CHAPTER 1 INTRODUCTION AND BACKGROUND

1.1 PIPE JACKING

Pipe jacking is a trenchless technique for installing underground pipelines by jacking specially designed pipes through the ground from a thrust pit to a receiving pit. Pipes are advanced using hydraulic power packs located in the thrust pit as the ground in front of the pipeline is mined. Excavation can be carried out within a shield using either pneumatic tools or a tunnel boring machine, depending on ground condition and cost. The spoil is transported along the pipeline to the surface. Corrections to the alignment of the pipes are made using hydraulic rams in the shield in conjunction with frequent surveying to fixed reference points. The technique is illustrated in Figure 1.1.

Pipe jacking offers the benefits of non-disruptive construction coupled with a unique one-pass lining without need for temporary ground support or secondary lining. Pipe jacking is now available under most types of ground condition, from hard rock to highly unstable silts and sands with high water table (Pipe Jacking Association 1995a, Thomson 1993, Sharp and Turner 1989). It offers considerable advantages over the traditional open-cut methods (Read 1986, Kosowatz 1987, Thomson 1993). Besides minimising surface disruption to the public and the environment, the risk of settlement is far less and the inherent strength of monolithic linings almost eliminates the risk of damage to existing utilities from further adjacent excavations. Moreover, it offers excellent flow characteristics with a smooth internal finish and lends itself easily to mechanised excavation. A summary of the conditions in which pipe jacking is a competitive alternative to other forms of primary lining is given by Craig and Muir Wood (1978). The main disadvantage of this method is its limitation to access the jacking front, which may result in heavy cost if unexpected failure happens (Shullock 1982).

The pipe jacking technique is thought to have first been used in Roman times. The first records of pipe jacking jobs were for installations done in the USA between 1896 and 1900 for the Northern Pacific Railroad Company (Moss 1993, Thomson 1993). Gradually it became standard practice for a number of railroad companies to jack cast iron pipes under rail tracks. By the late 1920s, concrete pipes were being used. Pipe sizes ranged from 750mm to 2400mm in diameter. Records reveal that working methods were similar to a simple hand-mined jacking job of today. Excavation was mainly by shovel and pick, helped by pneumatic clay spades in tougher soil with air hammers to break out boulders (Thomson 1993).

Prior to World War II, the pipe jacking technique was used on an isolated basis in various countries, among them the UK, Germany and Japan (Moss 1993, Thomson 1993, Anon 1978). In the earlier stage, pipe jacking covered mainly the man-entry size and was used for short lengths of tunnel. It was usually used for sections of pipelines under embankment, roads and railways, where open cut methods would be particular uneconomic because of the need to keep traffic moving continuously. Likewise there had been many occasions when the method had been used for small sections of long tunnels that passed below buildings. With the improvements in techniques and the application of modern technology, for example, the introduction of mechanised excavation and computerised laser guidance, pipe jacking may be used for long distance driving and different pipe sizes from 100mm up to very much larger diameter as an alternative to open-cut methods either for direct economy or for less disturbances at the surface (Moss 1993). Pipe jacking now became one of the popular methods to install underground pipelines in many countries (Wang 1982, Anon 1987, Roisin 1989, Pau et al 1993, Liao and Cheng 1996). The basic principles of the technique have been presented in detail by the American Concrete Pipe Association (1960), Richardson (1970), Hough (1974) and Thomson (1993). To improve the quality of pipe jacking, the British Pipe Jacking Association published notes giving guidance on design and practice (Pipe Jacking Association 1981, 1986, 1995a and 1995b).

The renewed interest in pipe jacking is a natural consequence of the needs of the market. Throughout the Western World a massive replacement programme was put underway to make good war-damaged networks and to meet the demands of rising living standards. Many of the pipelines had to be installed under busy roads and railways and at depths where open-cut was impracticable. Pipe jacking offered a solution which allowed short crossings to be made in a way that is safe, as well as economical. To cope with different ground conditions and to meet the varying needs of authorities in different countries, new methods were devised and equipment improved.

Recently, the pipe jacking technique developed very rapidly. Advances in technology have been particularly significant in Japan and Germany. The computerised laser guidance was widely used to reduce the pipeline misalignment (Lock 1988). Pipe jacking techniques can now be used at great depths below ground and in unstable ground conditions (Sharp and Turner 1989, Pipe Jacking Association 1995a) and pipes can be pushed along a curved line if needed (Wallis 1984, Nomura al et 1985). Pipe jacking over a very long distance was reported in Germany (Remmer 1995, Lauritzsen et al 1996). To deal with the larger diameter pipes and long lengths of thrust, lubrication along the pipeline and intermediate jacking stations were introduced to reduce the jacking forces (Jones 1990, Washbourne 1986, Durden 1982, Pipe Jacking Association 1995a and 1995b).

1.2 CONCRETE JACKING PIPES AND PIPE JOINTS

The most common pipe used in pipe jacking is concrete pipe, although different pipes have been used in practice, such as steel pipe, plastic pipe and clay pipe. The choice of the pipe can be influenced by diameter, length of drive and by ground condition (Pipe Jacking Association 1995a and 1995b). The concrete pipes used for pipe jacking were usually man-entry diameters from 900mm to 3000mm, in compliance with British Standard 5911: Part

120:1989. Pipes are generally available in lengths between 1.2 and 2.5 metres and are designed such that the jacking forces may be transmitted along the pipeline without damage to the joint. The fundamental requirements of the jacking pipes are as following (Thomson 1993):

- Resistance to internal and external corrosion
- Capacity to withstand static and dynamic loading
- Good flow characteristics
- Satisfactory whole-life costs

Precast concrete pipes are generally cast in manufacturers' works and transported to site by road. The pipes are manufactured by centrifugal spinning or vertical casting using concrete with a 28 day characteristic cube strength greater than 60MPa. Spirally wound reinforcement spot welded to longitudinal steel to form internal and external cages are often used to prevent damage during the temporary handling and installation stages.

The key part of a pipe is the pipe joint. The functional requirements of a joint on a jacked pipe are adapted from Clarke (1968):

(1) It should be designed to permit angular and axial movement large enough to tolerate maximum displacement likely to occur without damage or loss of watertightness.

(2) It should be designed to withstand the force applied during installation without detrimental damage.

(3) It should remain efficient throughout its working life.

(4) It should be simple to make and dismantle in the limited space of the jacking thrust pit.

Traditional jacked pipe joints in the United Kingdom have been an in wall spigot and socket as shown in Figure 1.2(a). Disquiet in the industry about the performance of the in wall joint and its ability to transmit longitudinal loads has led to the introduction of new joint details. The most often used of the new joints is shown in Figure 1.2(b), the steel collar joint.

The main reason for its introduction is the belief that jacking load may be better transmitted through the centre of the jacked pipe wall rather than at its edges. An increase of available end area may improve the pipe's jacking load carrying capacity.

Limited research has been carried out to ascertain the performance of the pipe with the steel collar joint. Milligan and Ripley (1989), and Boot and Husein (1991) suggested the use of packing material with low Poisson's ratio and low stiffness to improve the stress distribution on the pipe / packing interface. To improve the current understanding and to improve the pipe design, some more research on this subject should be done. In general, it is believed that the performance of the concrete pipe is mainly affected by:

- Properties of the packing material
- Properties of the concrete pipe and the joint type
- Properties of the surrounding soil
- Interaction between the pipe and the surrounding soil
- Pipeline misalignment at the pipe joint

1.3 THE NEED FOR RESEARCH AND OXFORD RESEARCH

Although pipe jacking has widely been used in many countries, many factors affecting the performance of such tunnels are still not clear. To improve the current understanding, more research is still needed, for example the stress distribution patterns and stress localization within the pipe due to pipeline misalignment. The need for research in several areas was first reported by Kirkland (1982) when he stated the need for researchers to establish more scientific facts of the pipe jacking. The Construction Industries Research and Information Association (CIRIA) then undertook a review of pipe jacking and published a Technical Note 112 (Craig 1983), which recommended that the following areas required research:

- Friction loads in different ground conditions

- Characteristics of joints and joint packing materials
- Effects of cyclic loading on pipes
- Effects of lubrications in reducing friction
- Development of a site investigation test to predict friction forces

As a result of increasing interest in the prediction and control of ground movements due to tunnelling, the measurement of ground movements and pressures has now been included within the list of objectives.

Following the recommendation and support of the Pipe Jacking Association (PJA) and the CIRIA, Oxford Research in pipe jacking began from 1986 and has now been in progress for over twelve years. Project one, from 1986 to 1989, involved laboratory testing of model concrete pipes and a number of common joint materials (Ripley 1989, Milligan and Ripley 1989). This research emphasised the need for suitable packing material at pipe joints along with careful control of pipeline alignment, and the superiority of steel banded butt joints to in wall spigot and socket joints for the transmission of large jacking forces. Project two, from 1989 to 1992, involved the monitoring of field behaviour of concrete pipes at full scale during pipe jacking on five active construction sites (Norris 1992, Milligan and Norris 1991, and Norris and Milligan 1992). Project two has obtained some important results from the field monitoring on pipeline alignment, tunnel stability, pipe-soil interface behaviour, pipe stress and ground movement (Milligan and Norris 1993a, Milligan and Norris 1993b, Milligan and Norris 1995, Milligan and Norris 1996).

Project three and project four were carried out after the success of the first two projects. The former involved a continuation of the site monitoring with a somewhat different emphasis and the latter mainly involved a numerical modelling of concrete pipe jacking to which this thesis refers. After identifying some possible design improvements from the numerical analysis and laboratory testing in the current research, further testing on these designs was carried out with prototype concrete pipes in project five (Holt et al 1997). Project five confirmed the

effect of local reinforcement and local prestressing on the improvement of pipe strength. Holt et al (1997) also found that the pipe's capacity was improved with one external prestressed steel band and that two external prestressed bands at the pipe joint would reduce the pipe's capacity. (The pipes used in this experiment were prototype pipes with butt-jointed incorporating steel collar.)

1.4 STRESSES / STRAINS IN THE PIPE

1.4.1 EXPERIMENTAL STUDY

To examine the performance of the concrete pipe in pipe jacking, Ripley (1989) carried out some laboratory tests with small scaled model pipes under both 'edge' and 'diagonal' loading conditions. The misalignment angle in the tests was provided by yokes. The strains at the middle length of the pipe were measured with strain gauges. The results showed that under 'edge' loading the strains were localized in the loaded region and the pipes failed at the pipe joint region. Under 'diagonal' loading high tensile strains existed on the external surface of the pipe and the pipes failed due to the cracking on the external surface along the line between the two loaded corners. In his research, some sand chamber tests were also carried out. In these tests, two concrete pipes were placed in the sand chamber with an initial misalignment angle between them. No packing material was used. The results showed that all the pipes came to an aligned position during the test instead of remaining in a misalignment position. The results were not consistent for different tests. Ripley (1989) concluded: 'The complexity and interaction of various loads applied to the pipe during this test series and the pipes constantly changing their orientation means that strain data from the various tests was not suitable for worthwhile comparisons between tests'.

Husein (1989), Boot and Husein (1991) examined the performance of the vitrified clay pipes in pipe jacking from the in-air testing and in-soil testing of a two pipe and packing

system as shown in Figures 1.3 and 1.4 respectively. From the in-air test, it was concluded that pipe failure was caused by tensile hoop strains in the joint region. The strains in the pipe were similar in the case of misalignment angle $\beta = 0.5^0$ and 1.0^0, which conflicted with the analytical theory (Haslem 1996, Concrete Pipe Association of Australia 1983). Strain distributions in the pipe were largely affected by the pipe end geometry under pure axial load and by the hoop geometry with pipeline misalignment. The prime attribute of a packing material was the ability to redistribute stress concentrations by compressing in a mechanically stable manner over a required level of deformation. For the in-soil tests, the results are not consistent. Husein pointed out: '... this has eventually led to serious concern regarding the validity of the strain gauge results ' and the possible reason was that '... it is suspected that defects in the jacking surface geometry may have seriously affected the strain results '.

1.4.2 NUMERICAL ANALYSIS

Husein (1989) also did numerical analysis by using the program ABAQUS. The analysis was mainly limited to his experimental model. In his in-air numerical model, the analyses were carried out with both 20-node brick elements and 8-node thick shell elements as shown in Figures 1.5 and 1.6 (due to the symmetric condition just a quarter of the system was used in this analysis). The interface elements were used to model the interaction between the pipe and the packing material. The initial position of the packer was determined so that a joint opening of 1^0 was obtained (0.5^0 in the analysis due to symmetric condition). The hoops were simulated by isoparametric beam elements and the contact between the pipe and the hoop was also modelled by interface elements.

The clay pipe was assumed elastic in the analysis. The packers were assumed to be elastic with the same Poisson's ratio as that of the clay pipe. The results (the strains on the internal and external pipe surface at the cross section of 20mm from the joint) showed good

agreement with this experimental data ($\beta = 0.5^0$). The axial strains were localized in the contact region (near the pivot) and the hoop strains showed periodic patterns along the line of r = constant with high maximum tensile values. The analysis was also extended to model different hoop geometries. In this analysis, the packers were modelled by a hyperelastic model (linear and with different Young's modulus at different stress stages). The actual Poisson's ratio was used for Chipboard and Medite fibreboard, and a value of 0.45 was used for Elastomer and LDPE. The results showed that hoop geometry had a great effect on the strains within the pipe, and that a packing material with a low Poisson's ratio (Chipboard) performs better than those with higher Poisson's ratio.

The in-soil (sand) numerical model consisted of one layer of brick elements of soil and the in-air model with shell elements but different misalignment angles. Again, the clay pipe and the packer were assumed to be linear elastic and to have the same value of Poisson's ratio. The interaction between the pipe and soil, and between the soil and fixed boundary were also simulated by interface elements. Lateral pressure was applied to model the pressure from the air bag in the laboratory tests. The soil (sand) was assumed to be linearly elastic because of the failure of convergence for both non-linear elastic and elastic-plastic soil models. 'The obtained final results were not as predicted ' as pointed out by Husein (1989). The numerical model was then modified, which included using non-linear springs to replace the interface between the soil and the fixed boundary, and using different soil stiffness at different regions according to radial stress variation. However, the results from this modified model showed significant tensile stresses on the pipe-soil interface. At last, two numerical techniques were used for further modification of this numerical model, that is, an initial strain in the packing material and a small stiffness for soil in the tensile zone. The results showed that the effect of the ground passive pressure was significant. Unfortunately, the results were still not satisfactory and there were discrepancies and conflicts between the numerical results and the experimental data.

The in-air numerical model in this research clearly showed the effectiveness of the finite element method. However, the numerical analysis was mainly a back analysis of this experimental data. The effect of the misalignment angle was not examined in this numerical analysis. Due to the significant effect of the hoop geometry, it is difficult to apply the results to fully explain the pipe behaviour in practice. Clearly, a numerical model with surrounding soil is needed. Unfortunately, due to the high non-linearity, the finite element mesh in this in-soil model was too coarse to obtain satisfactory results. To examine the behaviour of the pipe, more research is needed, for example, a parametric study of the influence of pipeline misalignment and the interaction between the pipe and surrounding soil.

1.5 PIPE JACKING LOADS

Jacking load is the force required to advance the complete pipeline forward, which depends on face resistance to penetration of the shield and the frictional force along the pipeline. It is important that jacking forces can be accurately predicted in practice to enable the operation to be designed appropriately. The amount of resistance encountered at the face depends upon ground conditions and the measures required to support the face. In hand drives it is solely due to the cutting edge resistance of the jacking shield and the friction acting on the external surface of the shield (Thomson 1993, Auld 1982). In machine drives the face pressure required to support the ground should also be taken into account. The principal factors affecting the jacking force include (Pipe Jacking Association 1995b, Rogers and Yonan 1992, Norris 1992, Ripley 1989, Thomson 1993):

- Resistance at the excavation face
- Type of soil and its variation along the pipeline
- Length, diameter and self-weight of the jacking pipes
- Depth of overburden and surface surcharge
- Amount of overcut during excavation

- Use of lubrication and intermediate jacking station
- Misalignment of the pipeline
- Jacking around curves
- Frequency and duration of stoppages

Whilst it is difficult to accurately assess the frictional force acting on the pipeline theoretically, pipe jacking contractors have, after years of experience, derived empirical values. The frictional forces usually fall between 0.5 and 2.5 tonnes per square metre of the external circumferential area of the pipe. Typical values of frictional forces for different ground conditions are given in Table 1.1 (Craig 1983):

Table 1.1 Typical values of frictional resistance

Soil type	Frictional resistance (kPa)
Rock	2 to 3
Boulder clay	5 to 18
Firm clay	5 to 20
Wet sand	10 to 15
Silt	5 to 20
Dry loose sand	25 to 45
Fill	up to 45

To predict the frictional force, Auld (1982) presented an analysis for a pipe driven through a cohesionless material, in which the soil was assumed to collapse onto the pipe and to exert radial pressure around its circumference. In this model, it was assumed there was no change of the vertical and horizontal soil stress with the change of the depth below ground level. The frictional force (F) per unit length is derived as following:

$$F = \pi R (\sigma_v + \sigma_h) \tan \phi_l \qquad (1.5.1)$$

Where R is the external radius of the pipe, σ_v and σ_h are the vertical and horizontal soil stress, ϕ_1 is the angle of friction between the pipe and the soil. Ripley (1989) modified this model to include the change of the soil stress with the changing depth:

$$F = \pi R (\sigma_{vc} + \sigma_{hc}) \tan \phi_1 \qquad (1.5.2)$$

Where σ_{vc} and σ_{hc} are the vertical and horizontal soil stress at the depth of the pipe centre. This model seems to overestimate the frictional force when comparing with experimental data (Norris 1992) possibly due to the overcut in practice.

Haslem (1986) studied the behaviour of pipe jacks in London Clay and proposed a model to estimate the frictional force in cohesive soil. The model was based on the behaviour of an elastic cylinder resting in a cylindrical void in an elastic continuum. The frictional force is given by:

$$F = \alpha S_u b \qquad (1.5.3)$$

Where α is the adhesion factor commonly used in pile skin-frictional formulae, S_u is the undrained shear strength of soil, b is the contact width. However, this model was found to underestimate the frictional force when comparing the measured data, possibly due to the soil collapsing onto the pipe in practice. O'Reilly and Rogers (1987) used a similar technique to predict the frictional force in clay and rock (sandstone). The equation for frictional force in their model is as follows:

$$F = \frac{W_p \tan \phi_1}{\cos \psi_1} \qquad (1.5.4)$$

Where W_p is the weight of pipe per unit length. ψ_1 is the offset angle of reaction from vertical. The model produced a good fit to the available data for pipejacks through rock. For pipejacks in clay, the model also gave an underestimate of frictional force. Laboratory tests were carried out to assess the contact area between pipes and clay. The results from experimental data confirmed a need to account for plastic behaviour of clay and the effect of time.

1.6 LOAD TRANSFER AT PIPE JOINTS

The distribution of load transferred across the pipe joint is mainly determined by the pipeline misalignment and the properties of the packing material used. The Concrete Pipe Association of Australia (1983) published a theory based on a linear stress approach and material properties as shown in Figure 1.7. In this model, the normal stress was assumed to be linearly distributed over the contact area on the joint. The stresses were constant in the pipe along the pipe length and in the packer throughout its thickness, and had the same distribution pattern as that on the interface. The axial strain in the pipe and the packer were linearly related to the normal stress at the joint, which in turn resulted in linear distributed deformation of the pipe and packer at the joint. The expression for joint misalignment angle β is given as follows:

$$\tan \beta = a \sigma_{max} / (E_j Z) \qquad (1.6.1)$$

$$E_j = (a t E_c E_p) / (a t E_c + L t_j E_p) \qquad (1.6.2)$$

Where a is the thickness of the packer, σ_{max} is the maximum normal stress on the joint, Z is the diametrical contact width as shown in Figure 1.7, L is the length of the pipe, E_c and E_p are the Young's modulus of the concrete and the packer respectively, t and t_j are the thickness of the pipe wall and the wall thickness at the joint. The Australian model has widely been adopted for its simplicity (Pipe Jacking Association 1995a).

To extend the Australian model to include the tensile stresses, Haslem (1996) proposed a flexible pipe model in which the stress distribution at the joint and the deformation of the packing material were the same as in the Australian model. However, the pipes were assumed to sustain tensile stresses and suffer uniform pressure and bending moment due to the eccentric jacking load at the joint. Moreover, it was assumed that the end of the pipes remained plane during bending and that the bending moment was constant along the two pipe halves adjacent to the joint. The misalignment angle of the pipeline β consisted of the angular deformation of the compressed packer (β_p) and the sum of the angular deflections (β_c) due to the bending of the two halves of the pipe abutting the joint:

$$\beta = \beta_p + 2\,\beta_c \qquad (1.6.3)$$

Where:

$$\beta_p = a\,\sigma_{max} / (E_p\,Z) \qquad (1.6.4)$$

$$\beta_c = \frac{M L}{2 E_c I_c} \qquad (1.6.5)$$

Where M is the bending moment due to the eccentric jacking load, and $I_c = \pi (R^4 - r^4) / 4$ is the second moment of area of the cross-section of the concrete pipe.

Another model to deal with the load transfer at the joint was proposed by Hornung et al (1987). In this model, no allowance was made for pipe elasticity and only the material properties of the packing material and orientation of the pipe were considered (Ripley 1989). In practice, there is also shear stress at the pipe joint (Norris 1992). However, due to its complicated nature, the shear stress is assumed to be zero in all these analytical models.

1.7 SCOPE OF THE CURRENT RESEARCH

Previous research (Norris 1992, Ripley 1989) suggested that concrete pipes could be under 'edge' loading or 'diagonal' loading conditions due to the misalignment of the pipeline as shown in Figure 1.8 and 1.9 respectively. Under 'edge' loading, pipes usually failed at the joint due to cracking with plane pipe and spalling with reinforced pipe (Ripley 1989, Holt et al 1997). Under 'diagonal' loading, failure of the pipes could be due to cracking on the external surface of the pipe along a line between the two loaded corners (Ripley 1989).

However, the failure model for concrete and the mechanism of the reinforcements are complex and beyond the scope of the current project. In the current research, the emphasis is

on the pipe performance under working conditions without reinforcement (except for the back analysis in Chapter 9) and to seek possible improvement on the design of pipe and pipe joint. The study is concentrated on the influence of the properties of the packing material, the properties of the pipe and the surrounding soil, and the misalignment angle at the pipe joint. Under the working condition, it is assumed that:

- Interface gaps may occur between the pipe and the surrounding soil
- Packing material may lose contact with the pipe over some area at the pipe joint
- Soil may yield
- Concrete pipe stays elastic

To assess the influence of different factors on the performance of the concrete pipe, stresses under working conditions are used as an indicator of safety against cracking failure. Concrete usually fails under tension due to the low tensile strength. The tensile strength reduces if accompanied by high principal compressive stress but remains the same under biaxial tension (Neville 1981, Jiang and Feng 1991). In this project, the results of the numerical analysis are used to identify the most probable damage zones of tension, and are interpreted by the most tensile principal stresses (the maximum principal stresses) and the most compressive principal stresses (the minimum principal stresses) within the concrete pipe.

This thesis starts with literature reviews in Chapter 1. Chapter 2 describes some developments of the finite element model used in the research. A two-dimensional preliminary study is discussed in Chapter 3. The simplified plane strain model in Chapter 3 is mainly used to obtain numerical experience of the use of interface elements and the non-linear solution procedure. Chapter 4 discusses the three-dimensional analysis using numerical model A, a single small scaled model pipe. The emphasis in Chapter 4 is on the effect of the load distribution and the thickness of pipe wall. The analysis using numerical model B, a full

scale pipe with surrounding soil, is described in Chapter 5. The parametric studies with this model are the interaction between the pipe and the surrounding soil due to the pipeline misalignment and the effect of the soil stiffness. A symmetric three-pipe system (three concrete pipes with two pipe joints and packing materials) is presented in Chapter 6. The analysis with this model is focused on the influence of the properties of the packing material and the influence of the pipeline misalignment. In Chapter 7, the deformation, the maximum normal stress and the diametric contact width at the pipe joint are discussed and compared with the data from the Australian model and Haslem's (1996) flexible pipe model. Chapter 8 describes the laboratory tests of improvements of pipe joint design. The numerical back analysis of the laboratory tests is discussed in Chapter 9. Finally, a summarisation of the research in this project and a few recommendations of future work are given in Chapter 10.

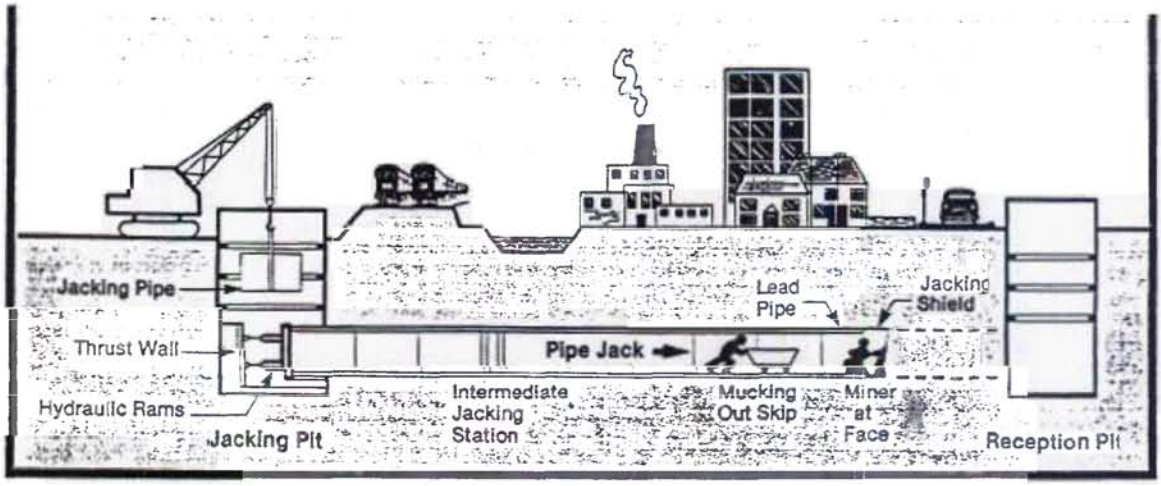
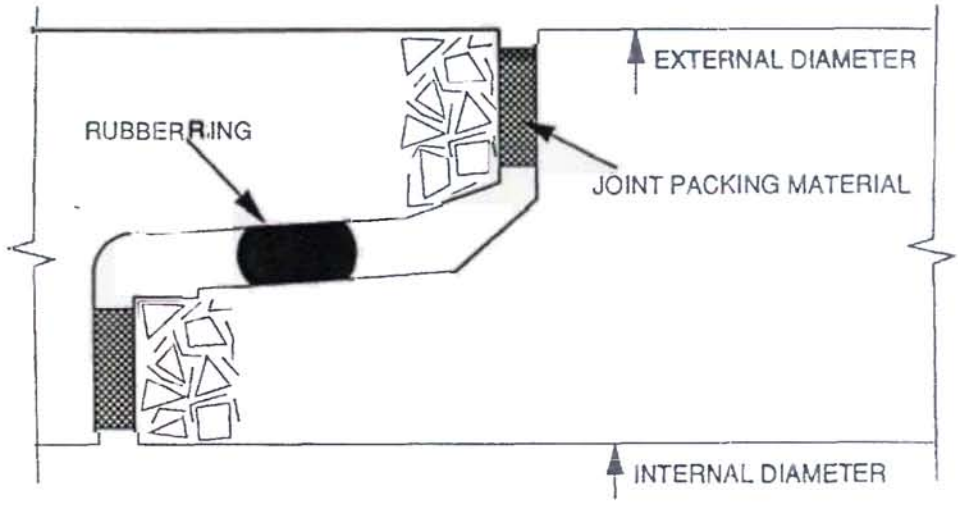
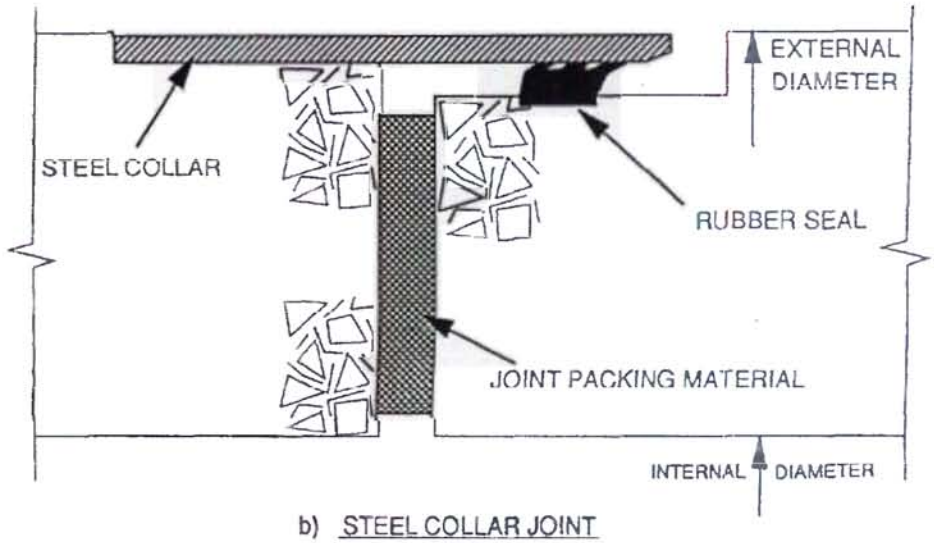


Figure 1.1 A typical arrangement of jacking equipment



a) IN WALL SPIGOT AND SOCKET JOINT



b) STEEL COLLAR JOINT

Figure 1.2 Types of pipe joint common in Britain

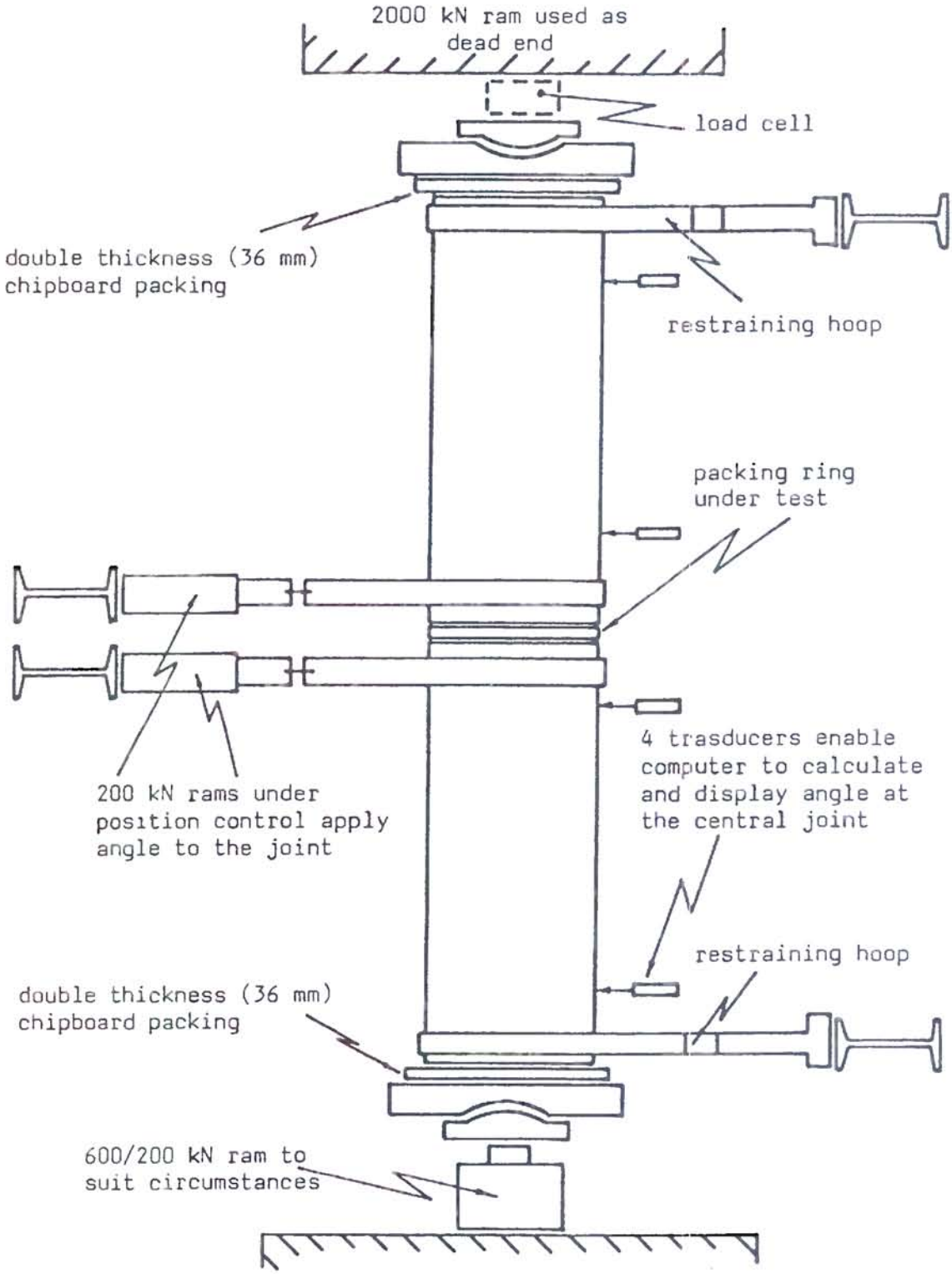


Figure 1.3 Apparatus in Husein's (1989) in-air test

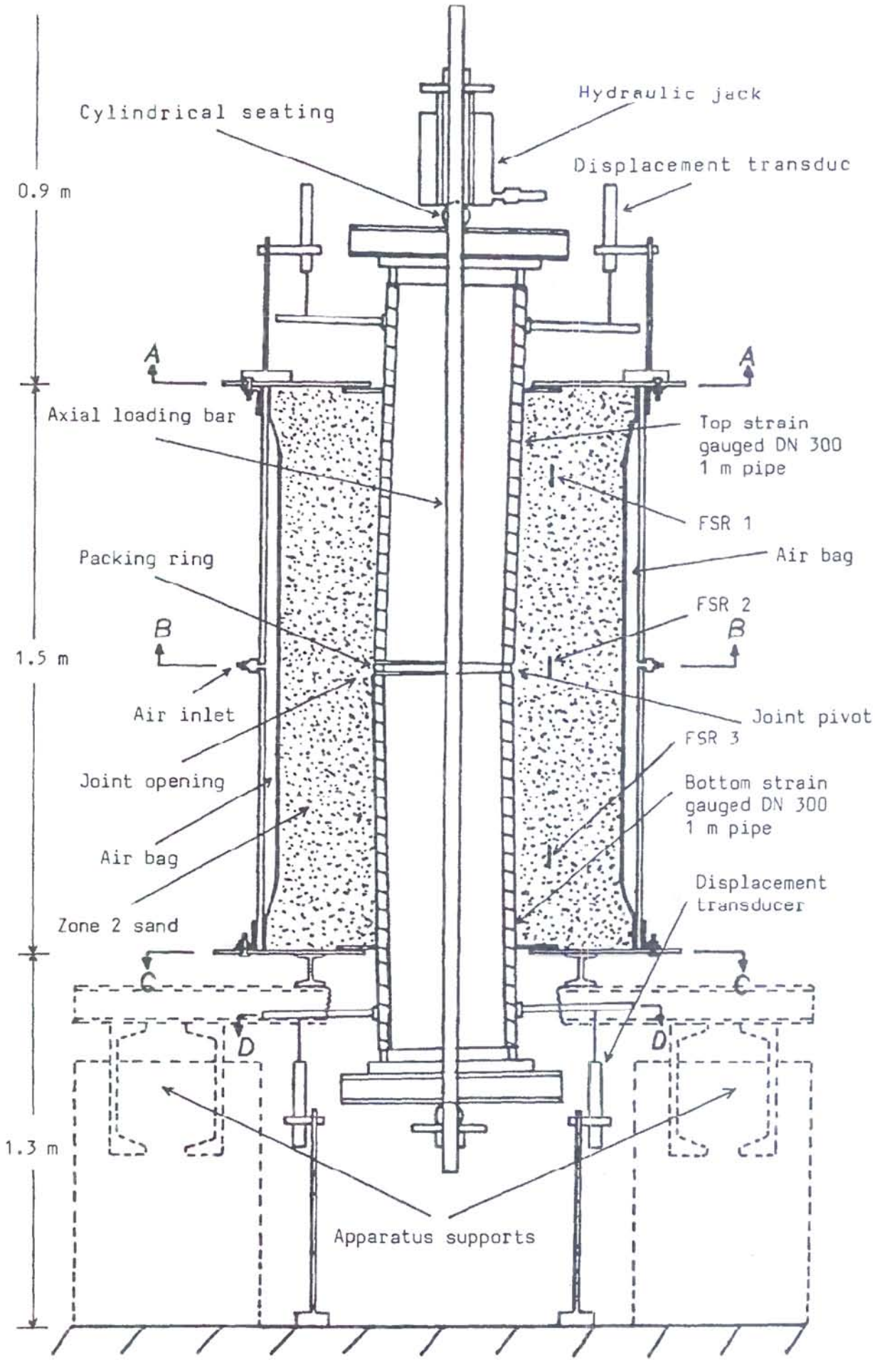


Figure 1.4 Apparatus in Husein's (1989) in-soil test

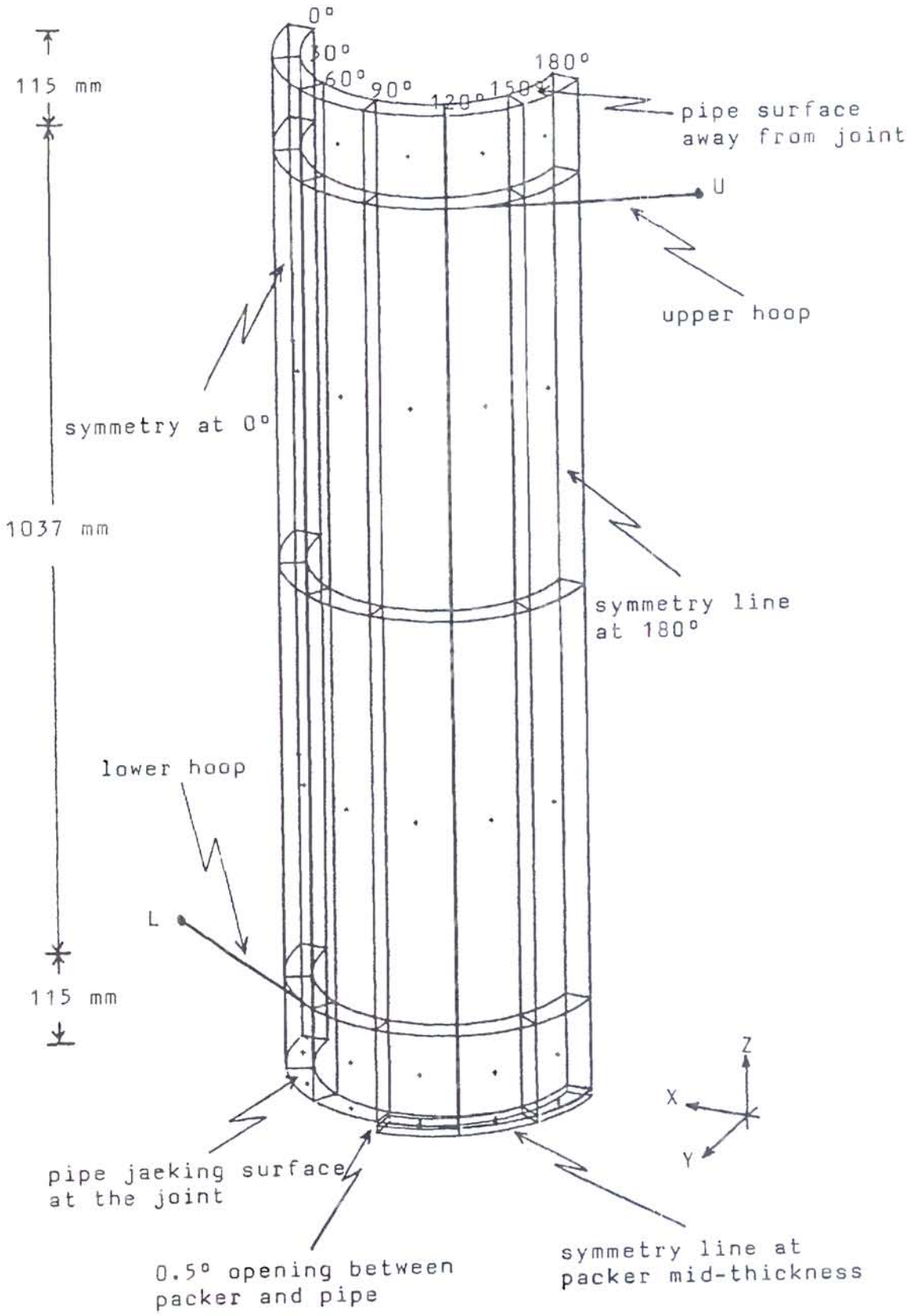


Figure 1.5 Numerical in-air model with brick elements (From Husein 1989)

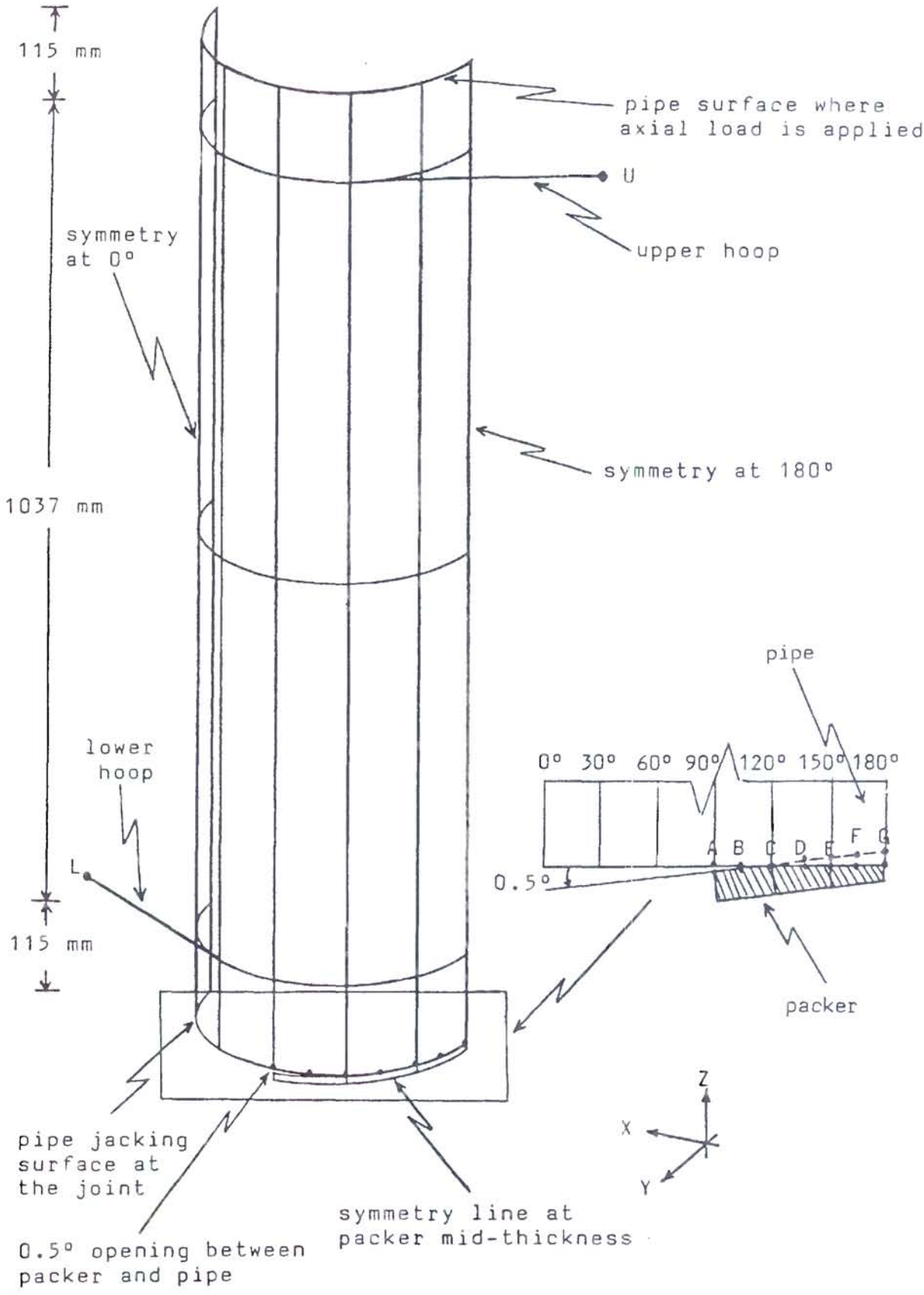


Figure 1.6 Numerical in-air model with shell elements (From Husein 1989)

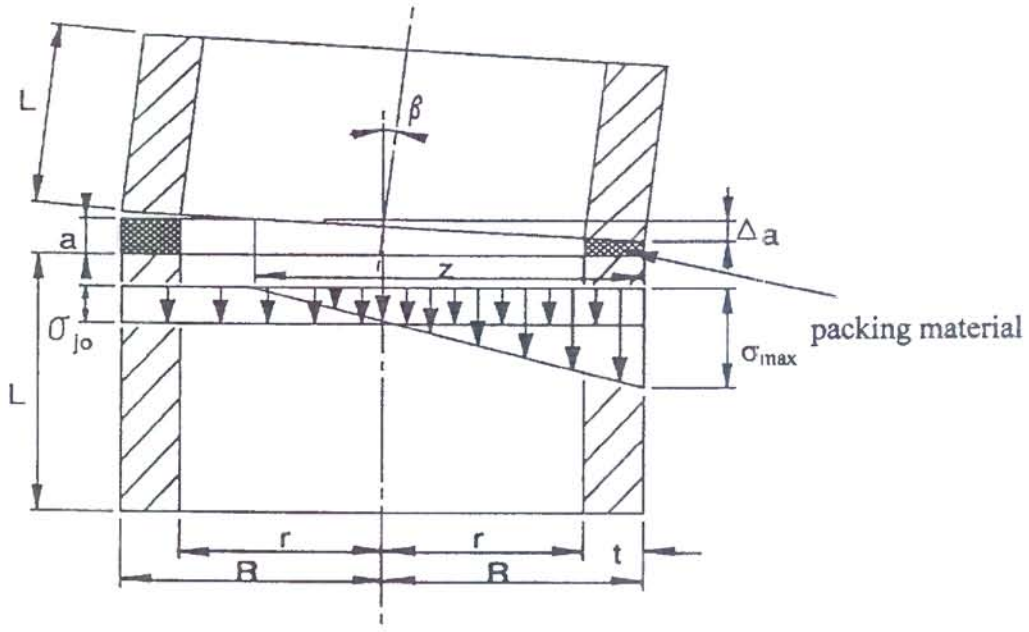


Figure 1.7 Australian model for stress distribution at joint

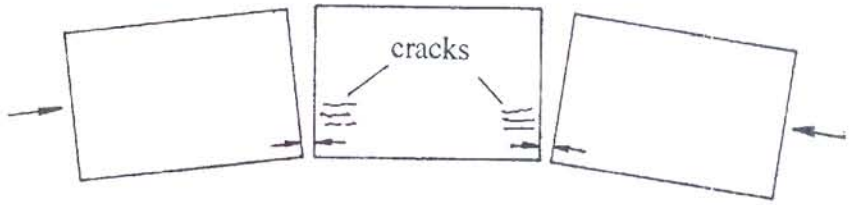


Figure 1.8 A typical edge loading condition

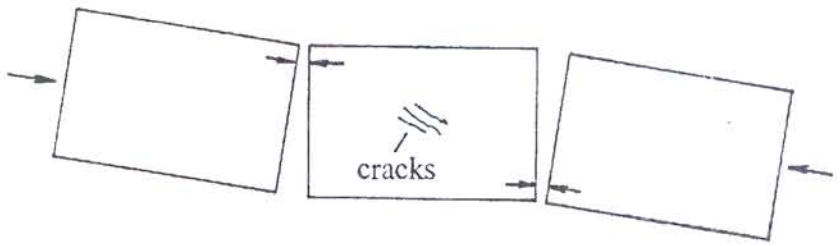


Figure 1.9 A typical diagonal loading condition