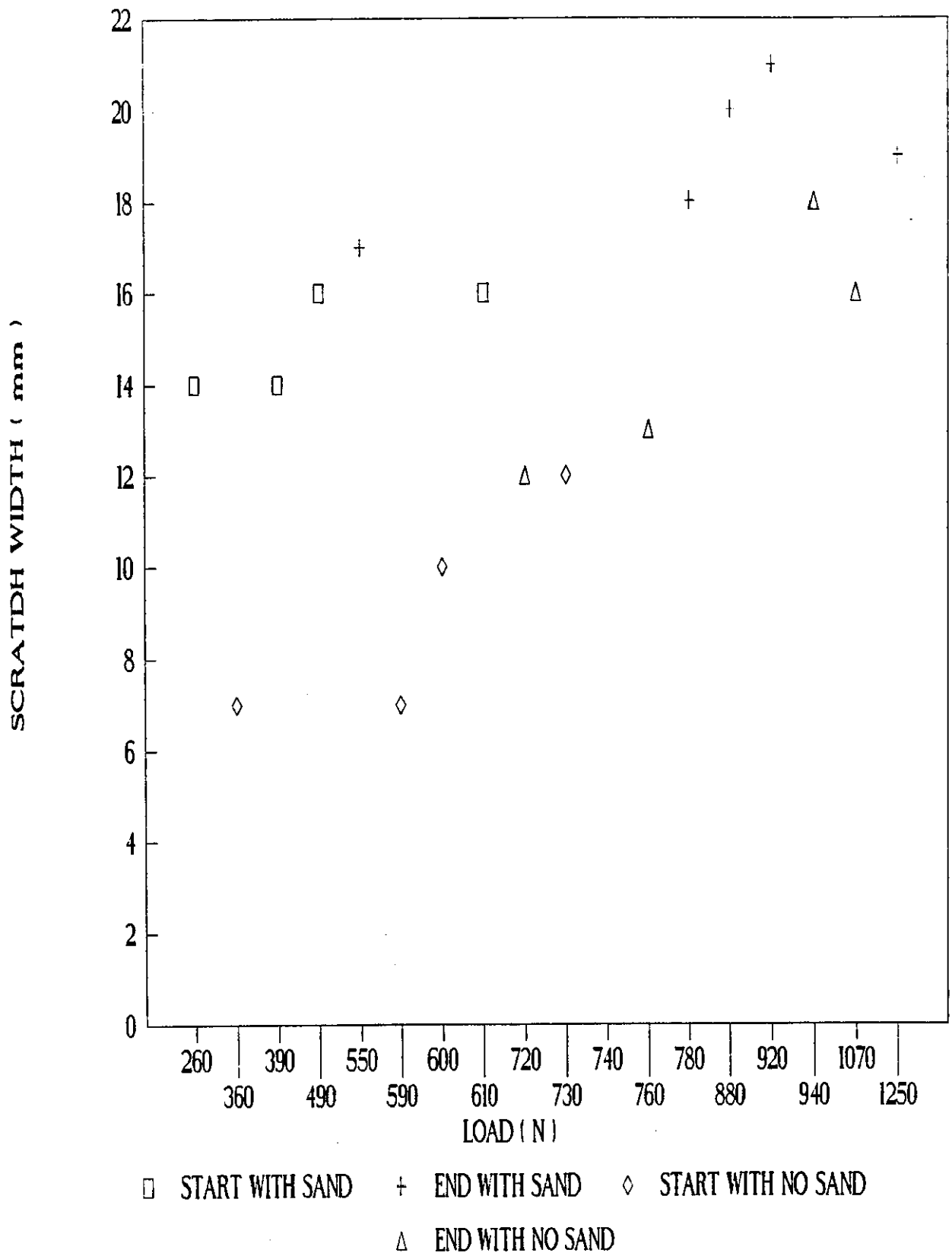


Figure 5.23

Single Layer Glass Reinforced Polyester Resin Coating Scratch Test Results

# POLYESTER RESIN AND 4 LAYERS OF FIBRE

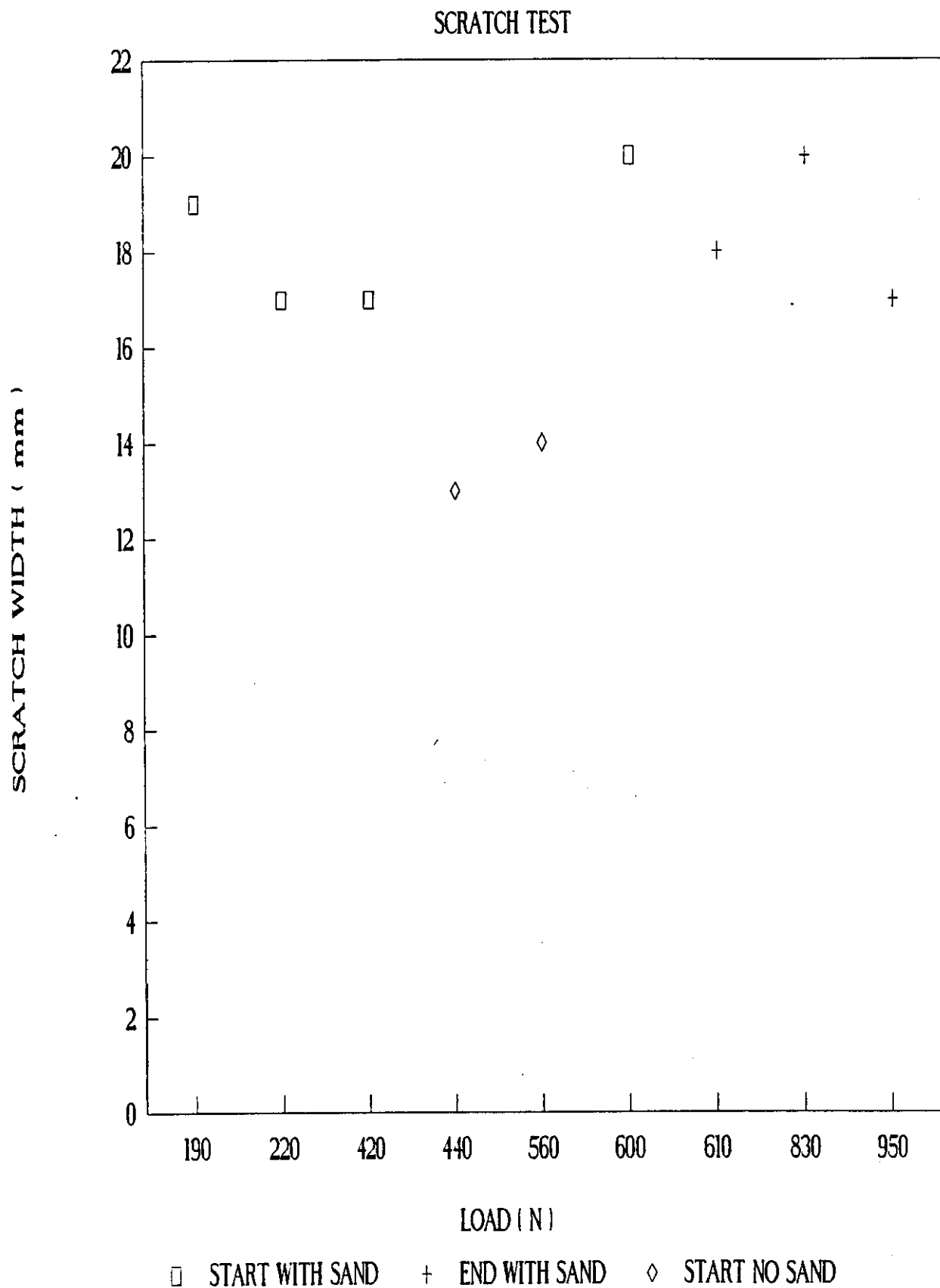


Figure 5.24

Multi Layer Glass Reinforced Polyester Resin Coating Scratch Test Results

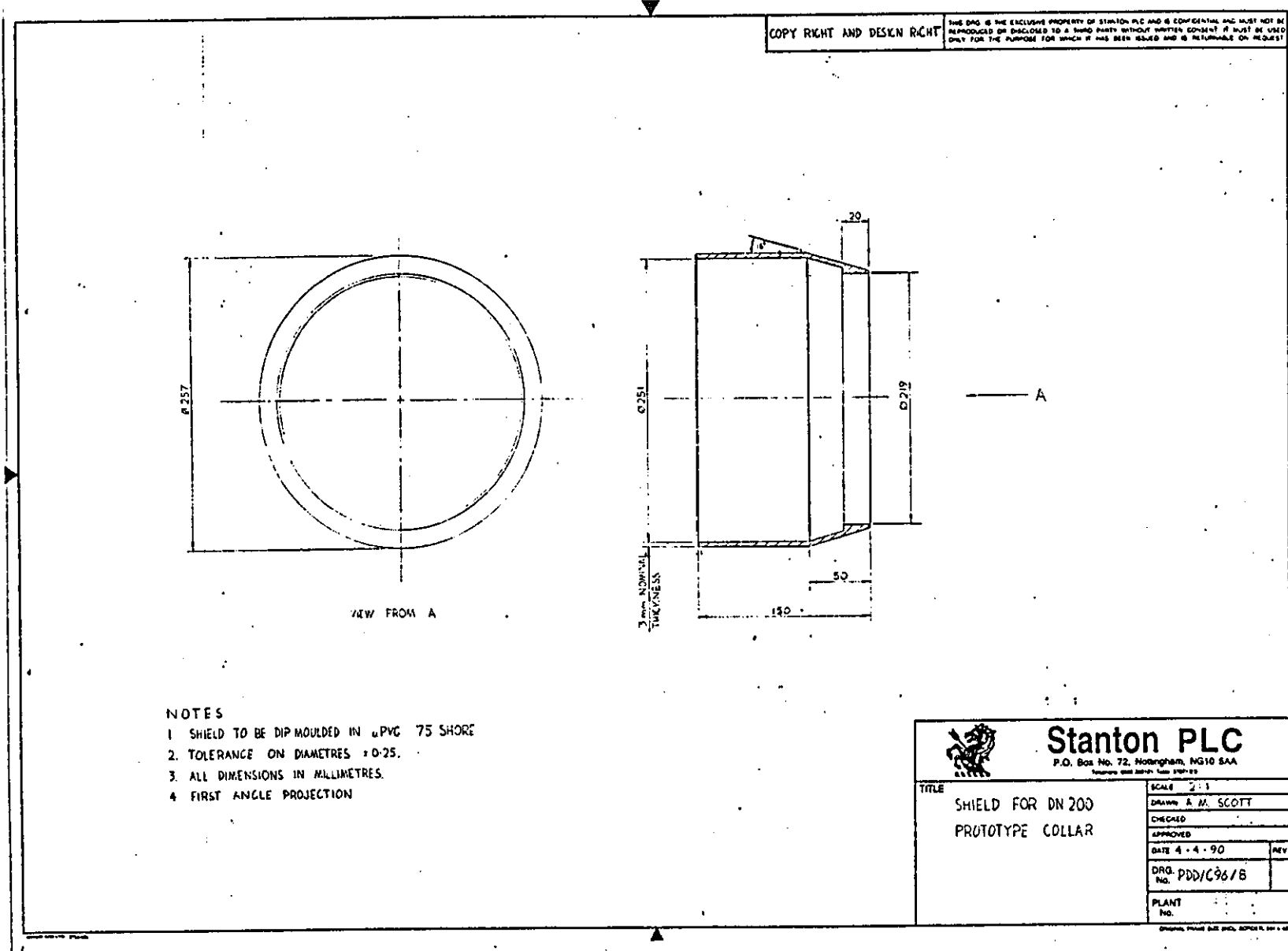


Figure 5.25

Shield for Prototype Collar

**CHAPTER SIX**  
**FIELD PERFORMANCE**

## 6.1 INTRODUCTION

Having developed the new collar joint and assessed the jointing capability, pressure integrity and load carrying capability in the laboratory, the next stage in the research was to carry out a field trial. Unlike the polyethylene pipe normally used with the pipebursting process, the new joint will not sustain tensile load. Consequently, the field trial was primarily aimed at developing an efficient system of site working practice that ensured continuous joint compression throughout the installation. The opportunity was also taken to monitor jacking loads and to assess the proposed methods of corrosion protection.

## 6.2 DESCRIPTION OF THE TRIAL

### 6.2.1 Site description and location

The site chosen was located adjacent to the A43 near Kettering, England and a plan of the site is given in Figure 6.1. The existing main consisted of a 200 mm nominal diameter grey iron pipe which although in reasonably good condition, had been abandoned due to re-routing of water services in the area. The length of main replaced was 40m in a single straight run, the direction of operations being up a slight incline from Point A to Point B. The depth to the pipe axis was approximately 1.2m. The soil conditions throughout the site consisted of firm to stiff clay with relatively high 'cohesive' properties, natural soil having been used to backfill the trench during original pipe installation. Soil movements were monitored half way along the replaced main at Point C.

There were a number of other services in the area, as detailed in Figure 6.1. Information regarding the locations of these services was gained from the appropriate authorities at the planning stage. The plans received only gave an approximate location of these services and in order to determine the exact locations, a number of inspection holes were dug prior to the trial. Most notable of the existing services were two British Telecom ducts running parallel to and either side of the main. At the closest point the roadside duct was 1.3m from the main.

The replacement pipes were DN 200 with a nominal thickness of 8.4mm (K12), and the new collars having an outside diameter of 250mm. The pipe bursting mole expanded the 200mm grey iron main to a diameter of 290mm leaving an annulus of 20mm around the collar joints. The most effective pipe coatings as determined in Chapter 5, were tested, these being polyurethane, heat shrink tape wrap and glass reinforced plastic. However, the glass

reinforced plastic coating was changed such that the fibre reinforcement was placed close to the pipe surface in order to avoid the 'pick-up' experienced during the scratch test in Chapter 5. The uPVC shield was used for protection of the collars.

#### 6.2.2 On site procedure

Jacking systems are not normally used with pipe bursting machines and so a new method of attaching the lead pipe to the moling machine was devised, as shown in Figure 6.2. The pipe bursting shield was extended by welding an extra section of tube onto the existing shield. The lead pipe was inserted into the extended shield and was attached to the moling machine using a chain. This section of chain was sufficiently long to allow the mole to advance independently of the pipes, but the chain ensured that the lead pipe was never able to exit from the shield completely. Since the joints will not transfer tensile loads, it was important to ensure that the pipes were continually under compressive loads. To this end, another length of chain linked the lead pipe to the last pipe in the train for every drive, and this kept the pipes and joints in compression. Machine and pipe advance were carefully monitored throughout the trial to ensure operations were fully synchronised. Since the jacking system (Plate 6.1) could not advance the pipe at the same rate as the machine advance, the two operations were carried out separately. The machine was advanced a set distance of 1.25m and then stopped while the pipes were jacked the same distance. The arrangement at the jacking station is shown in Figure 6.3 and Plate 6.1.

#### 6.3 JACKING LOADS

Jacking loads were monitored throughout the trial and the maximum load each day is given in Table 6.1. The corresponding contact stress, calculated by dividing the jacking load by the pipe surface area, is also given in the table. The contact stress values range between 8.2 kN/m<sup>2</sup> and 14.3 kN/m<sup>2</sup>, these values being in general agreement with Auld (1982) who quoted values albeit for pipe-jacking, through firm clay and wet clay of 5 - 20 and 10 - 15 kN/m<sup>2</sup>. These data are also comparable with the laboratory trial results over 8.7m which gave a maximum contract stress of 14.7 kN/m<sup>2</sup>.

The maximum load reached was 260 kN which is well below the 400 kN load carrying capability of the joint. Figure 6.4 shows the geometry of the pipes being jacked and this shows that the maximum angle of deflection, assuming no local compression of the tunnel wall, was 0.92 degrees. Although this is well below the maximum joint deflection of 3 degrees, the load

Table 6.1:            Maximum Daily Jacking Loads and Corresponding Contact Stresses

DAY	1	2	3	4
DISTANCE JACKED (m)	10	20	35	40
MAXIMUM LOAD (kN)	100	170	200	260
CONTACT STRESS (kN/m <sup>2</sup> )	14.3	12.2	8.2	9.3

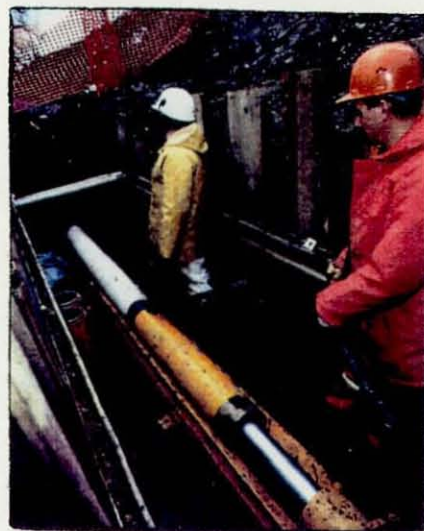


Plate 6.1

Field Trial Jacking System

is still transferred via a point contact. Under such conditions, the data presented in Chapter 5 predicts a maximum compressive strain of approximately  $1400 \mu\epsilon$  which is below the yield strain of the pipe material. Hence no yielding of the spigot end was expected at 260 kN jacking load and this was confirmed when inspecting the pipes retrieved from the trial, the pipes showing no sign of damage at the spigot end. It was considered that the cohesive nature of the natural soil and trench backfill was an obvious factor in keeping the jacking loads low on this site, since the clay would tend to be self-supporting due to negative porewater pressures. The contact stress at the start of the trial was relatively high being similar to the values experienced during the trial in compacted sand. The results in Table 6.1 are shown graphically in Figure 6.5 and this clearly illustrates the reduction in contact stress as the drive progressed. This could be caused by a softer or less dense pipe surround in the middle of the drive. The decreased level of contact stress, by comparison with the trial in compacted sand, could also be due to the reduced pipe profile.

#### 6.4 GROUND MOVEMENTS

Ground movements at the surface were measured at Point C halfway along the main, as shown in Figure 6.1. Surface measurements were taken using a standard land surveying levelling technique as the pipe bursting machine approached the measuring point, when the machine passed the cross section and three months after the trial. The results which were published by Rogers, Robins and Scott (1991) are reproduced in Table 6.2.

Initial movement at the surface occurred when the burster approached within 1.5m of the monitoring cross section, although significant movements only occurred once the burster was between 0.5 and 1m from this cross section. Hence, with the depth to pipe axis being 1.2m, significant movements in advance of the bursting machine only occurred at an angle of  $50^\circ$  and  $70^\circ$  to the horizontal. This is in general agreement with other published work. Tasker and Leach (1988) noted increases in strain in an adjacent pipe at angles of approximately  $45^\circ$  and Chapman (1992) noted similar results during laboratory trials as indicated in Figure 2.29. During the full scale laboratory trial reported in Chapter 4, surface movements were recorded after 3m of pipe bursting, which relates to an angle of  $42^\circ$  to the horizontal. It can be expected that the angle at which surface movements occur ahead of the pipe bursting machine is dependent upon conditions such as type and density of soil, degree of expansion of existing main and type of bursting machine employed. From the studies carried out in compacted sand and in stiff clay and from other published data, it is concluded that the angle may vary between approximately  $40^\circ$  and  $70^\circ$ .

Table 6.2:            Surface Ground Movements (mm) Observed During a Pipebursting Field Trial

DISTANCE FROM CENTRELINE	0m A	0.5m B	1.2m C	1.6m D
BURSTER 2.0m AWAY	0	0	0	0
BURSTER 1.5m AWAY	1	0	0	0
BURSTER 1.0m AWAY	2	1	0	0
BURSTER 0.5m AWAY	9	4	0	0
ABOVE BURSTER	19	12	1	0
AFTER 3 MONTHS	10	3	0	0

Table 6.2 also shows that the extent of lateral surface movement is limited to between 0.5m and 1.2m from the centreline of the main. Therefore, significant soil movements were experienced within a block of soil with boundaries extending from the main to the surface at an angle of between 45° and 70° to the horizontal. This is more clearly demonstrated by the sub-surface movements which were measured using buried extensometers. Chapman and Rogers (1991) compared the movements with those produced by a fluid flow model and the results are shown in Figure 6.6. The measured ground movements obtained from the trial correlate remarkably well with those predicted by the flow model. The maximum surface heave is virtually the same for both the measured and predicted cases. The flow model over-estimated the lateral extent of surface heave but this could be due to the confining effect of the original backfill trench.

The maximum ground movements occurred as the pipe bursting machine passed below the monitoring point. The maximum surface heave was 19mm, which equates to 42% of the radial expansion of the main. Furthermore, sub-surface movements immediately above the main were measured at 33mm, which equates to 73% of the radial expansion. These results reveal a number of interesting points. As reported in the laboratory trial, the soil was partially compressed and moves upwards as a single mass. Leach and Reed (1989) presented likely patterns of ground movements during pipe bursting, as shown in Figure 6.7. This work indicated that in trench conditions the pattern of movement is expected to be generally upwards. However, comparison of the ground movements immediately above the main (Point D, Figure 6.6) with the radial expansion shows that the pattern of ground expansion was similar to that expected in homogeneous ground. This differs with the results of the laboratory trial where the pattern of movement was generally upward. During the laboratory trial the combined effect of the compacted sand and trench floor were clearly influencing the direction of soil movements. During the field trial, the natural soil allowed a more even distribution of expansion around the main.

The transient nature of the ground movements is shown in Table 6.2, with a large proportion of the surface heave settling after 3 months. This is consistent with the predicted behaviour of time dependent collapse of the expanded clay soil and is partially due to the expanded diameter of the main being larger than the diameter of the replacement pipes.

## 6.5 CORROSION PROTECTION

The first four pipes were provided with different corrosion protection coatings, these being glass fibre (RPM), heat shrink tape wrap, a hard polyurethane and a softer polyurethane. In addition, the collars were protected with the uPVC sleeves described in Chapter 5. Each of these coatings was 2 - 3mm thick and was applied in addition to the normal active protection of metalised zinc.

The hard polyurethane and glass fibre coatings gave good protection to the pipe during installation and these coatings will be recommended for future work. There was some superficial damage to the hard polyurethane but this did not penetrate through to the zinc coating. The glass fibre coating suffered very little damage and there was no evidence of the glass fibre strands being ripped from the coating as was experienced during the scratch tests in Chapter 5. The softer polyurethane and tape wrap were severely damaged, the mechanism of damage being by a peeling action from an initially damaged area. The uPVC sheaths were found to provide satisfactory protection to the collar joints during installation, although there was some evidence of the sheaths lifting at the leading edge and allowing soil to infiltrate into the gasket area.

## 6.6 SITE PROCEDURE

The method of jacking short pipe lengths behind a pipe bursting mole was found to be reasonably successful. The importance of monitoring machine advance and continually comparing this with pipe-jacking advance cannot be stressed too greatly. Site personnel must be fully aware of the sequence of operations and must fully understand the importance of following the procedures.

The speed of advance was impeded by the jacking rate which was slow compared with the rate of advance of the pipebursting machine. In addition to the slow rate of advance of the hydraulic jack, the stroke of the piston was only 0.75m so in order to jack a full 2.5m length of pipe, four spacers were required. The time taken to repeatedly advance and retract the hydraulic jack greatly increased the time required for the jacking process. Consequently, the pipe train only advanced approximately 10m per day. Subsequent pressure testing of the newly replaced main was found to be successful up to 6 bar.

It was not possible to increase the pressure above 6 bar and further investigation showed that the first joint installed was leaking. It is likely that at some stage during the trial the pipe bursting machine progressed further than intended, thereby imposing tensile load on the pipe train and causing the joint to separate. Alternatively it is possible that the joint separated either whilst removing the specially coated pipes (since these were adjacent to the leaking joint) or during testing due to insufficient restraint at each end of the pipeline. Since it is possible that the separation occurred during installation of the pipes a review of the installation procedure is recommended.

## 6.7 FIELD TRIAL CONCLUSIONS

The proposed design of pipe joint and its installation procedure have been tested under site conditions. The procedure for installing this ductile iron pipe system behind a pipe bursting mole was found to be satisfactory except for the method of ensuring the pipes remain under compressive loads at all times. There is a possibility that during the trial the pipe bursting machine advanced too far forward, causing the first joint to separate. Hence improvements may be required in order to ensure the pipes are not subjected to tensile loads. This will require a review of the method of connecting the lead pipe to the pipe bursting machine, and a review of the method of monitoring machine and pipe advance. In addition, improvements are required in order to increase the speed of the installation process.

The load carrying capability of the pipes was found to be adequate, although jacking loads were potentially low due to the cohesive or self-supporting properties of the clay soil. Contact stress values were consistent with those predicted by Auld (1982) and also with the results obtained during the laboratory trial described in Chapter 4.

Measurement of ground movements showed that surface heave occurs at an angle of between  $40^\circ$  and  $70^\circ$  to the horizontal ahead of the pipe burster. In addition, soil movements can be expected to occur laterally and the limit of this lateral movement lies at an angle of between  $45^\circ$  and  $70^\circ$  to the horizontal. It was thought that the profile of ground movements would be affected by the fact that the original grey iron main would have been laid in a trench and that the existing trench boundaries would provide a plane of preferential movement. In this way the trench backfill would have risen *en bloc* as a plug. The results however indicate this was not the case.

Of the methods of pipe protection, the hard polyurethane and glass fibre coatings performed satisfactorily and these can be recommended for future work. Further studies are required concerning an industrial method of applying these coatings to the pipe.

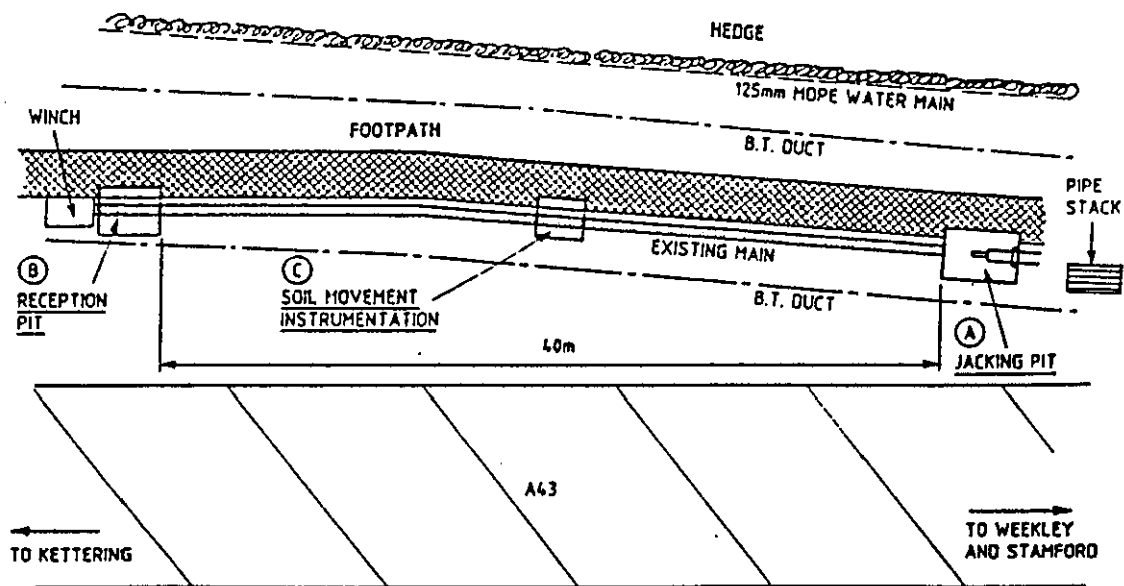


Figure 6.1

Site Layout for Field Trial

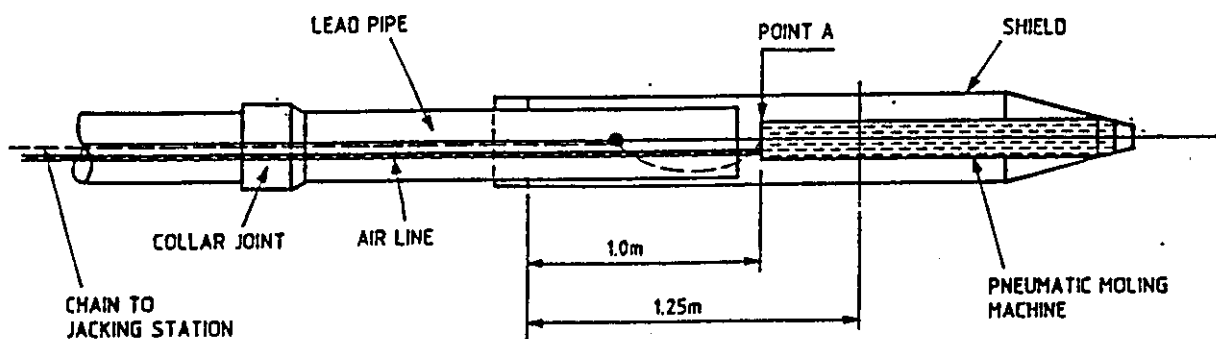


Figure 6.2

Method of Connecting Pipe to the Bursting Machine

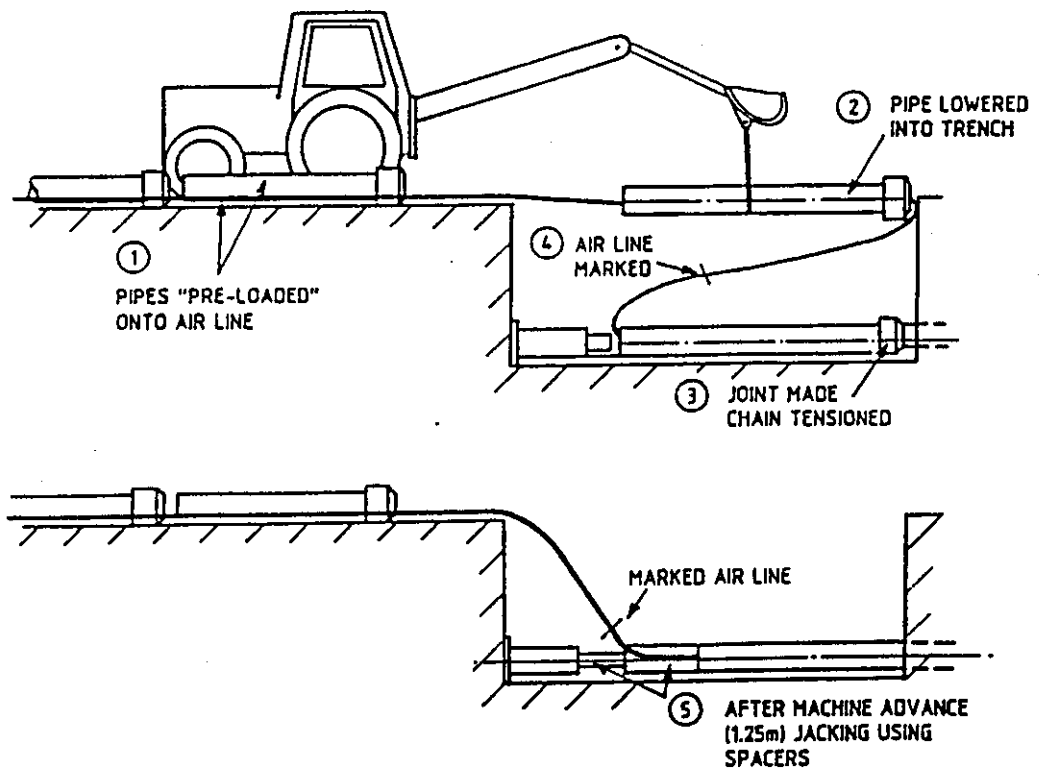


Figure 6.3

### Procedure for Pipe Installation

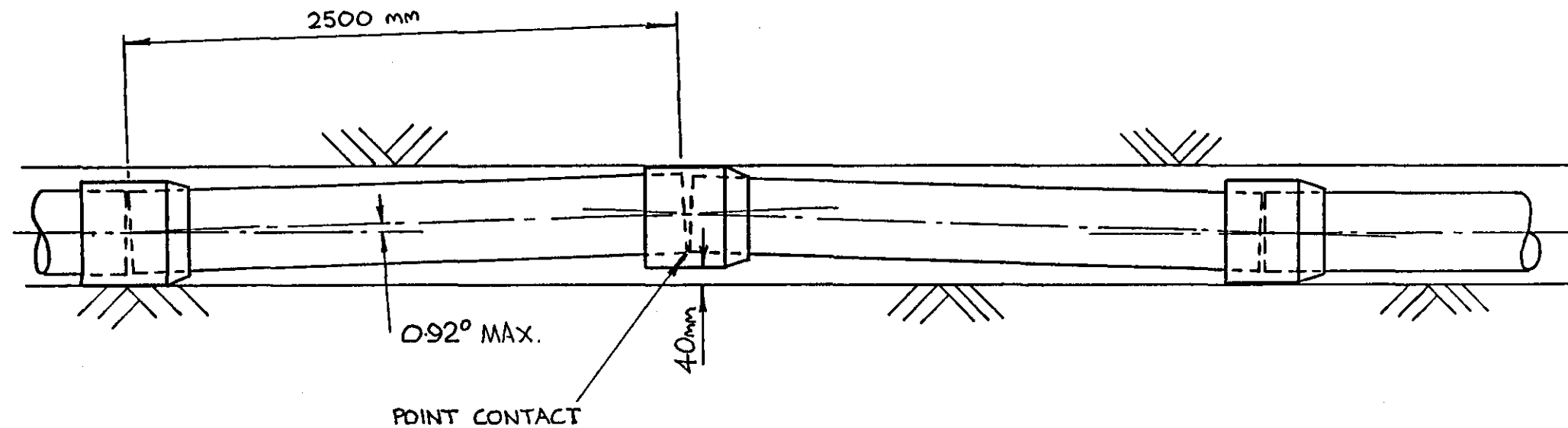


Figure 6.4

Geometry of Deflected Pipe during Field Trial

# FIELD TRIAL

## JACKING LOAD AND CONTACT STRESS

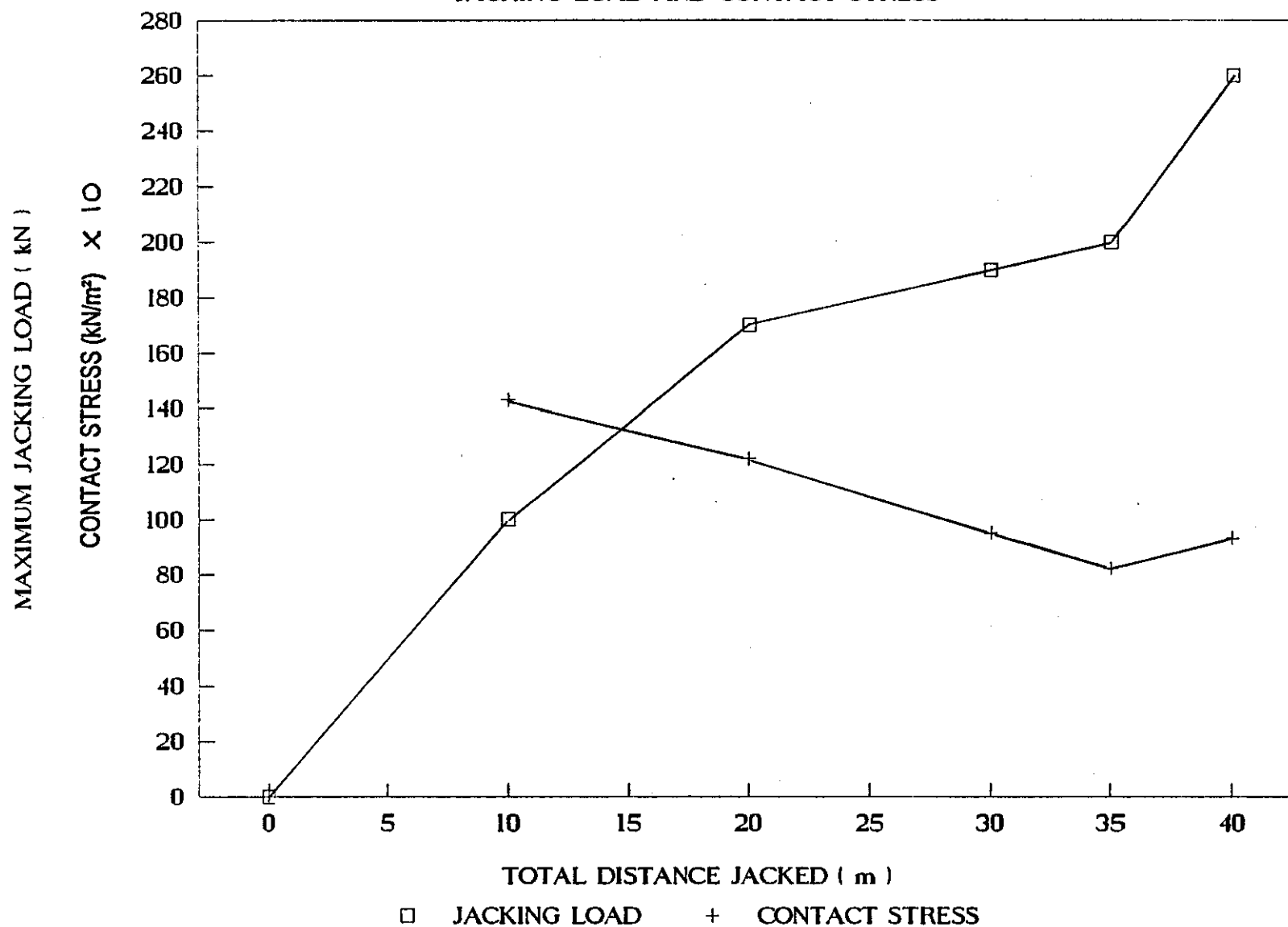


Figure 6.5

Graph showing Field Trial Jacking Loads and Corresponding Contact Stresses

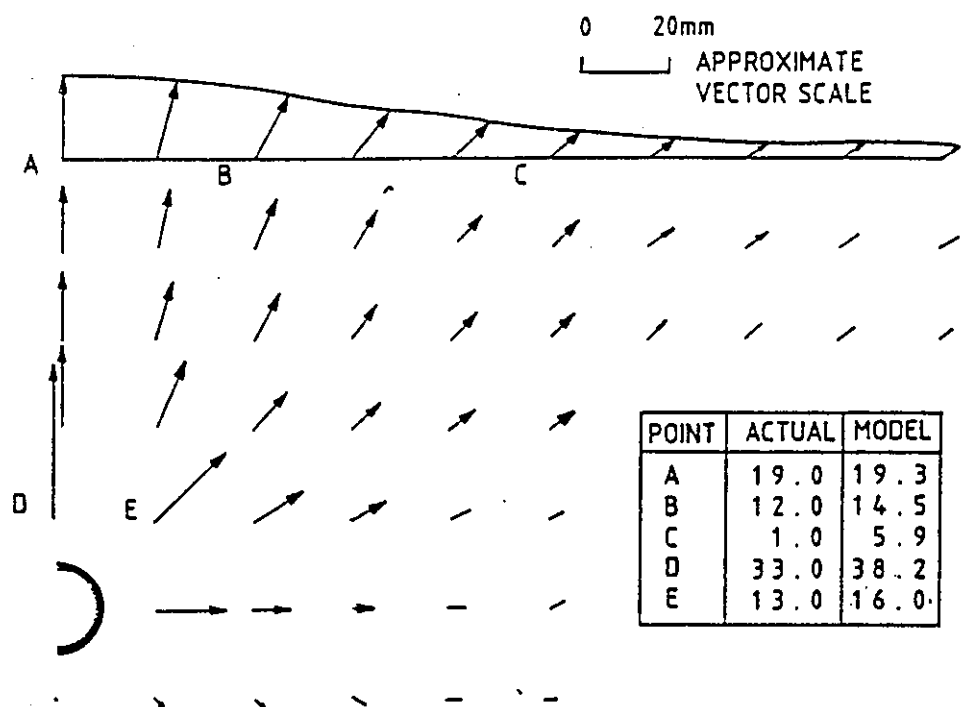
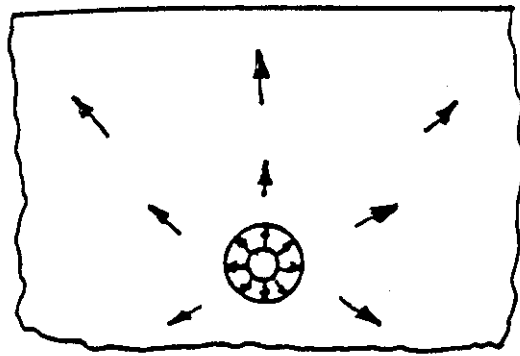
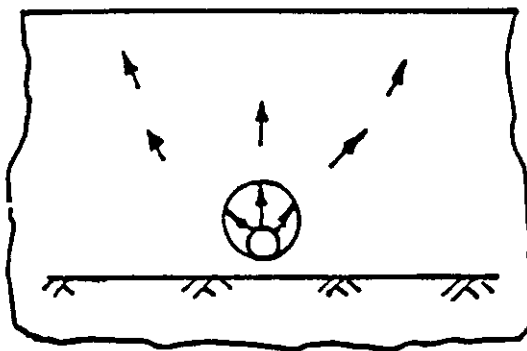


Figure 6.6

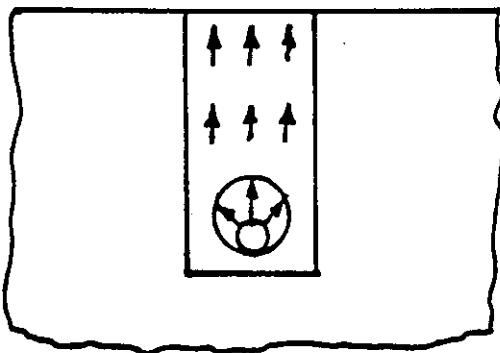
Sub-surface Movements Measured during Trial, Compared with Predicted Ground Movements (after Chapman, Rogers 1991)



(a) Homogeneous



(b) Hard Layer



(c) Trench Conditions

Figure 6.7

Likely ground movements caused by pipebursting operations (after Leach and Reed, 1989)

## **CHAPTER SEVEN**

### **CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK**

## 7.1 DUCTILE IRON JACKING PIPES

A review of literature and market survey concerning trenchless pipelaying techniques and pressure pipe design was conducted. This included a review of the condition of the existing water supply system and future water industry requirements. The factors affecting pipe installation were studied with particular emphasis placed on methods of predicting ground movements and jacking loads. It was established that a ductile iron pipe product suitable for installation by the pipebursting technique would satisfy water industry requirements for pipe renovation of existing water mains. The potential market for such a product was considered to be significant since there are an estimated 150 000 km of unlined grey iron water mains in service which were laid before 1940. The need to replace this ageing water system is already recognised by water companies since over 50% of annual water mains expenditure is allocated to rehabilitation of the water supply system.

Laboratory trials were conducted in order to assess the overall suitability of ductile iron for use with the pipebursting and to determine the parameters which would influence pipe joint design. Recorded jacking loads were found to be in general agreement with figures quoted by Auld (1982) and with figures calculated by analytical methods. This indicated that the laboratory trials gave satisfactory reproduction of typical site conditions. Socket profile joints caused average jacking loads to increase by 33% over those for smooth profile joints. However, maximum loads increased by only 17%. It was therefore established that the new pipe product could have some protrusion at the joint to house the sealing gasket, but that this protrusion should be kept to a minimum.

When installing socket-spigot profile pipes, ground movements were greater than those for smooth profile pipes. At a cover depth of 1m in compacted sand, compression of the sand mass was restricted to the immediate vicinity of the pipebursting mole and vertical displacement of the sand *en bloc* extended to the surface. The surface movements were found to be transient with initial displacement followed by settling of the surface. Initial and final surface movements were generally greater than those recorded in the field by other authors. This was considered to be due to the constraining effects of the test trench and the difficulty in reproducing site soil conditions.

The pipebursting operation had little influence on an adjacent pipe at a distance of 1m. The strains recorded were within acceptable limits and were consistent with those obtained by other authors (Dorling, 1984 and Leach and Reed, 1989).

A new joint and gasket system has been developed to satisfy the requirements of pipes used with the pipebursting process. Such requirements include high strength and pressure ratings, suitability for installation with minimum surface disruption and ability to withstand high axial jacking loads. The new joint is a collar connector into which the two pipe spigots are inserted. The protrusion of the collar above the pipe barrel was minimised in order to reduce potential jacking loads. This was achieved by using a small gasket which, on a DN 200 pipe, gave a 42% reduction in protrusion compared with a conventional socket-spigot arrangement. The collar has a tapered leading edge to aid radial displacement of the soil as the pipes advance. A plastic shield was designed to prevent soil infiltration at the joint and to reduce risk of corrosion. Pressure tests were conducted under adverse tolerance conditions with maximum design deflection and maximum spigot eccentricity. These tests showed that a minimum gasket compression of 7.6% was required to achieve a leaktight seal up to 65 bar.

When the pipes were axially loaded under deflected conditions, high strain levels were recorded at the spigot ends and there was little circumferential distribution of load. With minimum thickness K9 pipe the maximum allowable jacking load was 260 kN. At this load, the predicted maximum jacking length was 26m, which was perceived as too short for practical application. By refining the profile of the spigot end, the maximum load was increased to 400 kN giving a predicted jacking length of 40m. The use of K12 pipe gave a 500 kN maximum load and predicted jacking length of 50 - 55m, with the added advantage of avoiding damage to the pipe cement mortar lining. Further increases in allowable maximum load were predicted by modifying the spigot taper.

The new jointing system and installation methods were assessed by undertaking a field trial during which a satisfactory procedure for installing the pipes was developed. Further improvements are required in order to increase the speed of the installation process and this will make the system more marketable. The load carrying capability of the pipes was found to be adequate, although jacking loads were potentially low due to the cohesive properties of the clay soil. Analysis of ground movements confirmed there was a high degree of upward movement and this extended to the surface. Surface movements were noted to occur ahead of the bursting operation and the extent of lateral movement was not limited to the original trench wall.

At various stages of this research, methods of protecting the pipes against the potentially abrasive nature of soil and broken iron fragments have been studied. This has involved assessing a number of coating materials for scratch resistance. The studies were limited to

coating materials which were readily available either commercially or at the pipe production plant. Coatings which gave satisfactory performance were identified but further studies are required concerning both the performance of these coatings and methods of applying the coating to the pipe.

## 7.2 RECOMMENDATIONS FOR FUTURE WORK

The work described in this thesis provides a detailed investigation of laying ductile iron pipe by the pipebursting technique. The test data have been presented in a manner which established an adequate understanding of the technical parameters of the technique and this formed the basis for the pipe joint development work. Since there is a general lack of field data concerning trenchless techniques, the test data could be used in conjunction with those of other authors to form a database. Parameters such as ground movements and jacking loads are obvious candidates for inclusion in such a database.

Concerning the development work described in this thesis, there are a number of areas where further work is recommended. It may be possible to increase the load carrying capability of the joint by introducing a compliant material between the spigot end and the joint. This would distribute the jacking load and so reduce stress concentrations in the deflected joint. The increased jacking load would enable allowable jacking lengths to be increased thereby making the system more attractive to potential users.

Methods of corrosion protection have been studied but were limited to coating materials which were readily available. It is recommended that further work should be undertaken to develop a specialist corrosion protection system for this application. As part of this work, the level of performance required needs to be fully defined. The study should also include industrial methods of applying the coatings.

A site working procedure has been developed which can form the basis for future work. As the system is used in practice this will require further refinement in order to obtain the most efficient method of site working. With improvements in site procedure, it may be possible to increase the laying rate.

The method of connecting the lead pipe to the pipe bursting machine should be reviewed and a more accurate method of monitoring machine and pipe advance should be investigated to ensure the pipes are not subjected to tensile loads.

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APPENDIX A

GASKET COMPRESSION CALCULATION PROGRAM

```

*LIST
1  NODE 3
10  PRINT " TAB(10); "GASKET COMPRESSION CALCULATION PROGRAM"
20  INPUT " " ENTER MAX AND MIN SOCKET DIAMETER AT GASKET BULB ",cmax,cmin
30  INPUT " " ENTER MAX AND MIN SOCKET ENTRY DIAMETER ",emax,emin
40  INPUT " " INPUT THE NOMINAL DIAMETER OF PIPE ",dn
50  PROCdiameter
60  PROCannulus
70  PROCgasket
80  PROCmodbulb
90  PRINT " MINIMUM GASKET COMPRESSION ",a," %"
100 PRINT " MAXIMUM GASKET COMPRESSION ",b," %"
110 PRINT "Maximum Bulb Diameter Required = ";bmax
120 PRINT "Minimum Bulb Diameter Required = ";bmin
130 PRINT "Maximum Gasket Diameter = ";gmax
140 PRINT "Minimum Gasket Diameter = ";gmin
150 END
160 REM
170 DEFPROCdiameter
180 IF dn=200 THEN 170 ELSE 190
190 LET nmax =223.0
200 LET nmin =219.0
210 IF dn=250 THEN 200 ELSE 220
220 LET nmax=273.0
230 LET nmin=269.0
240 ENDPROC
250 REM
260 DEFPROCannulus
270 amax=((cmax-nmin)/2)+((emax-nmin)/2)
280 amin=(cmin-nmax)/2
290 ENDPROC
300 REM
310 DEFPROCgasket
320 gmax=cmin*1.038
330 gmin=cmax*1.023
340 ENDPROC
350 REM
360 DEFPROCmodbulb
370 LET a=6.0
380 LET b=49.8
390 bmin=amax/(1-(a/100))
400 bmax=amin/(1-(b/100))
410 bmax=bmax/((gmax/cmin)^0.5)
420 bmin=bmin/((gmin/cmax)^0.5)
430 ENDPROC
440 >RUN

```

# GASKET COMPRESSION CALCULATION PROGRAM

ENTER MAX AND MIN SOCKET DIAMETER AT GASKET BULB 7235.0,234.5

ENTER MAX AND MIN SOCKET ENTRY DIAMETER 7223.75,223.25

INPUT THE NOMINAL DIAMETER OF PIPE 7200.0

MINIMUM GASKET COMPRESSION 6 %

MAXIMUM GASKET COMPRESSION 49.8 %

Maximum Bulb Diameter Required = 11.2425661

Minimum Bulb Diameter Required = 10.9124542

Maximum Gasket Diameter = 243.411

Minimum Gasket Diameter = 240.405

