EARTH PRESSURES ON SPILLTHROUGH ABUTMENTS

By

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INTRODUCTORY NOTE

The Writers Contribution to the Project of Abutment Monitoring at Wisley

The writer joined the research team in the Civil Engineering Department, University of Surrey, in October 1982. At this time, all the boundary pressure cells had been installed on the spillthrough abutment at Wisley and the backfill was completed up to the top of the abutment columns. From this time until the end of construction the duties of recording the readings from the instrumentation were shared equally among the team members, namely Dr P Lindsell, Mr H Abdul Razak, Mr S Robinson and the writer. The writer was primarily concerned with the subsequent placement of the backfill until completion. During this period, the writer was responsible for installing several of the embedment pressure cells and for creating the protective sand pockets adjacent to the boundary cells. All other work reported within this thesis, including the density measurements and pressure cell calibration tests, have been predominantly the responsibility of the writer.

Each of the members of the research team were responsible for distinct aspects of the overall project as given below:

Dr P Lindsell - Team Leader, Lecturer
Mr T J M Kennie - Surveying, Lecturer
Mr S R Moore (writer) - Soil Mechanics and Earth Pressures, Research Student
Mr H A Razak - Concrete, Research Student
Mr S Robinson - Research Officer.

The contents of the above statement are agreed by the other members of the research team.

................................. .................................
The earth pressures exerted by cohesionless backfill against spillthrough abutments have been investigated by instrumenting two full size structures with vibrating wire earth pressure cells which were calibrated in soil under laboratory conditions. The abutment deformations were recorded with inclinometer tubes and precise surveying techniques, and the column bending was measured using vibrating wire strain gauges. The earth pressures were found to be influenced by the concrete expansion during hydration which caused transverse bending of the columns after the capping beam pour, as well as longitudinal backward rotations and bending of the abutments after the deck slab pours. High residual lateral earth pressures were exerted against the rear of the capping beams due to heavy compaction of the backfill at this level, thus causing the abutment to rotate forwards and become effectively propped by the deck slab. Traffic loading and deck slab temperature fluctuations were found to cause seasonal earth pressure variations. The lateral earth pressure profiles as predicted by the existing design methods were found to be totally unrepresentative of the Wisley results, and a modified design approach has been proposed.

Model tests were performed in the laboratory to investigate the behaviour of embedded laterally loaded columns within a spillthrough abutment. At small lateral displacements, the friction of the soil against the column sides was found to contribute significantly to the total soil resistance. Soil deformations were measured using specialised photography and the interaction between columns was found to be negligible.

A nuclear density probe was compared with other common methods of measuring the in-situ density of compacted backfill. A modified resin impregnation technique was developed to measure the density variations within a laboratory test specimen of dry sand.
For
Sue
Mum Dad
and Andrew
I would like to express my sincere gratitude to my supervisor, Dr Chris Clayton, whose enthusiasm, inspiration and guidance has been so willingly offered throughout the course of this work.

I would also like to acknowledge the contributions made to the overall study at Wisley by the other members of the team, namely, Dr Peter Lindsell, Mr Simon Robinson and Mr Hashim Abdul Razak.

I am thankful to the Science and Engineering Research Council who have provided the financial support to make this work possible.

I am indebted to Peter Haynes and Mike Jackman, whose technical advice and assistance has been invaluable in ensuring the smooth running of the laboratory experiments, and to Kevin Shaughnessy who assisted with the specialised photography. I am also grateful to the other technical staff within the Civil Engineering Department, namely, Paul, Martin, Ron, Ian, Eric, Dennis, Bob and Ken.

My debt to my fiancee, Sue, is immense for her support and encouragement and I cannot adequately reveal the time and care that she has put into the typing and production of this thesis.

Lastly, I cannot forget to mention the continual encouragement that I have received from my good friends within the Civil Engineering Department, namely, Norman, Colin W, Eric, Colin H, Bahman, Azam, Amin and Tony.
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CHAPTER 1

INTRODUCTION

1.1 General Introduction

Over the past 30 years, the motorway development programme in Great Britain has resulted in the construction of a large number of road bridges. The design of these structures has taken many forms, each of which has been evolved to satisfy the specific requirements at each individual location. Apart from the obvious variations in deck slab designs, closer inspection reveals a wide variety of abutment types.

In general, the types of abutment can be divided into two distinct categories. Firstly, there is the wall abutment category which describes the type of abutment which retains a vertical bank of soil at the roadside, as well as providing a bearing support for the deck slab, as shown in Figure 1.1a. The second category consists of the open types of abutment. These vary from the wall abutments by being constructed away from the roadside and are generally sited within the approach road embankment, thus resulting in a larger length of bridge deck because of the additional span required to reach the abutment, as illustrated in Figure 1.1b.

There are a number of basic types of open abutment that are commonly used, such as bankseats, bankseats on piles, buried walls and spillthrough abutments. It is the spillthrough type of abutment that is the subject of this investigation, and a typical bridge supported on such abutments at Wisley, Surrey, is shown in Plate 1.1. It is this structure that has been considered in detail in this thesis.

The general form of a spillthrough abutment consists of a capping beam with a curtain wall, upon which rests the deck slab. The dead loads are transmitted vertically downwards through the embankment from the capping beam to a foundation base slab, via a series of concrete columns. The embankment at the rear is retained by the capping beam up to road level, and at the front, slopes downwards at a gradient of approximately 1 in 2 to the level of the lower carriageway. Cantilever wing walls are
commonly included to retain the upper levels of the embankment and so limit the width of the abutment. Figure 1.2 illustrates the typical form of the spillthrough abutment that is considered in this study.

The decision of whether to adopt a spillthrough type of abutment is highly dependent on the requirements of the particular situation. The most important factors that are often contemplated when considering the use of a spillthrough abutment, are briefly as follows:

(i) The bearing level of spillthrough abutments can be sited at the existing ground level, provided the soil offers an adequate bearing capacity, unlike bankseat abutments which depend on adequate backfill compaction beneath. Therefore, spillthrough abutments are generally less susceptible to settlement problems.

(ii) The construction of the concrete abutment and embankment can be carried out in alternate stages, thus minimising the amount of falsework required. Alternatively, the concrete structure can be completed before commencing the construction of the embankment, although this method often results in difficulty in compacting the soil between the columns due to the presence of the capping beam.

(iii) The appearance is much less obtrusive than solid wall abutments. Spillthrough abutments allow considerably better visibility which is particularly important for the design of road bridges.

(iv) Generally, less concrete is required for the construction of spillthrough abutments than for solid wall abutments. However, any saving in the cost of concrete must be considered in relation to the added cost of more elaborate formwork and an additional span of bridge deck at each end of the bridge. Furthermore, they can often prove to be a cheaper solution than bankseats supported on expensive piles. In addition, only very small wing walls are required to prevent soil from affecting the bearings.

There are a large number of loadings that must be considered during the design of an abutment, and these must be estimated as accurately as possible so as to produce the most suitable structure. The following loadings are those most commonly considered.
(i) Earth pressure
(ii) Traffic loading
(iii) Bearing friction
(iv) Wind loading
(v) Impact loading
(vi) Self-weight of structure.

Nowadays, the majority of these loadings have been fairly well defined as a result of continued research. However, for the case of spillthrough abutments, the definition of the earth pressures has until now remained rather vague. It would appear that this may be partly due to the fact that the common reason for using spillthrough abutments is based on the assumption that the soil will tend to flow rather than exert lateral pressure if openings exist in the structure. The uncertainty relating to the prediction of earth pressures on spillthrough abutments was highlighted by a survey carried out by the Building Research Establishment, (Hambly (1979)). It was reported that a survey of 20 statements of practice revealed 12 different arbitrary methods of design. The design rules varied between the two extremes of considering the abutment to support a total lateral load equivalent to a solid wall and considering no lateral loads for embankment slopes of 1 in 2 or shallower. It appears that none of the methods have been based on experimental observations and are therefore thought to be rather speculative.

Consequently, in 1981, the Department of Transport initiated a full research programme into the design of spillthrough abutments. A major part of this study was undertaken by the University of Surrey and constituted the monitoring of two full size spillthrough abutments supporting a new M25 overpass bridge at Wisley, Surrey. In the next section of this chapter, the main objectives of this full size investigation are outlined. In addition, a brief outline is given of the laboratory investigations that were performed to investigate particular aspects of the behaviour of spillthrough abutments. The final section of this chapter describes the format of the thesis.
1.2 Scope Of The Present Work

The prime objective of the work reported within this thesis has been to investigate the behaviour of spillthrough abutments, with particular reference to the prediction of earth pressures. A considerable proportion of the work has been involved with the full size monitoring of two spillthrough abutments at Wisley, as part of the research contract awarded to the University of Surrey by the Department of Transport. Furthermore, a selection of laboratory model tests were performed to investigate, in more detail, certain aspects that were thought to be relevant to the behaviour of spillthrough abutments but which were inadequately defined in the full size investigation.

The instrumentation of the two full size abutments at Wisley consisted firstly, of a series of boundary earth pressure cells installed over the surface of the abutments and of embedment earth pressure cells positioned within the embankment. Secondly, strain gauges were installed within the reinforced concrete abutments to record the bending effects of the columns. Finally, the deformation characteristics of the abutments were recorded by a precise surveying technique and also by an inclinometer tube. Further pressure cell information was obtained from an additional series of pressure cells that were installed by the Transport and Road Research Laboratory (TRRL).

Together, this instrumentation was designed to provide detailed information pertaining to the understanding of the abutment behaviour, both during construction and during subsequent traffic loading. Of particular interest, was the development of earth pressures in relation to the effects of compaction and the movements of the abutments.

In order to obtain the best estimate of the earth pressures recorded by the pressure cells, it was necessary to perform a series of calibration tests within the laboratory. These tests investigated the cell response for vibrating wire pressure cells with different diaphragm flexibilities and for the pneumatic cells, as were used by the TRRL. After having established a suitable method of calibration, the effects of hysteresis and cyclic loading were also investigated, so as to be able to represent, as closely as possible, the effects of the pressure variations that were observed for the cells installed at Wisley.
A considerable amount of time was spent determining the density of the backfill placed around the abutments at Wisley. This involved a number of density measurements using sand and water replacement tests and a nuclear density probe.

The effects of friction of the soil against the sides of the columns and the effects of column interaction were not obvious from the full size investigation. Therefore, it was decided to investigate these factors separately in the laboratory. A series of model tests were designed to assess the contribution of soil resistance against the side and front faces of a translating column. These tests also provided an indication of the zone of influence created within the soil at the front of a translating column, which was relevant to the investigation of column interaction. Additional tests were performed to study the influence of a base slab on the deformation characteristics of a single laterally loaded column embedded in a mass of soil. Finally, the development of the frictional resistance at a soil/structure interface was investigated by a series of shear box tests and pull-out tests.

In order that all the model tests should be repeatable and comparable with one another, it was necessary to implement a method of creating a uniform specimen of dry sand. This led to the immediate need to find a suitable technique for measuring the density variations within a sand specimen. Consequently, a resin impregnation technique was developed, based on a previous technique described by Griffen (1954). Thereafter, a suitable method of compaction was formulated by trial and error, which produced a uniform specimen of sand.

Having accumulated all the information from the experimental investigations, it was then possible to check the validity of the existing design methods and to compare the results with a proposed modified form of analysis based on the observed trends.

The final section of this introduction provides a general description of the subject matter contained within the remainder of this thesis.
1.3 Composition Of The Thesis

In the following chapter, Chapter 2, a review of existing literature has been presented, which is considered to be pertinent to the topic of this thesis. It would appear that although spillthrough abutments are now widely used in this country, there has not until recently been any detailed research into their behaviour. Therefore, as the initial insight into their behaviour was so limited, it has been necessary to review a wide range of literature which was thought to be relevant to the present work. Apart from an appraisal of the existing design methods for spillthrough abutments, the design of earth retaining structures has also been considered, with particular reference to aspects such as compaction effects, wall friction, deformation characteristics and long term effects. Furthermore, the design of laterally loaded piles embedded within a mass of soil has been discussed, as well as the vertical loading characteristics of friction piles. The effects of interaction between adjacent embedded structures has been described with reference to past research on pile groups and lines of buried anchor plates. A further section of the chapter reviews the techniques used for measuring the density of cohesionless soil. The final section discusses the performance of earth pressure cells, as reported by previous researchers.

In Chapter 3, the details of the experimentation performed during the course of this study are presented. The first section includes a description of the instrumentation used at the Wisley abutments. In addition, the method of calibrating the earth pressure cells is presented and the results are discussed. The final part of this section describes the methods used for measuring the density of the backfill at Wisley, and includes a report of the findings. The second section of Chapter 3 gives detailed descriptions of the development of the laboratory model tests and the corresponding test procedures that were adopted. Furthermore, the resin impregnation technique that was developed for measuring the dry density of sand is explained and the results of its applications are presented.

The results from the experimental investigations are presented and discussed in Chapter 4. Firstly, the findings of the Wisley study are discussed in detail in accordance with particular periods of construction and in the long term. Secondly, the findings of the laboratory model tests
are discussed with reference to particular aspects of the interaction between a laterally loaded column and the surrounding soil within which it is embedded. This has included a discussion of the frictional characteristics of a soil/structure interface, the effects of column aspect ratio on the soil resistance and the zone of influence within the soil. The final section of the chapter attempts to consolidate the information gained from the experimental investigation and assess its implications on the design of spillthrough abutments in general. As a result a comparison is made between the existing methods of design and a proposed modified approach.

In the final chapter of this thesis, Chapter 5, the main conclusions from the present work are summarised. In addition, some suggestions are made as to the possible directions of future research which may provide a beneficial addition to the existing state of knowledge.

1.4 Summary

This chapter has provided a general description of the form and application of a spilsthrough abutment. In addition, the main objectives of the present work have been briefly discussed. Finally, the organisation of the thesis has been outlined with a brief description of the subject matter contained within each chapter.

The following chapter contains a detailed review of the existing literature which has been considered to be relevant to this course of study.
Figure 1.1 Types of bridge abutment

Figure 1.2 Typical form of a spillthrough abutment
CHAPTER 2

LITERATURE REVIEW

This chapter presents a review of the existing literature which is considered to be relevant to the present work. As such, it contains a review of a variety of topics relating to the prediction of earth pressures on spillthrough abutments. In addition, the previous work concerning laboratory density measurement techniques for cohesionless soil is reviewed. Finally, a brief review is given of the factors which have, in the past, been found to influence the performance of earth pressure cells.

The following section discusses the previous work by other authors which it is felt may contribute to a better understanding of the behaviour of spillthrough abutments constructed in a cohesionless backfill.

2.1 Studies Relevant To The Prediction Of Earth Pressures Exerted Against Spillthrough Abutments

2.1.1 Introduction

The need for the present study has evolved from the lack of existing information relating specifically to the prediction of earth pressures against spillthrough abutments. Therefore, in order to obtain a broader understanding of the possible factors involved with the behaviour of spillthrough abutments it has been necessary to review topics associated with other aspects of soil/structure interaction. In particular, this has involved a study of the factors which have been found to influence the generation of earth pressures against earth retaining walls and piled foundations and these are reviewed in the following sections.

2.1.2 Traditional Methods Of Retaining Wall Design

It has for a long time been realised that the earth pressures acting on retaining walls is highly dependent on the manner in which the structure moves relative to the soil mass. Two limiting states of pressure are considered to be caused by a wall either moving
towards a mass of soil to produce a 'passive' state of failure or moving away from a mass of soil to produce an 'active' state of failure. An intermediate condition known as the 'at-rest' condition is also considered in which the wall is assumed to be perfectly rigid (ie, restrained from moving laterally), as in the case for strutted excavations or rigid basements.

One common approach for evaluating the earth pressures acting on earth retaining structures is to consider the value of an earth pressure coefficient, $K$, which equals the ratio of horizontal to vertical effective stress within the soil. This approach results in a lateral pressure that increases linearly with depth for a soil of constant density. The value of the earth pressure coefficient has been found to vary vastly, depending on the nature of the movement of the structure, as shown in the Figure 2.1, (Lambe and Whiteman (1979)). From this figure it can also be seen that the amount of lateral movement required to reach the limiting values of earth pressure varies according to the direction of movement. The magnitude of the movement is related to the ratio of lateral displacement, $S$, to the height of the structure below the soil surface, $H$. The corresponding values of $S/H$ required to mobilise the limiting states are found to be much smaller for the active condition than the passive condition. The precise values of $S/H$ have been found to vary according to the soil properties and the type of movement of the structure (ie, whether rotational or translational movement) and this is discussed in more detail in section 2.1.5.

The design of earth retaining structures today, is based to a large extent on the classical theories proposed by Coulomb in 1776 (translated into English by Heyman (1972)) and Rankine in 1857, although continued research has led to considerable refinements and extensions of the original theories.

Both of the above theories result in a hydrostatic pressure distribution for a cohesionless soil which carries no surcharge loading. A critical analysis of the above two theories was made by Terzaghi (1936), in which it was concluded that Rankine's theory did
not represent an attainable state of stress within the soil and should thus be disregarded. It was also concluded that the hydrostatic pressure distribution, as obtained by Coulomb for the active case in dense soil, was only valid if the upper rim of the wall moved laterally by at least 0.0005H. If this was not the case, it was suggested that a non-hydrostatic distribution may be created due to arching effects. Terzaghi suggested that for loose sands an average yield somewhat greater than 0.0005H was necessary to produce the limiting Coulomb pressure, although the pressure distribution remained approximately hydrostatic regardless of the lateral movement.

Rankine's theory has been extended by Resal (1910) and Bell (1915) to cater for soils with both friction and cohesion. Coulomb's theory has been extended to give a similar analysis for passive pressure conditions but in this case, the shear resistance of the soil on the failure surface acts in the opposite direction to that for active conditions. Mayniel (1808) included the influence of soil cohesion in the analysis and a simple modification by Muller-Breslau (1906) allowed for the conditions of a sloping soil surface and a sloping back to the wall. The Coulomb method has been refined by considering further the effect of a curved failure surface which is particularly relevant to the passive state of failure. The values of the earth pressure coefficients have been tabulated in the Civil Engineering Code of Practice No2 (1951) and are shown in Table 2.1. Caquot and Kerisel (1948) have produced a set of tables for evaluating the values of $K_a$ and $K_p$ based on a failure surface consisting of a combination of a log spiral and a plane.

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*Table 2.1 Earth pressure coefficients for cohesionless soils (from CP2 (1951)*
The coefficient of earth pressure at-rest can be calculated from the theory of elasticity and depends solely on Poisson's ratio, $\nu$, of the soil, such that

$$K_0 = \frac{\nu}{1 - \nu}$$

Additional theories based on soil strength properties have been proposed by Kezdi (1966) and Sowada (1968 and 1969) but the most used theory is a simplified form of the original equations proposed by Jaky (1944), which estimates $K_0$ for a sand and a normally consolidated clay as

$$K_0 = 1 - \sin \theta'$$

where, $\theta'$ = effective angle of internal friction

Typical values of $K_0$ based on the above equation range from 0.3 to 0.6 for dense and loose sands respectively.

The design of earth retaining structures based on the 'classical' theories of Rankine and Coulomb has until recently generally been adequate for most situations. However, in recent years the materials and methods of construction have altered considerably and it is now felt that the underlying assumptions of the 'old' theories may be somewhat inaccurate. One of the major changes has been the introduction of heavy vibrating rollers for compacting the soils immediately behind the retaining structures, the effects of which are discussed in the following section.

### 2.1.3 Experimental Investigations Of The Effects Of Compaction

In recent years, it has become very common to use heavy compaction plant to densify the backfill material behind earth retaining structures. This has been particularly relevant to bridge abutments where it is desirable to achieve a near maximum density of the backfill, so as to limit the subsequent settlement and maintain an even road surface. The effect of the intensive
compaction is to create very large stresses within the soil, a proportion of which remain as residual stresses after completion. The effects of compaction are further complicated by the method of placing and compacting the fill in a series of horizontal layers. The conventional approach used to be to prop the retaining structures during the placement of backfill. However, this is generally no longer the case, except for basement walls and some bridge abutments. The lack of propping results in progressive deformation of the structure as the level of backfill is raised, (Sims et al (1970), Casagrande (1973)). Therefore, the original assumption of an active earth pressure being exerted after the removal of the props at the end of construction is no longer valid. A study of the changes in retaining wall design over the past 150 years has been reported by Jones (1979).

In the past, it has been conventional to assume that the retained soil exerts a hydrostatic earth pressure on the rear face of the wall. The magnitude of this pressure, based on classical theories, has been determined by applying an earth pressure coefficient to the vertical overburden pressure, typically those recommended in the Civil Engineering Code of Practice No 2 (1951).

Evidence of the incorrect assumption of a hydrostatic pressure has been obtained as a result of model tests and full size tests, designed to investigate the effects of compaction. The lateral pressures exerted by tamping sand into a 5ft x 8ft x 5ft deep (1.5m x 2.4m x 1.5m deep) concrete lined test pit were measured by Sowers et al (1957). The results indicated that the residual pressures exceeded the at-rest pressures and were considerably larger than the pressures measured if the sand was dumped loosely. Davies and Stephens (1966) investigated the lateral pressures exerted on the walls of a 24" (0.6m) cubic container due to the compaction of a cohesionless soil within it. The lateral pressures were found to increase significantly in the upper region of the container.

The lateral pressures on a 1500ft (457.2m) long reinforced cantilever wall at Grange Mill Lane, which had been backfilled with conditioned hopper ash, were monitored by Sims et al (1970). A
A series of earth pressure cells were placed in the backfill, at a distance of 2ft (0.6m) behind the wall and at different levels, to record the lateral and vertical pressures. The initial pressures after compaction indicated an approximately rectangular pressure distribution of $51b/in^2$ (34kPa) up the back of the wall. This significantly exceeded any predictions that could reasonably be made using classical theories.

A study of the pressure exerted by a fine sand, on seven purpose built 18ft (5.5m), high cantilever walls supported on 'H' piles, was reported by Coyle et al (1974). The sand was compacted in 8 inch (200mm) layers by three passes of a bulldozer per lift. The pressures were found to be 50% greater than the active pressures over the upper 7ft (21m) of the wall. In the lower region the pressure increased further still and tended towards at-rest conditions, assuming $K_o = 0.8$. The high pressures at the base were accounted for by the small lateral movements that were measured in this region.

A subsequent investigation on a precast panel wall supported on drilled piers was reported by Coyle (1977). The panel was held in position at each end by a T-shaped pilaster extending upwards from the footing. The recorded soil pressures varied across the width of the panel. At the edges, near a pilaster, a high pressure was recorded at the base but less than active pressures were recorded towards the top. The central section of the panel indicated a greater than active pressure at the top and a lower than active pressure towards the base.

An experimental retaining wall facility at the Transport and Road Research Laboratory was used to measure the residual stresses after compaction of a washed sand, (Carder et al (1977)). A central 2m square steel panel was instrumented with three types of pressure cell, as was the rigid end wall of the concrete trough within which it was located. The sand was placed in 0.15m layers and was compacted by six passes per layer of a 1.3Mg twin-roll vibrating roller. The measured residual pressure was found to be
approximately constant with depth at a value of 15kPa. The pressures were considerably greater than the at-rest pressures calculated from Jaky's simplified equation, where \( K_\theta = 1 - \sin \theta \). The pressures on the concrete end wall were noted to be slightly greater than on the steel test panel as a result of the greater rigidity of the end wall. The performance of a conventional cantilever retaining wall was reported by Ingold (1979b). The wall was 70m long and built at two heights of 7.8m and 5.7m. The very silty, slightly sandy clay and gravel fill was placed in 0.4m thick layers and was compacted by four passes per layer of a Stothert and Pitt 54T vibrating smooth-wheeled towed roller. The wall was observed to be 94mm out of plumb at the top after completion of the backfilling behind it. Excavation of the soil behind the wall revealed a 2mm wide crack running along the base of the wall stem near its junction with the base slab, thus indicating that the resistance of the wall of 315kNm must have been closely approached. However, the corresponding calculated induced bending moment, based on the measured soil parameters and classical earth pressure theory, was only 12kNm, illustrating the theory's inability to account for the effects of compaction.

The above investigations have been related to compaction pressures on retaining walls which are able to rotate about the base. However, if the compaction pressures on non-yielding structures such as bridge abutments and basement walls are now looked at, it would be reasonable to assume that they would be predicted even more incorrectly by the conventional earth pressure theories.

The earth pressures exerted on the abutments of 152.5m and 110m long rigid frame bridges has been reported by Broms and Ingelson (1971 and 1972 respectively). The details of construction are given in Table 2.2.
<table>
<thead>
<tr>
<th>Length of bridge</th>
<th>152.5m</th>
<th>110m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Height of retained material</td>
<td>4.2m</td>
<td>9.4m</td>
</tr>
<tr>
<td>Thickness of compaction layer</td>
<td>52.0cm</td>
<td>60.0cm</td>
</tr>
<tr>
<td>Compaction plant</td>
<td>3.8 tons (10 passes)</td>
<td>3.0 ton</td>
</tr>
<tr>
<td>(vibrating roller)</td>
<td>uniform sand</td>
<td>uniform sand overlying sandy gravel</td>
</tr>
<tr>
<td>Backfill material</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 2.2 Details of instrumented abutments  
(Broms and Ingelson 1971 & 1972)

In each case, the abutments were supported on drilled piers or founded directly onto rock, thus effectively preventing horizontal and vertical displacements of the base. The lateral movement at the top was restricted during compaction by the lateral restraint offered by the connected deck slab. The soil was compacted immediately adjacent to the abutment by plate compactors. The lateral earth pressures after compaction for the 152.5m long bridge were found to have a parabolic distribution with near active pressures at the base. The 110m long bridge experienced lateral pressures varying linearly with depth corresponding to a lateral earth pressure coefficient equal to 0.4. The earth pressures exerted on four standard motorway bridge abutments with cantilever wing walls, which were backfilled with various materials, have been reported by Jones and Sims (1975). The abutments to the south bridge were backfilled with Bunter sandstone and the north abutments were backfilled with a site mixture of limestone and Bunter sandstone. The fill at the south abutments was compacted in 150mm thick layers with sheepsfoot or grid rollers and at the north abutments by vibrating rollers. Pressure cells were used to record the pressure on the rear face of the abutments and the wing walls. It was found that the pressures were parabolic in distribution between the wing walls and were generally much larger than the pressures predicted by an equivalent fluid pressure of 30lb/ft^3 (4.7kN/m^3) with an HB surcharge. High lateral pressures were recorded near the base due
to the wedging action of the compacted backfill between the abutment and the surrounding existing intact soil. A decrease in pressure was observed at a level beneath the soffit of the wing walls due to the lack of confinement in this region. The pressures measured on the wing walls were constant at 700lb/ft² (3.4kPa) but reduced towards the soffit level due to poor compaction. The magnitude and distribution of the pressures on the rear face of all four abutments were consistent regardless of the type of backfill or its method of placement. However, the wing walls were found to have a significant effect on the residual pressures due to their containing action.

Casagrande (1973) reported the results of the earth pressure measurements on some bridge abutments in Germany. An abutment supported on piles and backfilled with granulated slag experienced a pressure distribution far greater than the active pressure distribution, (Muller (1939)). Two abutments monitored by Siedek (1969) and von Becker (1970) indicated at-rest pressures due to a lightly compacted granular fill.

The lateral pressure exerted on basement walls by the placement of a granular backfill material has been investigated by Rehnman and Broms (1972). The tests were performed on a 6m long by 2.5m high reinforced concrete wall in a sand pit. Two backfill materials were used, gravelly sand and silty fine sand, and were compacted with four passes of either a 400kg or 140kg vibratory plate compactor. The layer thicknesses were 20cm and 40cm for the 140kg and 400 kg compactors respectively. The earth pressures were measured with an array of pressure cells mounted on the wall. The wall was supported at the top and bottom during the compaction process. The pressure distribution was found to be approximately rectangular for the gravelly sand with a magnitude varying from 5kN/m² to 11kN/m² for the 140kg and 400kg compactors respectively. The pressures were found to decrease with depth for the silty fine sand compacted by the 400kg compactor. The pressures decreased from 10kN/m² at the top to 5kN/m² at the base of the wall. The decrease of pressure with depth was attributed to the tilting of the wall which was observed to be up to 0.0008 radians.
It can be seen from the above studies that the use of modern compaction plant causes earth pressures to be exerted on structures which exceed those predicted by the classical theories. This therefore means that in order to accurately predict the lateral earth pressures exerted by a compacted backfill, new theories must be developed to account for the effects of compaction. Some recently developed theories which attempt to do this are discussed in the following section.

2.1.4 Theoretical Prediction Of The Effects Of Compaction

The case histories clearly illustrate the need for modified design theories to take account of the residual lateral pressures caused by compaction of the backfill. Consequently, a number of design theories have been proposed by Broms (1971), Aggour and Brown (1974) and Ingold (1979a).

A theory for calculating the residual lateral earth pressures after compaction of the soil against rigid unyielding structures was proposed by Broms (1971). The vertical stress increase due to the effective weight of a vibratory compactor was found to be twice that created by a static roller of the same weight, (Whiffen (1954)). A separate study by Forssblad (1963) indicated that the vertical stress distribution below a vibratory compactor was closely approximated by the Boussinesq stress equation when the relative density of the sand is high. Broms therefore assumed that the maximum vertical stress increase, $\bar{\sigma}_v\max$, could be calculated by the Boussinesq equation and that the corresponding maximum lateral earth pressure, $\bar{\sigma}_{ho\ max}$, was equal to $K_0\bar{\sigma}_{v\max}$, where $K_0$ equals the at-rest earth pressure coefficient during loading. When the compactor load is removed the vertical stress is partially relieved. As shown in Figure 2.3, the lateral stress is assumed to remain at its maximum value unless the vertical stress, $\bar{\sigma}_v$, is less than $\bar{\sigma}_{ho\ max}/K'_o$, where $K'_o$ equals the earth pressure coefficient on unloading. If the vertical stress is less than this value, then the residual lateral pressure, $\bar{\sigma}_{ho}$, is given by $K'_o\bar{\sigma}_v$. The depth of soil resulting in a vertical overburden pressure, $\bar{\sigma}_v$, equal to $\bar{\sigma}_{ho\ max}/K'_o$ is defined as the critical depth $Z_{cr}$. When the compaction consists
of a series of layers, then this distribution is superimposed successively for each layer to give a resultant pressure distribution as shown in Figure 2.4. At sufficiently large depths, the lateral pressure exceeds the maximum residual lateral pressure, caused by compaction and is calculated as \( \delta_h = K_o \delta_v \).

The theory derived by Ingold (1979a) assumes a similar idealised stress path to that used by Broms (1971), except that Ingold's theory assumes that there is sufficient lateral yield of the structure to mobilise the shear strength of the soil. An expression derived by Holl (1941) has been used to give the vertical stress vertically below a line load, \( p \), at the ground surface. By equating the lateral pressure derived from (i) the active pressure coefficient applied to the vertical effective overburden pressure and (ii) Holl's expression at the initial depth, a new equation is obtained giving the residual lateral pressure, \( \delta_{hm} \), as

\[
\delta_{hm} = \sqrt{2p\gamma/w} \quad \text{where} \quad \gamma = \text{unit weight of soil} \\
p = \text{effective surface line load}
\]

The conventional active pressure is assumed to act at depths where it exceeds the compaction induced pressure. The theory for yielding walls has been adapted by Ingold (1979c) to apply to non-yielding walls. The resulting equation for the residual lateral earth pressure is essentially equivalent to that obtained by Broms (1971).

A rigorous theoretical method for analysing the effects of compaction has been developed by Aggour and Brown (1974). The solution was developed using an incremental elasticity formulation (Brown and Goodman (1963)) and was solved with a computer using a finite element iteration process developed by Aggour (1972). The method enables the calculation of the magnitude of pressure and deflection during the period of compaction and also of the pressure remaining in the soil after compaction. The analysis takes account of the wall and backfill geometry, the wall flexibility, the degree of compaction and the wall boundary conditions. A subsequent study by
Aggour and Brown (1979) has compared the pressure distributions obtained by this method with the experimental data reported by other authors. It was shown to produce a fairly good correlation to the measured values but was found to be sensitive to the wall boundary conditions. Nevertheless, it is felt by the writer that such a complex analysis cannot be truly justified with the present knowledge of the effects of compaction being so limited. In addition, the need for a suitably programmed computer facility tends to restrict its use in practice. Ingold (1980) has also expressed the need for there to be a greater understanding of the mechanics of soil compaction, before a definitive analytical method for computing residual earth pressures can be developed with any confidence.

It is therefore felt that the methods proposed by Broms (1971) and Ingold (1979a) provide a simple improvement on the classical approaches and as yet, they represent the most suitable method of accounting for the effects of compaction. However, it is also necessary to consider whether a structure will experience subsequent lateral displacements that may further affect the pressure distribution. The effects of wall movement are discussed in the following section.

2.1.5 Effects Of Wall Deformations

In the past, it has been conventional to design earth retaining structures for either the at-rest or active conditions. The development of active pressures has been traditionally assumed to be due to the displacement of the wall away from the soil. The full active conditions are reached when the full shear strength of the soil is mobilised. However, the wall movement that occurs during the process of compacting the backfill does not cause the active pressures to be created. In these circumstances, the soil is tending progressively to push the wall outwards due to the compaction of each individual layer. There are no relative displacements within the soil and consequently, the soil shear strength is not mobilised. For active pressures to be created, the wall has to be moved by an external load after the soil has been placed so as to relieve the residual pressures caused by compaction. Alternatively, if the wall
is propped whilst the backfill is compacted and is subsequently released after compaction, then an equilibrium position will be reached, such that the residual earth pressures will diminish and tend towards the active condition. Partial active conditions may develop if the compactive effort applied to the upper layers is adequate to cause the entire wall to deform, but is inadequate to re-compact the lower layers of soil. This means that the upper layers of soil cause the wall to move away from the soil at a lower level and reduce the residual stresses at the lower level. The soil shear strength will be fully mobilised at the lower level if the wall movement at this level is sufficient to cause active conditions within the adjacent soil mass. When this occurs, it tends to cause the pressure distribution to be parabolic in shape, similar to that found by Sowers et al (1957), Sims et al (1970), Broms and Ingelson (1971), Jones and Sims (1975) and Coyle and Bartoskewitz (1977).

The movement of a structure after the backfilling has been completed is especially common for the case of bridge abutments. Typically, lateral movements of an abutment can be caused by the expansion and contraction of the bridge deck or by the braking forces exerted by vehicles crossing the bridge. Consequently, the residual pressures, created as a result of compaction of the backfill, are often considerably affected by the subsequent movements of the abutment. It is generally accepted that the displacements required to create the full active and passive conditions are affected by the way in which the structure moves. The most common forms of deformation are translation or rotation about the base or the top. The minimum deformation requirements to produce the limiting states of equilibrium have been determined by numerous authors. Table 3.3 shows a selection of displacement values, which have been determined for a rigid structure embedded in a dense cohesionless soil. The displacements are given as a proportion of the height of the retained soil.
Table 2.3 Wall displacements necessary to cause active and passive conditions

Terzaghi (1936) showed that if the active wall translation in dense sand was less than the critical value, then the soil would tend to arch onto the upper part of the wall, thus causing a non-hydrostatic pressure distribution. A hydrostatic pressure distribution would only be realised after the wall had moved sufficiently to cause the soil shear strength to be mobilised at every depth behind the wall. Ingold (1982) has suggested that the high residual pressures from compaction may cause the rigid retaining wall to slide and so reduce the pressures to the active value. However, it was suggested that the bending moment at the base would still be higher than that calculated from the conventional design approach based on quasi-hydrostatic pressure distribution. An explanation for this discrepancy was derived from the work of Vargin (1968) and Dubrova (1963) which has indicated that a wall translation of $H/500$ can lead to a parabolic pressure distribution rather than a hydrostatic distribution. The resulting lateral thrust was found to be equal to the active value but acted at a height greater than $0.5H$ above the base, rather than at a height of $0.33H$ as is obtained from the traditional
assumption of a hydrostatic pressure distribution. In contrast, the results of Matteotti (1970) for a model quay wall indicated that the centre of pressure tended to drop as the wall rotated about the base. An average value of 0.42H up from the base for the position of the line of thrust has been determined from model tests by Sherif et al (1982). However, this was based on the assumption that the active condition is created at the state when the shear strength between the wall and the soil is fully mobilised. The derived relationship between wall displacement and wall height for various angles of internal friction for the soil are shown in Figure 2.2. Since the soil shear strength is proportional to the soil density, it can be inferred from the graph that denser soils require less wall movement to reach the active state. The resulting wall displacements required to cause the active condition had an average value of H/600, which is somewhat less than those found previously.

Model tests to investigate the passive pressure distributions in sand were investigated by Narain et al (1969). It was found that a translation of a wall would produce a triangular pressure distribution, whereas a wall rotation would cause the distribution to be parabolic. This may be useful for predicting the type of pressure distribution caused on the rear of a capping beam, as a result of being thrust backwards by an expanding deck slab.

In conclusion, it would appear that the wall deformations subsequent to completion of the compaction of the backfill can have a significant effect on the earth pressures. However, the variation in pressure would appear to be dependent on the mode and magnitude of the wall deformation, for both active and passive directions of movement. Furthermore, it would seem likely that the variation in lateral earth pressures after compaction should be affected by the degree of friction between the wall and the soil and this is discussed in more detail in the following section.

2.1.6 Effects Of Wall Friction

The effects of wall friction are particularly relevant to the case of spillthrough abutments for three main reasons. Firstly, as for
retaining walls, wall friction will cause principal stress rotation and affect the failure surface within the adjacent soil, which in turn affects the magnitude of lateral earth pressures. Secondly, the relative longitudinal movement between the soil and the abutment columns will inevitably cause a frictional force to be exerted on the column sides. Thirdly, wall friction affects the magnitude of the residual pressures that remain after compaction of the backfill.

The influence of wall friction on the development of residual pressures after compaction has been observed by Sowers et al (1957), Sims et al (1970), Bros (1972) and Ingold (1979a). The wall friction has been found to lock in a proportion of the vertical stresses induced during the process of compaction by preventing full recovery of the soil adjacent to the wall. The lateral pressure on the wall is therefore equivalent to the normal stress acting across the soil/wall interface. The existence of wall friction was observed by Sims et al (1970) on a retaining wall at Grange Mill. Strain gauges within the concrete indicated a tensile stress at the top of the front face of the wall. However, this observation tends to suggest that the friction on the rear face of the wall must be acting downwards, thus indicating that the soil is tending to hang on the wall rather than be restrained from vertical recovery. It was suggested that the soil was tending to arch between the wall and the rear boundary of the backfill wedge.

The magnitude of the wall friction, $S$, is highly dependent on the shear strength characteristics of the soil and the surface texture of the wall. The Civil Engineering Code of Practice CP2 (1951) recommends a value of $S = 20^\circ$ for the interface between a concrete wall and a cohesionless backfill material, under active conditions. It is suggested that the value is halved when considering passive conditions. The friction is assumed to increase if the material is coated in tar and a value of $S = 30^\circ$ is given for steel coated with tar under active conditions. Model tests by Pearson-Kirk (1967) confirmed the values of wall friction under active pressure as suggested by CP2. However, it was suggested that the wall friction for passive conditions was significantly underestimated by using $S_{\text{active}}/2$. It was also found that $\tan S$ and $\tan \theta$ ($\theta =$ internal angle
of friction of soil) both decreased with increased normal pressure and it was explained as being due to the crushing of angular sand particles to create a smoother surface. The values of $\delta$ suggested by CP2 are very general and are not related to the properties of the soil. It would therefore seem more appropriate to relate the values of wall friction to the shear strength parameter $\phi$ of the soil. The effect of wall friction was included into Coulomb's theoretical analysis by Packshaw (1946). The value of $\delta$ was taken as being in the range of $3/4 \phi$ to $\phi$ for a dry cohesionless soil against concrete. If the wall was coated in bitumen it was suggested that a value of $\delta$ equal to $\phi$ should be used. A comprehensive study of the values of friction between various soils and construction materials was performed by Potyondy (1961). Shear box tests between smooth concrete and sand indicated that the coefficient of friction, $\tan \delta$, was approximately equal to $0.84 \tan \phi$.

It was suggested by Matteotti (1970) that the value of wall friction should be taken as approximately equal to the internal angle of friction of the soil, $\phi$, because he found that the full shear stress is often already mobilised during the backfilling operation.

The vertical shear stresses on the rear face of a retaining wall have also been observed to be influenced by arching actions created within the adjacent backfill. This has been found to occur particularly when a wedge of backfill is compacted between the wall and the original ground. This is discussed briefly in the following section.

2.1.7 Effects Of The Shape Of The Backfill Wedge Behind A Structure

If a wedge of backfill is placed between a structure and the existing ground, then it is possible that additional lateral pressures may be created on the rear face of the wall, as a result of a tendency of the soil to arch across onto the relatively rigid boundaries at either side. This is particularly relevant to the case of bridge abutments because the approach embankment is often constructed prior to the completion of the abutment. Consequently, there often remains a wedge shaped region of embankment that has to be infilled between the two at a later date.
The shape of the backfill wedge was shown by Jansson et al (1948) to influence the magnitude of the pressures induced on a retaining wall. It was found that larger pressures were exerted on the wall when the soil was compacted in a wedge shape between the wall and the existing backfill. This was confirmed by an analytical investigation by Aggour and Brown (1974), from which it was deduced that higher residual pressures could be expected for a narrower wedge of soil. However, work on retaining walls and bridge abutments (Sims et al (1970) and Jones and Sims (1975)) has indicated that the shape of the wedge has only a small effect on the residual lateral pressures compared with the effects of poor compaction in difficult areas.

Having discussed the factors which can influence the earth pressures on a structure during the period of construction, the following two sections discuss the seasonal and traffic loading effects that occur in the long term.

2.1.8 Seasonal Effects

It has been customary to base the design of an earth retaining structure on a single prediction of the likely earth pressures to be exerted by the soil. This has traditionally been an assumption of an active or at-rest pressure or more recently that of a residual compaction pressure. However, in reality the majority of structures experience seasonal fluctuations which sometimes tend towards a state of equilibrium with time. One cause of increased pressure during the winter months has been observed by Rehnman and Broms (1972), when they recorded an increase in lateral pressures due to the accumulation of water in the backfill behind a basement wall. The pressures were increased further still when the temperature dropped and the water froze, thus causing a lateral expansion of the backfill. Similar increased pressures due to freezing were recorded at the base of a rigid frame bridge abutment by Broms and Ingelson (1972). However, the pressures at the top of the instrumented abutment were observed to reduce to zero. This was assumed to be
due to the contraction of the bridge deck, as a result of the low winter temperatures which caused the abutment to be pulled away from the mass of soil. During the summer, the deck expanded and the earth pressures increased towards the passive values. Similar seasonal fluctuations for another rigid frame bridge was reported by Broms and Ingelson (1971). Seasonal fluctuations of pressure on an unpropped retaining wall were reported to be due to temperature fluctuations of the backfill material by Coyle and Bartoskewitz (1977).

2.1.9 Effects Of Traffic

The presence of traffic near to the rear of a structure has been observed to have a considerable effect on the lateral earth pressures. A continual flow of traffic tends to cause a dynamic surcharge loading to the surface of the backfill. A surcharge load has been shown to increase the stresses in the soil below by Spangler (1938) and Spangler and Mickle (1956). If the surcharge loading is applied repetitively, then the effect will be essentially similar to that of a vibratory roller and compaction of the soil beneath will result. An increase in earth pressures on a motorway retaining wall due to traffic loading has been measured by Sims and Jones (1974). The pressure distribution was found to be parabolic and the pressures at mid-height were observed to increase from 5psi (34kPa) to 19psi (130kPa) over the initial period of four years after the completion of the backfill. In this case, the traffic was passing at a distance of over 14ft (4.3m) from the rear face of the wall. An increase in pressure due to traffic loading on a 12m high bridge abutment in cohesionless soil was also recorded by Muller (1973).

For the case of bridge abutments, it is not unusual for the effects of deck expansion and contraction and of compaction due to traffic loading to be going on simultaneously. When an abutment is pulled away from a mass of soil during the winter, the traffic will tend to compact the soil and fill the region previously occupied by the wall. Thereafter, when the deck expands during the summer, the pressures will tend to increase beyond the maximum values recorded for the
previous year because the lateral movement of the abutment will now cause large deformations of the compacted soil to occur immediately. This process is a likely explanation of the increase in pressure towards the passive value, as recorded by Broms and Ingelson (1972).

Up to now the literature that has been discussed has been primarily concerned with the factors affecting retaining walls, in connection with the design of bridge abutments. The following section considers the influence of the limited width of a bridge abutment on the prediction of earth pressures across it.

2.1.10 Three-Dimensional Aspects Of Structures Of Limited Length

Nearly every method of analysis for an earth retaining structure has been based on a two-dimensional problem by assuming that the structure is infinitely long. Although this may be reasonable for long retaining walls, it is unlikely to be truly representative of a bridge abutment, which has only a limited length. For the case of spillthrough abutments, this is particularly relevant because of the additional localised effects of the columns.

The three-dimensional nature of a motorway bridge abutment was illustrated by Jones and Sims (1975). It was observed that the confining nature of cantilever wing walls caused high residual pressures to act on the rear face of the abutment and inner faces of the wing walls. In addition, it was noticeable that the pressures on the rear face of the abutment below the level of the wing walls were somewhat reduced. This was due to the ability of the soil to flow outwards beneath the wing walls. Furthermore, the difficulties of compacting the soil beneath the soffit of a wing wall were found to limit the residual pressures due to compaction in this region. The consequence of the high lateral pressures on the wing walls was to cause them to rotate about a vertical axis parallel to the back of the abutment rather than a horizontal axis parallel to the ground as is usually assumed for earth retaining structures. As a result, the wall of the abutment was subjected to bending stresses both horizontally and vertically. The three-dimensional passive earth
pressure problem in front of suspension bridge abutments was investigated by Horn (1972). He acknowledged the formation of a failure 'shell' in front of an abutment rather than simply a two-dimensional failure surface. The formation of a similar shell failure surface has also been observed in front of anchor plates by Buchholz (1930) and Dickin and Leung (1983). Horn performed tests for a translation of an abutment with a width, B, in the range of 0.3H<B<3.3H, where H is the height of the abutment. For widths less than 0.3H, it was deduced experimentally and theoretically by Weissenbach (1961) that the structure would cut into the soil and displace it laterally, thus not creating the assumed failure shell. In addition, Horn found that for widths greater than 3.3H, the three-dimensional nature of the failure shells would not affect the resistance by more than 10% as calculated by a two-dimensional approach. A simple equation was presented which related the three-dimensional resistance to the two-dimensional passive earth pressure by means of Shape and Model factors.

\[ E_p = E_{pe} A \left(1 + \frac{C H}{B}\right) \]

where

- \( E_p \) = three-dimensional passive earth pressure
- \( E_{pe} \) = two-dimensional passive earth pressure
- \( A \) = Model factor
- \( C \) = Shape factor
- \( H \) = Height of wall
- \( B \) = Width of wall

The factors A and C were evaluated by a statistical analysis carried out on the results of numerous tests by the author and others. A further statistical analysis resulted in an equation relating the wall displacement at a state of passive failure, \( \Delta L_f \), to the wall height, \( H \), and relative density of the soil, \( D_r \).

\[ \Delta L_f = 10.4 H^{1.2}(1 - 0.625 D_r) \]

However, this equation appears not to take into account the effect of the width of the column and it is felt that this must be significant because a larger width would involve the failure of a
larger zone of soil. It would therefore seem likely that correspondingly larger displacements would be required to fully mobilise the shear stresses at the outer extremes of the failure region.

In conclusion, there seems to be no doubt that the three-dimensional failure region within the soil adjacent to a laterally moving structure can have a significant effect on the magnitude of the earth pressure. However, such effects are still not generally considered for the design of bridge abutments. The following section reviews the existing methods of predicting the earth pressures against spillthrough abutments, which only vaguely, if at all, account for the three-dimensional soil resistance against the columns.

2.1.11 Existing Methods Of Predicting The Earth Pressures Against Spillthrough Abutments

Although spillthrough abutments are now widely used, the design criteria remain very vague. Over the years, designers have used simplistic approaches when calculating earth pressures on bridge abutments. It has become common practice when designing simple structures to assume an equivalent hydrostatic pressure on drained freestanding abutments of $5H \, \text{kN/m}^2$, where $H$ is the depth below the ground surface. The underlying assumption of this method is that the abutment is free to move by tilting or sliding, enabling the supported soil to develop a fully active state. However, for more complex structures, such as spillthrough abutments, this simplistic approach cannot be justifiably applied. Firstly, the resistance of the sloping side of the embankment must be considered as this may prevent full active conditions being achieved at the rear. Secondly, the spillthrough abutment is essentially a three-dimensional problem and as yet the effects of soil arching and side friction on the columns have not been adequately investigated.

In a survey carried out by Hambly (1979) for the Building Research Establishment, it was reported that out of 20 statements of practice there were 12 different methods of calculating the earth pressure.
The four most common design approaches that were noted were as follows, in order of magnitude of assumed earth pressures:

(i) The approach in which lateral earth pressures are not considered for embankments with a slope of 1 in 2 or shallower. This is based on the assumptions that the abutment does not affect the embankment stability nor does it influence the movement of the embankment.

(ii) The Chettoe and Adams (1938) approach which assumes a net active pressure distribution over the entire rear face of the abutment. The net active pressures on the rear of the columns are based on the assumption that the forwards flow of soil between the columns would result in an increase in pressure on the rear faces and a decrease on the front faces. It is suggested that an arbitrary allowance of up to 100% should be applied to the active pressures on the columns to take into account the possible effects of side friction, soil arching, settlement and an outwards flow of soil. No suggestions are made as to how the arching of the soil varies with the spacing to column width ratio.

(iii) The Huntington (1937) approach which recommends that full active pressure should be assumed to act over the gross rear width of the abutment, if the space between columns is less than twice the column width. In effect, this approach means that the soil is then assumed to arch perfectly between the columns at narrow column spacings. However, no information is given as to how the distribution of the earth pressures should change for wider column spacings. In addition, the fill in front of the columns was assumed to offer up to active resistance only on the front face of the column, with a reduction to take into account the descending slope, although the size of this reduction was not quantified. It was suggested that the active pressures on the rear of the abutment should be increased by 25%, if the crest of the abutment was unyielding.
(iv) Full active pressure over the gross width of the rear of the abutment with no allowance for the soil resistance on the front. This conservative approach considers the abutment to act similar to a retaining wall and gives no consideration to the action of the columns.

The Chettoe and Adams approach was based on the assumption that it is difficult to achieve good compaction in front of the columns and therefore the backfill cannot offer any resistance to the forward movement of the columns. However, they suggested that the weight of the earth over the whole breadth of the footing may improve the stability and that the horizontal friction force from the bearings in either direction should be considered. They also appreciated the likelihood of downdrag on the columns due to settlement and the possibility of soil flow in an outwards direction.

The Hampshire Sub-Unit of the South Eastern Road Construction Unit (1972) compared the column section design bending moments derived from three different loading assumptions, which are as follows:-

(i) At-rest pressures acting on the upper region of the rear face, down to a depth where a 4m width of soil exists across the sloping embankment at the front. Below this depth, the abutment is assumed to be fixed such that the net pressures on the columns are equal to zero and above this level the active pressures are doubled on the rear face of the columns.

(ii) The Chettoe and Adams approach with at-rest pressures acting on the rear face of the capping beam and double at-rest pressures on the rear face of the columns.

(iii) The Chettoe and Adams approach with active pressures on the rear face of the capping beam and double active pressures on the rear face of the columns.
The design bending moment for the columns obtained from the 4m assumption was found to be only 46% and 16% of those obtained from the Chettoe and Adams active and at-rest approaches respectively. This was primarily due to the fact that in the 4m assumption the columns were designed for the bending moment at the level where a 4m width of embankment exists at the front of the abutment, whereas the Chettoe and Adams approaches designed for the bending moments at the column roots.

A further consequence of the 4m assumption is that the fixity of the abutment below a certain level leads to the assumption that the base slab is not subjected to bending, whereas in the Chettoe and Adams approaches bending of the base is assumed. A comparison of the costs of construction of the abutments based on each of these design approaches, revealed that the 4m assumption gave the cheapest solution with the Chettoe and Adams active and at-rest approaches being 19% and 33% more expensive respectively. It appears that the differences in cost were mainly due to the need to provide reinforcing steel in the base slab to resist bending for the designs based on the Chettoe and Adams approaches.

The Hampshire Sub-Unit report also contained some design calculations for estimating the soil pressures on the front and rear faces of an abutment column, based on the deflections that would be caused by an assumed simple distribution of pressures on the rear of the abutment. The two initial loadings that were considered for comparison were those predicted by the Chettoe and Adams approach with active and at-rest pressures. The trial method involved the calculation of the deflections of the columns at discrete levels due to flexure and base rotation, as caused by the applied loadings. A simple passive failure wedge was assumed at the front of the columns but with modifications to cater for the sloping embankment and an active failure wedge at the rear. The zone of soil straining was considered to be solely limited to within these wedges. From this, the horizontal soil strains were then calculated for each of the discrete levels, both at the front and at the rear. The strains were used to determine a coefficient of pressure at each of the discrete levels from the relationship
proposed by Lambe and Whiteman (1979), shown in Figure 2.1. The horizontal pressure profiles at the front and back were then determined by applying the coefficient of pressure to the vertical overburden pressure. This resulted in a computational prediction of a modified distribution of applied pressure on the rear of a column and the resistant soil pressure exerted on the front of a column, as calculated from the initial estimate of the abutment displacements. The resulting pressure distributions were found to vary according to the assumed degree of base rotation.

This alternative approach to the estimation of the lateral pressures on the columns of a spillthrough abutment at first appears to be a fairly practicable method of analysis. However, in order for it to be realistic to the majority of real life situations, it would require considerable modifications to enable it to cater for the effects of incremental construction and compaction pressures. It would also need to cater for the three-dimensional nature of the failure wedges.

Lee (1982) and subsequently Ah-Teck (1983) have reported on a series of model tests, in the Cambridge geotechnical centrifuge, designed to investigate the behaviour of piers in a spillthrough abutment which are subjected to embankment loading. A plane strain model was built in the strong box to represent a row of piers embedded in an embankment with a sloping front face. A limit state condition was induced by sliding a base plate, which supports the sloping front face of the embankment, away from the piers, whilst the model was subjected to the required centrifugal acceleration, as shown in Figure 2.6. The failure mechanism created was intended to represent the condition of toe washout or differential settlement of the foundation, such that a failure wedge was developed within the soil at the front of the piers. However, it is felt that such a failure mechanism is not particularly representative of the actions commonly involved in the majority of ordinary spillthrough abutments. Instead, it is likely that these failure conditions may be more relevant to the case of spillthrough abutments which are supported on piles through soft underlying soil.
In order to understand better the behaviour of the columns of a spillthrough abutment with regard to their interaction with the surrounding soil, it may well be useful to make a comparison with laterally loaded piles, and this is discussed in the following section.

2.1.12 Design Of Laterally Loaded Piles

The problems involved with analysing the response of laterally loaded piles are very complex due to the many inter-related factors that exist. One of the most dominant single factors is probably that of the pile stiffness. This governs whether a failure mechanism is one of rotation of a short rigid pile or of flexure leading to failure in bending of a long flexible pile. Fortunately, most of the existing theories can be applied to both short and long piles simply by introducing different boundary conditions into the analysis. At present, it is not immediately apparent which of these two mechanisms is most representative of a column in a spillthrough abutment. The latter would be more likely to be the case if the base slab offered sufficient resistance to effectively cause the column to be fixed at its lower end and thus represent a deeply embedded pile. Alternatively, significant base rotation would tend to cause the column to act more like a short rigid element.

For the purpose of this investigation, namely a spillthrough abutment in cohesionless soil, only laterally loaded pile theories applicable to cohesionless soils are reviewed. The theories are discussed according to whether they are based on ultimate or working load conditions.

(a) Ultimate Loads

The design of bridge abutments is generally not concerned with ultimate loading conditions. Instead, it is considered more applicable to evaluate the deflections, bending moments and shear forces at working loads. Even so, consideration of the pressure distributions at ultimate conditions as proposed by Brinch Hansen (1961), Broms (1964) and Petrasovits and Awad (1972) may provide a useful insight into the form of the distributions to be expected at lesser loading
conditions. In each case, the ultimate lateral resistance is calculated from the equilibrium requirements at failure. Failure is assumed to take place either by failure of the soil along the entire length of the pile or by failure of the pile itself when it reaches its ultimate bending capacity. The soil resistance, \( P_u \), at any depth, \( z \), against a short rigid pile was given by Brinch Hansen (1961) as

\[
P_u = qK_q + cK_c
\]

where \( q \) = effective overburden pressure  
\( c \) = cohesion (\( c=0 \) for sand)  
\( K_q, K_c \) = factors dependent on \( \phi_{\text{soil}} \) and \( z/d \)

This pressure is assumed to act on the front face of the pile but is reversed below the centre of rotation. A simplified pressure distribution was later used by Broms (1964) which ignored the active pressure on the back face and assumed the pressure acting on the front to be three times the Rankine passive pressure

\[
P_u = 3qK_p
\]

where \( K_p = (1+\sin\phi)/(1-\sin\phi) \)

For short rigid piles, the high negative earth pressure below the centre of rotation was replaced by a concentrated load. A pressure of 3.7 times the Rankine passive earth pressure on the front with active pressure acting on the rear face was assumed by Petrasovits and Awad (1972). The resulting lateral soil resistance at any depth was given by

\[
P_u = q(3.7K_p-K_a)
\]

where \( K_p, K_a \) = Rankine's coefficients of earth pressure
Again, the pressure was reversed below the centre of rotation for a short rigid pile.

A comparison of the theory with a series of pile tests in cohesionless soil was carried out by Petrasovits and Awad (1972). Their results indicated that the predicted values of ultimate lateral resistance were as much as 26.7% less than the measured values. A similar degree of inaccuracy was recorded from a comparison with data published by several other authors by Broms (1964).

If, as in the case of spillthrough abutments, there is a large base slab connected to the lower end of the columns, then the ultimate lateral resistance of the structure may be significantly affected. A form of ultimate analysis which considers the contribution of friction against the base of free and tied pier foundations has been presented by Roscoe (1957).

(b) Working Loads

There are three main approaches upon which the majority of theoretical methods are based for determining the lateral movements of piles at working loads.

Firstly, there is the method of subgrade reaction which considers the soil as acting as a series of independent elastic springs, based on the proposal by Winkler (1867). The analysis requires the solution of a series of finite difference equations in addition to the equations derived from equilibrium and the boundary conditions. The increase of elastic modulus with depth for a cohesionless soil has been represented by varying the spring stiffness (or modulus of subgrade reaction \( k_h \)) along the length of the pile, (Palmer and Thompson (1948), Reese and Matlock (1956), Matlock and Reese (1960) and Broms (1964)). The method of subgrade reaction was extended for two layer soils with a constant \( k_h \) in each layer, (Davisson and Gill (1963), Reddy and Valsangkar (1968)) and subsequently for an increasing \( k_h \) per layer (Meyerhof et al (1981)). The above methods all assume that the soil acts as a linear elastic material and so more accurate methods have been developed by replacing the linear...
springs with non-linear p-y curves, (McClelland and Focht (1958), Kubo (1965), Matlock (1970), Madhav et al (1971), Reese et al (1974)). Furthermore, an extended analysis to include the effects of axial loading was reported by Reese (1977).

Some of the above methods of analysis have been compared with results of full size or model tests on laterally loaded piles, (Broms (1964), Reese et al (1974) and Meyerhof et al (1981)). Broms (1964) compared his method of analysis with published test results of several other authors for piles in cohesionless soils and found that the maximum bending moments were closely predicted for single piles but the calculated lateral deflections consistently exceeded the measured lateral deflections. Reese et al (1974) used their method to predict the behaviour of piles at Mustang Island. A comparison of predicted and measured ultimate loads, bending moments and deflections was only obtained after introducing empirical coefficients to accurately define the p-y characteristics of the soil. The agreement with the test results of other authors was not reported to be as good. Consequently, it is evident that the method is highly dependent on the ability to deduce the empirical coefficients for each type of soil encountered. Meyerhof et al (1981) used their method to predict the ultimate lateral loads and deflections of model and full size piles. The accuracy was described as reasonable, although the ultimate lateral loads were found to be unsafely over estimated by as much as 30%.

The accuracy of all of these methods is limited by the inability of the Winkler model to represent the continuous nature of the soil. Furthermore, it is very difficult to accurately estimate the load-displacement relationships along the length of a pile for the various conditions relating to the size and type of pile and the properties of the soil. If this method of analysis was to be applied to the case of spillthrough abutments, there would be additional errors due to the problems of accurately representing the effects of soil friction on the column sides and the effects of a large base slab.

The second common approach for analysing laterally loaded piles has been to model the soil as an elastic continuum which, unlike the
Winkler approach, caters for the continuous nature of the soil. This method idealises the pile as an infinitely thin strip of the same width and stiffness, which is discretised into a number of elements. Each element is assumed to experience a uniform horizontal stress from the soil mass which is constant over the width of the strip. The value of soil modulus and Poisson's ratio for the soil are assumed to be unaffected by the presence of the pile. The analysis involves the solution of a set of equations which equate the soil and pile displacements at the centre of each element, together with a number of equilibrium conditions. The soil displacements are evaluated from the Mindlin (1936) equation for horizontal displacements within a semi-infinite half space. The pile displacements are obtained from the equation of flexure of a thin strip expressed in finite difference form. The solution is generally performed on a computer using the Boundary Element Method. The analysis has been carried out for a homogeneous elastic soil by Spillers and Stoll (1964) and Poulos (1971a,b), and both have modified their analysis to cater for an elasto-plastic soil. Further work by Poulos (1973) and Bannerjee and Davies (1978) has been performed to develop the method of analysis to cater for non-homogeneous soils. The work of Poulos (1973) was concerned with the case of a pile embedded within soil undergoing lateral movement. This may be particularly relevant to spillthrough abutments because a certain degree of soil movement is often expected due to the effects of compaction behind the capping beam. A correlation of the theoretical and measured bending distributions was performed by Poulos (1971a) and was generally found to be good. The prediction of the load-displacement characteristics was found to become less accurate towards the ultimate values, with the error becoming greater for the stiffer piles. A comparison of experimental and theoretical bending moments by Bannerjee and Davies (1978) has shown that good correlations can be achieved by adopting the non-homogeneous soil model. However, the accuracy of the elastic continuum approach is often limited by errors that result from predicting the soil deformation characteristics without firstly performing full size pile loading tests. In addition, the analysis of an elastic continuum requires a complicated solution procedure which
in turn requires the facilities of a computer. Also, the accuracy of
the method has been shown, by Evangelista and Viggiani (1976), to
be dependent on the manner in which the pile is discretised.
Furthermore, the idealisation of the pile as a thin strip, may present
difficulties when trying to incorporate the effects of a base slab.

The third type of approach for the analysis of laterally loaded piles
has recently been developed for computer use and enables piles with
complex geometries to be analysed. The Finite Element Method has
been used by several authors to investigate the two-dimensional
effects of soil/structure interaction. One such analysis was
performed by Yeigan and Wright (1973) for a cohesive soil. A
plane-stress analysis was carried out for a horizontal section of the
column at depth, in which the soil/pile interface was represented by
a non-linear 'slip' element. A three-dimensional study for a linear
isotropic material was performed by Baguelin and Frank (1980). This
analysis assumed perfect connectivity between the soil and the pile
and thus does not adequately represent the likely behaviour of a
spillthrough abutment. A simplified finite element analysis, based on
the Winkler approach was reported by Sogge (1981). The pile and
soil were represented by beam and bar elements respectively. The
analysis was performed for a linear subgrade reaction with several
variations with depth. A finite element analysis on laterally loaded
long flexible cylinders has recently been carried out by Randolph
(1981) for a soil with either constant or linearly increasing modulus
with depth.

The Finite Element Method would appear to be suitable for carrying
out parameter studies for structures with complex geometries.
However, an analysis of a non-linear, non-homogeneous soil, allowing
for relative movements between the soil and pile in three-
dimensions, results in an extremely complex problem which would
require extensive computing facilities to produce a satisfactory
solution. Unfortunately, until all these problems can be solved
simultaneously, the analysis of laterally loaded piles in cohesionless
soils will remain less than 100% accurate.
The methods of analysis that have been mentioned have provided a worthwhile background to the problem of spillthrough abutments, although no single approach can be considered to be totally applicable. The accuracy of all the above methods is limited by the inability to accurately include the effects of side friction between a column and the soil.

One theory which does permit the consideration of the effects of side friction on a column in a cohesionless soil was derived by Rowe (1956). The theory was based on a similar approach to that of Terzaghi (1943, p233) for the design of anchor plates. It was assumed that lateral column movement was limited by the soil resistance at the front and also by the shear resistance of the soil over two vertical planes of a failure wedge parallel to the column sides. The soil stiffness modulus for a finite column face width was related, by its dimensions, to the soil stiffness modulus of a long retaining wall. This relationship enabled the side length of the column to be included into the analysis, although no experiments were conducted to prove its validity.

All of the above methods have been limited by their dependency on the load-displacement characteristics of the soil. Consequently, a considerable amount of research has been performed in an attempt to improve the estimation of such characteristics. Initial values of the coefficients of subgrade reaction for various soils were proposed by Terzaghi (1955). These were based on experimental results obtained from plate loading tests and full size lateral load tests on instrumented piles. Similarly, full size lateral loading tests have been performed for piles in clay by McCammon and Ascherman (1953), Matlock and Ripperger (1956) and Reese and Welch (1975) and for piles in sand by Reese et al (1974). Other methods have included the pressuremeter (Menard (1962), Baguelin et al (1978)) and consolidated undrained triaxial tests (McClelland and Focht (1958)). The values of soil modulus have been estimated from the analyses of experimental data by Poulos (1971a) and Bannerjee and Davies (1978).
Having reviewed the literature on laterally loaded piles in general, the following section reviews the experimental work that has been reported on the use of piles for supporting bridge abutments.

2.1.13 Abutments On Piles

It has been found that a small amount of research has been carried out in America and Canada, over the past 20 years, to investigate the behaviour of spillthrough abutments. However, it is important to note that the American definition relates to an abutment supported over a layer of soft soil by deep piles and as such represents a different type of abutment to that considered within this thesis. Nevertheless, several interesting characteristics of piled abutments have been observed which provide a broader understanding of the behaviour of abutments in general.

In 1968, Stermac et al reported some unusual movements of seven full size underpass structures which were supported on end-bearing piles. It was found that the abutments unexpectedly tended to rotate backwards rather than forwards. This was explained as being due to the consolidation of the underlying soft layer of soil, as a result of the increased vertical stresses caused by the construction of an embankment above. During the process of consolidation, the ground surface beneath the embankment tended to form a dish-like shape, with the largest settlement occurring beneath the highest point of the embankment. This caused the embankment to move towards the position of deepest ground depression, carrying the abutment with it. However, the subsequent discussion papers (Squier and Fujitani (1968), Peckover (1968) and Bjerrum (1969)) have all expressed that the observed movements were more likely to have been due to the negative friction on the piles caused by the consolidation of the soft soil. It was felt that the downdrag was greater for the piles at the rear of the abutment due to the relatively high level of embankment, thus causing the heel of the abutment to settle more than the toe. Tschebotarioff (1970) and Garlanger (1974) confirmed the significance of the downdrag on the piles. In addition, Tschebotarioff also concluded that other forces must have been involved to cause the backwards rotation. Firstly,
it was suggested that a horizontal component of load would be applied to the abutment from the battered piles at the toe. Secondly, it was suggested that the piles were subjected to bending stresses due to the lateral stresses exerted by the consolidating soft soil and this was later confirmed by Nicu et al (1971) and Marche and Lacroix (1972). Thirdly, the weight of the cantilever wing walls was considered.

The consequence of the backward rotation of piled abutments has been to reduce the pressures on the rear of the abutment. Although this type of action has been found to be primarily related to the interaction of the piles and the soft soil, Terzaghi and Peck (1948, p 326) have suggested that similar rotations could occur for abutments on spread footings underlain by soft clay. It was explained that such an action could occur because the heel of the abutment would be situated farther into the settlement dish than the toe. However, for the case of spillthrough abutments with spread footings bearing on cohesionless soil, the effects of differential settlement and consolidation are likely to be less significant.

As has already been mentioned, one possible mode of failure for a laterally loaded pile occurs when it rotates within the soil. Consequently, the following section discusses the position of the centre of rotation for short rigid structures embedded in soil.

2.1.14 Position Of The Centre Of Rotation For Short Rigid Piles

The centre of rotation of short rigid foundation structures within a cohesionless soil has been investigated, in the past, so as to help predict the distribution of earth pressures. A series of full size tests on posts of various cross-section in granular and silty clay soils and model tests on posts in clean sand was performed by Shilts et al (1948). The experimentally determined position of the centre of rotation was found to be reasonably estimated by the point below which there is 33.4% of the area of the vertical cross-section of the embedded portion of the post, as proposed by Seiler (1932). However, it was found that the depth of the centre of rotation would be lowered due to an increased length of embedment, h, or
for very low soil densities. The centre of rotation was found to be at 0.25h above the base but was decreased slightly for foundations with a rougher surface. Tests on a 2h inch long by 6 inch square caisson foundation in dense sand indicated that the centre of rotation was at 0.245h above the base, (Bhagat (1967)). Lateral load tests on piles in silt and on model piles in synthetic emery have been reported by Baguelin et al (1972) and Petrasovits and Awad (1972) respectively. They both concluded that the centre of rotation was about 0.22h-0.23h above the base.

So far, this review has considered the performance of a single structure embedded in soil. The following section describes the literature relating to the behaviour of two or more closely spaced embedded structures.

2.1.15 Interaction Of Closely Spaced Structures Embedded In Soil

The interaction of closely spaced piles can be assumed to have similar characteristics to that of columns in a spillthrough abutment. The performance of a line of piles, loaded in a direction perpendicular to the direction of pile spacing, has been investigated theoretically by several authors, (Yegian and Wright (1973), Ito and Matsui (1975), Poulos (1971b), Randolph (1981)). A two-dimensional plane stress finite element analysis was carried out for columns spaced at two and three times the pile diameter in a non-linear material by Yegian and Wright (1973). It was concluded that only a moderate reduction in maximum load capacity per pile was caused by interaction of the piles. However, the displacement required to fully mobilise the maximum load was found to be larger for closely spaced piles than for a single pile. Two methods for calculating the lateral forces on circular stabilising piles in a plastically deforming soil were developed by Ito and Matsui (1975). One method assumed a plastic deformation satisfying the Mohr-Coulomb yield criterion and the other assumed a state of plastic flow of a visco-plastic solid. Both theories took into account the effects of the pile spacing and were discussed in terms of centre to centre spacing, D1, and face to face spacing, D2, as shown in Figure 2.5.
Both theories indicated an exponential decrease of load acting on the pile with increasing $\frac{D_2}{D_1}$. The theory of plastic deformation implied that the lateral force increased exponentially with increasing values of internal soil friction, $\phi$, assuming a constant value of $\frac{D_2}{D_1}$. In the theory of plastic flow, the lateral force increased as the product of velocity and plastic viscosity of the soil increased but was not greatly affected by the yield stress of the soil. Both theories indicated a linear increase in lateral load as $(D_1-D_2)$ (or pile diameter) increased when $\frac{D_2}{D_1}$ was constant. Comparison of theoretical values with the results of some full size tests indicated an accuracy only to an order of magnitude. The theory of plastic flow was limited by the reliability of the estimation of the yield stress and plastic viscosity of the soil. The theory of plastic deformation was found to be most accurate for small soil movements because of the inherent assumptions that the pile was rigid and that the soil only reached a plastic state around the pile.

Poulos (1971b) developed an alternative approach which permits the analysis of a line of piles, by applying influence factors to the analysis of single piles as determined by Poulos (1971a). A series of graphs were presented which showed the variation of the influence factors with the spacing to diameter ratio. Each influence factor was related to a unique combination of loading (horizontal force or moment) and fixity (free-head or fixed head). In addition, the influence of one pile on an adjacent pile was found to increase for an increase in pile length and also for an increase in pile stiffness. A series of analyses using the Finite Element Method were performed by Randolph (1981). He extended his analysis of a single pile to the case of a row of piles, which led to simple expressions for interaction factors for fixed head and free head piles. The influence was found to be inversely proportional to spacing. Comparison of the theory with model tests by Williams (1979) showed a reasonable accuracy. For the cases of two and three piles in line at a spacing of eight diameters, the calculated efficiencies were 0.92 and 0.86 compared with the measured efficiencies of 0.95 and 0.85 respectively.
A series of centrifuge model tests on circular piers (Lee (1982)) and circular and rectangular piers (Ah-Teck (1983)), have been carried out at different pier spacings. The tests represented the situation of soil flowing between the piers, (as described previously in section 2.1.11). The loading on each pier was expressed in terms of "effective width", (i.e., width necessary to give measured load assuming Rankine active pressure). From the models tests, it was found that the effective width tended to decrease for an increasing height to diameter ratio of the pier. Consequently, it was deduced that the longer piers cause progressive yielding of the soil from the top downwards, resulting in a reduced earth pressure coefficient.

The effects of arching between piers were found to be negligible at a centre to centre spacing of greater than five and eight diameters by Lee and Ah-Teck respectively. A tentative design formula was proposed by Lee which suggested that an effective width equal to half the spacing should be used up to a spacing of five diameters. For any spacing greater than five diameters, an effective width of 2.5 diameters should be adopted. A more sophisticated approach, based on a limit equilibrium study, was presented subsequently by Ah-Teck. The limiting conditions of equilibrium were based on the observations from the centrifuge model tests. As such, an active failure wedge was assumed to occur within the sloping embankment at the front of the piers and the earth pressure coefficient, \( K_s \), was calculated from a Coulomb type of analysis. The earth pressures exerted by the embankment at the rear of the piers were assumed to vary such that the maximum earth pressure coefficient, \( K \), was experienced at the centre line of the pier and decreased to a minimum value, equivalent to the active earth pressure from the sloping front embankment, \( K_s \), at a distance of four pier diameters to either side of a pier. The maximum value of \( K \) was assumed to correspond to at-rest conditions but lower values were assumed for conditions where larger pier deformations were expected. The variation of the earth pressure coefficient, \( K(x) \), between these two extremes was based on the experimental data and was given as
\[ K(x) = K - (K - K_s)(x/N.D)^{2/3} \]

where 
- \( x \) = distance from centre line of pier
- \( D \) = diameter of pier
- \( N \) = upper limit of the distance to diameter ratio beyond which no further increase in pier loading occurs (A value of \( N=4 \) was chosen)

If the spacing of the piers, \( S \), was less than eight pier diameters, (ie \( S<2N \)), then the minimum earth pressure coefficient would not be obtained and would instead be equal to the value of \( K(x) \) at a position equi-distant from both piers. The resulting distributions of the earth pressure coefficients for piers with wide spacing (\( S>2N \)) and close spacing (\( S<2N \)) are shown in Figures 2.7a and 2.7b respectively. The lateral loading on the piers was calculated as the difference between the total force exerted by the rear of the embankment and that of the sloping embankment at the front and was expressed in terms of an "effective width", \( E \). A set of design envelopes were proposed, which relate the effective width to the pile spacing to diameter ratio for varying values of earth pressure coefficient, \( K \), behind the piers, as shown in Figure 2.8.

The major drawback with this approach was acknowledged by Ah-Teck to be the problem of how to estimate the appropriate value of \( K \) (ie earth pressure coefficient at the rear face of a pier) for individual cases and it was therefore suggested that \( K = K_0 \) would produce a safe yet possibly uneconomical design. A comparison of the experimental data with the design envelopes showed that for large pier spacings, a value of \( K = 0.65K_0 \) was obtained for the highest pier with a height to diameter ratio, \( H/D \), equal to 15, whereas a value of \( K \) approximately equal to \( K_0 \) was obtained for the shortest piers (\( H/D = 6.7 \)). This indicated that the longer piers yielded more than the shorter piers. Furthermore, at close spacing of the piers, the derived value of \( K \) tended towards unity, thus indicating that an increased number of piers resulted in smaller deformations of each individual pier.
The tests by Ah-Teck indicated that the effective width for the rectangular piers was less than that for circular piers at the same spacing to width ratio. However, the circular piers were 15mm diameter, whereas the rectangular piers were 13.3mm wide at the front with a side length of 20mm and it is therefore felt by the writer that this causes the comparison of effective widths to be misleading. Furthermore, Ah-Teck acknowledged that there was insufficient data for this observed trend to be conclusive. The effect of cross-section of shape on the stability of posts subjected to lateral loads was investigated by Shilts et al (1948). The results of some model tests on posts of various cross-section indicated that the lateral resistance of a square section post was equivalent to that of a circular section with a diameter equal to the diagonal of the square.

Although only limited research has been carried out on the lateral resistance of lines of piles loaded perpendicular to the line joining the centres, there has however been a number of complimentary studies to investigate the performance of a line of horizontal anchor plates. A set of empirical design charts were derived from a comprehensive series of model tests by Ovesen (1964). Tests were carried out on a line of rectangular ground anchors with varying spacings and depths of embedment, as shown in Figure 2.9. The results indicated that for \( h/H = 1 \), the load resistance of an anchor slab was unaffected for centre to centre spacings of \( L > 51 \).

Unfortunately, these results cannot be applied directly to spaced rectangular columns without further investigation because the tests were limited to a range of anchor slab dimensions where \( h < L < 15.9h \). The dimension requirements for the columns of a typical spillthrough abutment would be in the region of \( L = 0.4h \).

A study of the efficiency of a vertical line of spaced horizontal anchors in sand was investigated by Akinmusuru (1978). The anchor plates were circular and they were pulled horizontally through the soil. It was shown that the anchors act totally independently for centre to centre spacings \( (S_2) > 9 \) greater than nine times the plate diameter, \( r \). A minimum group efficiency was recorded for \( S = 3r \) but at smaller spacings the group efficiency was again found to
increase due to the effects of soil arching between the plates. Such an increase in group efficiency at spacings less than $3r$ has not been recorded by any other authors. Although the arching of the soil may well increase the group efficiency, it is felt that the tests of Akinsumuru have not been carried out at enough different spacings to adequately define this effect. A similar set of tests, this time for anchors pulled vertically, was conducted by Hanna et al (1972). These tests indicated that the anchors acted independently for spacings greater than six times the plate diameter.

In addition, Hanna et al (1972) measured the soil displacements near to a single vertically loaded anchor using an array of mechanical soil strain gauges. It was found that the displacements at positions remote to the anchor were always in the same direction for an increase in loading up to the failure conditions. The soil near to the edge of the anchor plate tended to change direction as the anchor was pulled past the measuring point. For large embedment depths, the displacements were found to radiate outwards from the anchor. In all tests, the magnitude of the soil displacements near to the anchor plate were less than that of the anchor plate itself and were found to decrease rapidly with lateral distance from the anchor plate. A recent finite element analysis of horizontally and vertically loaded anchors has been reported by Rowe and Davis (1982). It was concluded that the effect of dilatancy within a soil can increase the load capacity of an anchor plate. It was shown that for a vertically loaded anchor embedded at a depth of five times its width, the plastic deformation occurs over a region in front of the anchor which is five times the width of the anchor plate when the soil dilates. This compares with 1.5 times the anchor width for a non-dilating soil. The effects of dilatancy were found to have more influence for greater depths of embedment of the anchor plate. This type of failure zone might be relevant to predicting the extent of the failure zone of a column of a spillthrough column embedded in a dense cohesionless soil.

In conclusion, it is evident that centre to centre spacings of up to nine times the structure width can cause interaction effects at ultimate conditions. However, there appears to be a lack of
literature to explain how the effects of interaction vary with increased displacements of the structures. This may be important for the case of spillthrough abutments because the knowledge about column interaction is required primarily for working conditions.

2.1.16 Soil Resistance Contributions Against The Surface Of Embedded Structures

A study has recently been reported by Richter et al (1984) which investigated the effects of pile/soil interface conditions of a laterally loaded pile. The model tests consisted of a cylindrical pexiglass rod embedded in an elastic gelatin mixture. The gelatin was allowed to solidify and the pile was loaded laterally. A photoelastic analysis was performed on sections of the gelatin, after having been cooled to a near freezing temperature. The contribution of side load was eliminated in some of the tests by coating the glass with a layer of silicone grease overlaid by silicone oil. The results obtained indicated that around 40% of the total load was transmitted to the surrounding medium by horizontal shear stresses along the sides of the pile when there was adhesion.

It has been shown that there have been numerous attempts to theoretically represent the situation of a laterally loaded pile in a mass of soil. However, as yet very few authors have considered the contribution of the resistance from the soil on the pile sides in addition to the face load.

A considerable amount of research has been carried out to investigate the axial load capacity of large bored piles in clay, which essentially involves similar soil/structure interaction characteristics to that of a laterally loaded rectangular column within a mass of soil. The design of large bored piles is based on the determination of working loads and settlements due to the resistance mobilised by the shaft and base, which is in many ways analogous to the side load and front load respectively, on the faces of a rectangular column.
Several authors have commented on the likelihood that the shaft resistance and base resistance may not be totally independent.

A bearing capacity theory based on plastic theory was developed by Meyerhof (1951) which indicated that shaft resistance would contribute to a slightly increased base resistance. It was assumed that a failure zone would be created at the bottom of a pile and that it would extend above the base level. Consequently, the earth pressure coefficient acting on the lower region of the shaft would have an effect on the failure zone. A semi-empirical law relating skin friction to strain for piles embedded in sand was developed by Kezdi (1957). It was suggested that point resistance was not proportional to depth but was proportional to the vertical stress acting at the base of the pile. The shaft friction would tend to transfer some of the load from the pile to the adjacent soil and thus increase the vertical stress at base level above that of the soil self-weight alone. Skempton (1959) suggested that, although the bearing capacity factor, $N_c$, for long piles in clay was generally accepted as having a value equal to nine, the effect of shaft resistance may cause this to be reduced. The mobilisation of the shaft resistance would cause a lesser relative movement between the soil and the pile at base level. Consequently, the degree of mobilisation of the soil shear strength at the base would be less than that expected from the relative movements at the surface.

It is well known that the shaft resistance of a pile is fully mobilised for very small settlements compared with the large settlements required to fully mobilise the point resistance. Tests on underreamed bored piles in London clay were carried out by Fleming and Frischmann (1960). Their results indicated that a settlement of 0.2" (5mm) was necessary to cause full mobilisation of the shaft resistance of a 2'6" (792mm) diameter pile. This represents a settlement of 0.7% of the pile diameter. However, they suggested that the settlement required for maximum shaft resistance was likely to be independent of the shaft diameter. In 1961, Haefeli and Bucher carried out tests on 880mm diameter by 24m deep bored "Benoto" piles in a clayey silt soil. The pile was partially extracted and then re-loaded to evaluate the contribution of shaft friction.
alone. They found that the shaft resistance only increased very
slightly after a displacement of 2mm (0.25% of shaft diameter).
The point resistance was found to be half the total load for a
displacement of 1mm but had increased to 86% at 10\(\frac{1}{2}\)mm.

Tests on model piles with enlarged bases in London clay were
performed by Cooke and Whitaker (1961). They recorded that the
shaft resistance was fully mobilised for a settlement of 0.5% (ie
0.1mm) of the shaft diameter (19.1mm). The full bearing capacity
of the base (as represented by a value of \(N_c = 9\)) was attained
after a settlement of 10-15% of the various base diameters which
were between 19.1mm and 76.4mm. However, they estimated that
the ultimate bearing capacity would only be reached for settlements
exceeding 30% of the base diameter. A subsequent study on a
variety of deep bored cylinder foundations in London clay was
reported by Whitaker and Cooke (1966). The full size test results
were concluded to indicate that shaft length had no effect on the
development of shaft resistance. In addition, for a given percentage
of the ultimate load, the settlement was found to be larger for
increasing pile diameters. The settlement recorded to fully mobilise
the shaft resistance was 0.5% to 1.0% of the shaft diameter and for
the base resistance was 10% to 20% of the base diameter.

However, the mobilisation of shaft resistance being dependent on the
shaft diameter seems somewhat peculiar because there is no
immediately apparent reason why the pile diameter should affect the
way in which the shear stress is developed at the soil/structure
interface. Furthermore, it is felt that the elastic compression of a
long pile would cause the shaft resistance to be gradually mobilised
from the top downwards. This means that in order to fully mobilise
the shear resistance, relatively large displacements would be required
at the top of the pile so as to provide adequate displacements at
the base. Tests on large 3' (914mm) diameter bored piles in stiff
clay by Burland et al (1966) again indicated that full shaft
resistance was mobilised for a settlement of 0.7% (ie 6.4mm) of the
shaft diameter.

The individual shaft and base resistance of straight and under-
reamed piles was considered by Burland and Cooke (1974). The
contributions of resistance offered by the shaft and the base for the two types of pile are illustrated in the Figure 2.10 which shows that the relative significance of the base and shaft resistance may alter for varying pile types. However, it should be made clear that the ultimate load for an under-reamed pile is reached at larger settlements than for a straight pile.

In conclusion, it is evident that when a pile is subjected to only small axial settlements (or a column is subjected to small lateral displacements), the load carried by friction against the sides of the structure can contribute significantly to the total soil resistance.

Having reviewed the literature related to the estimation of earth pressures on spillthrough abutments, the remaining sections of this chapter review past work concerned with two of the major forms of experimentation used within the present work. As such, the following section describes the existing methods of determining the density of laboratory sand specimens and the final section discusses the literature relevant to the performance of earth pressures cells.

### 2.2 Density Measurement Techniques For Laboratory Sand Specimens

There are many well established methods for determining the bulk density of cohesionless soils, such as sand replacement tests, core tests and nuclear tests. Such methods are partly dependent on the soils ability to maintain its particle structure during excavation or sampling, as is the case for the majority of moist soils. However, when conducting experiments under laboratory conditions, it is often more convenient to use air-dried material in order to achieve better control of the sand properties, which would otherwise be affected by changes in moisture content. Many foundation structures are embedded in cohesionless soils and consequently, it has been common in the past to perform model tests in a similar soil. An air-dried medium to fine sand is often chosen partly due to its ease of handling. Unfortunately, such a material is usually unable to be self supporting due to the absence of negative pore water pressures within it. Consequently, the sand replacement tests and core tests have proved to be unsuitable. The nuclear technique is limited due to the problems involved
in obtaining accurate measurements in a limited volume of soil bounded by the walls of a container. This is a result of the relatively large operating space that is required and the effect of the adjacent walls on the radiation count. A further restriction is that the regulations applying to the use of radioactive materials would severely restrict simultaneous operations in the remainder of the laboratory.

The inadequacy of the above methods has led many researchers to develop a variety of alternative density measuring techniques, primarily applicable to air-dried sands.

A number of techniques have been developed for measuring the density of sand specimens created by "raining" the sand into a container. One method, using a 3" (76mm) diameter by 3" (76mm) long brass cylinder, with the top edge being formed to a knife-edge, has been reported by Walker and Whitaker (1967). The porosity of the sand was measured by placing a number of the calibrated measuring cylinders at different points within the container and raining the sand in the normal manner. The cylinders were then excavated and the sand contained within was weighed and an average density was determined. A similar technique using a number of moulded plastic samplers, 2.65" (67mm) diameter, 1" high and having a wall thickness of 0.055" (1.44mm) was reported by Kolbuszewski and Jones (1961).

A box density device, developed at the Waterways Experiment Station, has been used to measure the density of dry sand after having been compacted by vibration, (Sloan (1962)). The device consisted of a 4" (100mm) by 12" (305mm) by 3" (76mm) deep metal form, a box shaped scoop and cleanout tools. The metal form had tapered cutting edges along the bottom and had a 0.75" (19mm) thick flange surrounding the upper edge. The box was pressed down into the sand specimen so that the lower surface of the flange rested on the upper surface of the sand. A penetration of 2.25" (57mm) was the maximum that could be made without causing appreciable deformation of the box which would thus change its volume. The sand contained within was carefully excavated using the scoop and cleanout tools and was then weighed, so that the density could be calculated. The accuracy of this device has been reported to be in the range of ± 0.3pcf (±4.8kg/m³) in specimens created by various raining techniques by
Bieganousky and Marcuson (1976). A similar technique in principle, using a vacuum cleaner to excavate the sand within a driven cylinder, was developed at the Danish Geotechnical Institute by Christensen (1961). A known volume of sand was extracted by using two different mouth-pieces of different lengths within the cylinder and the removed sand was retained for weighing. It was suggested that this method was reliable provided that the method was adequately calibrated by testing it on reference sand samples. This method was later used by Jacobsen (1976) but due to the very high degree of care required to give satisfactory results, it was suggested that a better method needed to be developed.

A number of surface density measuring techniques has been investigated by Griffen (1954). The wedge method involved inserting two steel plates diagonally into the sand until they met, to form a wedge. The results indicated an overestimate of the density of up to 9%. The tube method involved pushing a cylinder into the soil and then removing it whilst supporting the contained soil from beneath. This enabled a soil sample to be removed intact. The sand replacement method was also investigated. The common problems associated with the above methods, as mentioned by Griffen, was firstly that they all caused large disturbance of the soil and secondly, that they all required a large surface area of soil per test, thus limiting the number of measurements per layer across a limited size specimen.

The above methods are all restricted to the determination of the density of the sand at the existing surface only and therefore cannot be used to measure the density throughout a completed specimen. Several techniques have been investigated which enable the density at depth to be estimated. The Standard Penetration Test (SPT) has been used as a means of estimating the soil density through correlations relating the penetration resistance to the overburden pressure and relative density. A study in a 1.22m diameter by 1.83m high laboratory specimen of sand was reported by Marcusson et al (1977) and on a 3' (914mm) diameter by 4' (1219mm) high specimen by Gibbs and Holtz (1957). However, these laboratory tests have been primarily aimed at providing empirical data that can be related to measurements in the field, rather than developing a laboratory technique for density measurement. Marcusson et al (1977) concluded that the SPT was far from accurate for density determinations on site and therefore,
such a technique is considered, by the writer, to be unsuitable for laboratory specimens. However, the bearing capacity of a $\frac{1}{4}$" (6.3mm) diameter by $7\frac{1}{4}$" (184mm) long model pile in sand was found to be linearly related to the porosity by Walker and Whitaker (1967). The accuracy was shown to be comparable with that of the brass cylinder method as was mentioned previously.

An indirect method of measuring the in-situ density of dry sand using a thermal probe has recently been developed by Singh et al (1979). The 102mm long by 1.2mm diameter probe contained a heater element and a thermocouple which together were able to measure the thermal conductivity of a soil. Calibration tests conducted in a small container indicated a linear relationship between the sands density and its thermal conductivity. It would appear that the probe has been designed to be installed before the sand specimen is built, which therefore makes it unsuitable for specimens created by a compaction technique. However, if a method of inserting the probe after completion of the specimen could be developed, then this technique may have considerable potential, limited mainly by its cost.

An alternative process for obtaining the density at depth within a sand specimen has been to introduce an additional substance into soil which is capable of holding the soil particles together so that an intact sample can later be removed for analysis. The major variations in the techniques so far developed have been firstly, in the form of the binding substance and secondly, the methods of introducing it into the soil mass.

A number of impregnation techniques have been developed for the purpose of investigating the orientation of particles within a mass of soil. The use of natural resins, plastics, carbowax 6000 and latex to bind soils has been reported by Brewer (1964). The introduction of the binding substance was carried out in small samples under a vacuum of approximately 26" (660mm) of mercury. Clearly, this vacuum impregnation technique is unsuitable for large sand specimens. A detailed description of the properties of various impregnation materials has been presented by Smart and Tovey (1982 p 69-75). Vestopal W which is a copolymer of the polyester group was suggested to be the best impregning substance, although its high viscosity had to be reduced by the addition of acetone. Since the acetone is non-
reactive, it therefore has to evaporate off, thus making the mixture unsuitable for density measurements because of the uncertainty of the mass of resin that would remain. A number of epoxy resin mixtures were described and an appraisal is shown in Table 2.4 below.

<table>
<thead>
<tr>
<th>Epoxy Resin</th>
<th>Appraisal of characteristics relevant to density determinations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Araldite CY212</td>
<td>High viscosity of 15 poise</td>
</tr>
<tr>
<td>Araldite AY18+HZ18</td>
<td>Long setting time of 14 days but viscosity only 0.2 poise</td>
</tr>
<tr>
<td>Araldite A215+H215</td>
<td>Long setting time of 30 days</td>
</tr>
<tr>
<td>Epikote 815+812</td>
<td>Needs to be cured at 150°C</td>
</tr>
<tr>
<td>TAAB</td>
<td>Uses non-reactive acetone diluent</td>
</tr>
<tr>
<td>ERL 4206</td>
<td>Low viscosity at 0.08 poise but 14 days setting time and harmful fumes emitted</td>
</tr>
</tbody>
</table>

Table 2.4 Appraisal of epoxy resin mixtures relevant to soil density determination (after Smart & Tovey (1982))

The fabric of granular material has been investigated by Oda (1972a) using a polyester resin with a viscosity of 0.5 to 2.0 poise which permeated into the soil under its own head. A subsequent study for a sand saturated with water resin (Ostika resin) having a viscosity of 0.1 poise, was reported by Oda (1972b). The fabric of the sand specimen was maintained after a triaxial test as the resin solidified after only six hours.

A resin impregnation technique for measuring the density of Ottawa 20-30 sand and graded Ottawa sand using a laminac polyester resin (50ml) mixed with Cobalt naphthenate (0.3ml) and Lupersol DDM (10ml) was reported by Griffen (1954). Unsuccessful attempts were made with plaster-of-Paris, molten paraffin wax, hot asphalt and carbowax dissolved in alcohol. The polyester resin was injected under constant pressure into the sand via a needle with 28 peripheral holes connected to a syringe. The resin was allowed 20 minutes to congeal in the dry sand before excavation of the intact sample. The sample was weighed and then coated with hot paraffin wax before weighing in water in order to determine its volume. The
density was determined by subtracting the mass of injected resin from the mass of the sample and dividing by the measured volume. The results in the Ottawa 20-30 sand indicated an overestimate of the density for low densities and an underestimate for high densities. A similar decreasing trend with increased density was observed for the graded Ottawa sand except that all values were lower than the true values. Published photographs illustrate the sections of the excavated samples and it can be seen that the forced injection has tended to 'blow up' the centre of the sample. Clearly, this must have led to deformation of the soil and the large resin void at the centre could partly explain the recorded errors. The 'blowing up' of the sample would tend to defeat the object of the process, in which it is intended to excavate an undisturbed sample.

Additional errors could have been introduced due to volume shrinkage of the polyester resin (Brydson (1969) quotes a value of 8% for a polyester resin mixed with cobalt napthenate) and the decision to neglect the mass of the paraffin wax coating to the sample. The corresponding volume shrinkage for common paraffin waxes is 12.5% to 14.5% but is only minimal for epoxy resins.

Density measurements of saturated sand has been performed by introducing a gelatine solution which tends to solidify as it cools, (Emery et al (1972)). The void ratio was calculated by measuring the mass of sand remaining after washing the gelatine out of a carefully cut section of initially intact material.

A recent method has been developed by Wersching et al (1983), such that a soil specimen is built which contains pockets of the sand mixed with up to 10% by mass of Kaffir 'D' gypsum plaster. After a known quantity of detergent/water solution has been added to these pockets via a plastic pipe, the sand particles tend to become cemented together. The sample could be removed after 20 minutes and the density was obtained by weighing the sample in and out of water.

In conclusion, it is evident that there are a considerable number of density measurement techniques which can be used for dry sand specimens in the laboratory. The methods which are suitable for measuring the density within a sand specimen have generally been found to involve the
introduction of a binding substance into the soil so that it can be removed at a later date. However, if these substances are introduced during placement of the sand then it may cause significant changes to the properties of the sand. Consequently, the method of injecting a substance into the sand after the specimen has been built would seem to be a method which holds considerable potential but as yet requires modifications to improve its accuracy.

The remaining section of this chapter reviews previous work related to the performance of earth pressure cells.

2.3 **Performance Of Earth Pressure Cells**

2.3.1 **Types Of Earth Pressure Cell**

There are many designs of earth pressure cell in use today. They can be divided into two main groups according to their application. Firstly, there is the group of embedment cells that are designed to be embedded within a mass of soil and measure the in-situ stresses. Secondly, there are boundary cells that are designed to be mounted onto a rigid structure to record the earth pressures exerted by an adjacent mass of soil. A few cells have been developed to measure shear stress as well as normal stress, such as the strain gauge aluminium box capsule (Arthur and Roscoe (1961)) and the telemetric cell (Prange (1972)). However, the majority of cells are capable of measuring normal stress only, usually as a result of the deformation of the front face of the cell. The most common designs use a rigid cylindrical piston or a flexible diaphragm which compresses or deflects under pressure. There are numerous methods for relating the deformations to the applied pressure, which has therefore resulted in a wide range of cell types such as hydraulic cells, pneumatic cells, vibrating wire cells, strain gauge cells and semiconductor cells. The design, calibration and performance of the most noteworthy cells through the years has been summarised by the US Waterways (1944), Hamilton (1960) and Hanna (1973).
2.3.2 Calibration Tests

It is universally accepted that the shear strength characteristics of a soil creates complicated stress regimes around a deflecting cell and therefore, fluid calibrations cannot be used to accurately predict the pressures. As a result, there is a need for the pressure cells to be calibrated under laboratory conditions which represent, as close as possible, the conditions experienced in the in-situ environment. A number of techniques have been used in the past but they all basically involve placing the pressure cell within or adjacent to a mass of soil, applying a known stress to the soil boundary and recording the consequent cell response. The two most common methods of creating the stress regime use either a large oedometer (i.e., $K_0$ conditions) or large triaxial apparatus.

The major problem involved with the oedometer tests has been to limit the effects of side friction between the soil and the sides of the chamber so as to minimise its dimensions. A number of different methods have been developed for reducing side friction. The US Waterways Experiment Station (1944) carried out calibration tests on 12" diameter WES pressure cells in a 28" diameter by 10" deep rigid steel tank. The side friction was reduced by either using a thin layer of cup grease and oilcloth or a 2" layer of moist loess to line the walls of the chamber but neither method was found to produce a uniform or reproducible pressure distribution on the base. A third method was to use a stack of concentric steel rings separated by layers of caulking cotton, which had sufficient clearance within the walls of the chamber. A similar method was adopted to test 250mm diameter hydraulic pressure cells in a 500mm diameter by 300mm high chamber by Kallestenius and Bergau (1956) but in this case, the rings actually formed the chamber itself. Smith et al (1972) covered the walls of a 24" (610mm) diameter by 8" (203mm) deep tank in grease overlaid by a thin sheet of polythene. This was particularly important in this case because the hydroelectric pressure cells were 10" (254mm) diameter and sometimes three cells were placed at a single level, thus spanning the chamber. A similar technique using a rubber membrane was adopted for calibration tests on a variety of pressure cells in a 12"
(395mm) diameter by 12" (305mm) high chamber by Triandafilidis (1974). A double layer of rubber membrane with grease between was used to line a cylindrical volume of test soil 36" (914mm) in diameter by 24" (610mm) high by Selig (1980). A combination of building paper, aluminium foil and polyethelene was used by Bozozuk (1970) to calibrate various cells in a 4' (1.2m) square by 42" (1.07m) high plywood box. An alternative principle was adopted by Trollope and Lee (1961) by which no attempt was made to reduce the wall friction but instead the actual pressure distribution on the base was measured. This was achieved by building the base from a series of individually supported concentric rings. The load on each ring was measured by three strain gauged supports. The pressure distribution within the 36" (914mm) diameter chamber was found to be uniform in the central region for a 6" (152mm) depth of soil but became less uniform for larger depths.

Side friction has been eliminated altogether by the use of a water supported specimen as used for triaxial testing. This method was adopted to calibrate 10" (254mm) diameter strain gauge pressure cell with a 37" (940mm) diameter by 39" (990mm) high triaxial soil specimen by Plantema (1953). Similarly, tests on a 15" (381mm) diameter strain gauge pressure cell were conducted by Dunn and Billam (1966) in a 9" (229mm) diameter by 18" (457mm) high triaxial soil specimen. Carder and Krawczyk (1975) performed tests on hydraulic, strain gauge and pneumatic pressure cells at a rigid boundary at the end of a 540mm diameter by 300mm high specimen subjected to an isotropic pressure increase. However, uniform three-dimensional compression of a specimen would appear to be somewhat unrepresentative of the majority of situations for which such pressure cells are used. Generally speaking, the lateral movement of a structure causes the soil to compress in one direction only, with varying degrees of expansion occurring in the directions parallel to the face of the structure.

Another criterion for determining the required size of a test chamber has been proposed by Taylor (1945) based on the volume of soil required to develop a pressure bulb above the face of a pressure cell. It was concluded that the height of the test chamber should
be at least twice the cell diameter and that the diameter of the test chamber should be at least four times its height.

The primary objective of a calibration test is to accurately recreate the conditions that exist for the in-situ pressure cells. It would seem therefore that the choice of whether to perform $K_o$ or triaxial tests should depend on the in-situ conditions according to whether lateral straining of the soil is expected or not.

However, it should be noted that triaxial testing of large specimens generally requires more expensive and elaborate apparatus than that for $K_o$ testing, and it is consequently felt that these factors have in the past had a large influence on governing the choice of test method. The effect of the different test conditions has been discussed by Weiler and Kulhawy (1982) and the most notable difference between the two tests is the large amount of hysteresis that is recorded for embedment cells using a $K_o$ calibration. In contrast, the hysteresis for the triaxial test is small and often disappears with cycled loading. The large hysteresis of embedment cells in the $K_o$ test was explained as being due to the increased ratio of lateral stress to axial stress during unloading compared with that previously experienced during loading. It is suggested that the relative increase in lateral stress is recorded by the cell due to a lateral stress rotation effect, as described by Askegaard (1963) for a rigid ellipsoidal inclusion within an elastic medium. It was found that a proportion of the lateral stress was recorded by the cell as a normal stress and that the effect was higher for a decrease in the Poisson's ratio of the soil and the aspect ratio of the cell. (Aspect ratio = cell thickness/cell diameter). Weiler and Kulhawy (1982) have suggested that, for the case of calibrating flush mounted cells, the lateral stress rotation only occurs due to the effects of wall friction, (see section 2.3.7). This tends to suggest that such hysteresis recorded in the calibration tests would be representative of similar hysteresis effects experienced by pressure cells installed into full size structures. Nevertheless, both embedment and boundary types of cell record a linear cell response during loading because the amount of lateral stress rotation is directly proportional to the axial stress, as a result of a constant value of Poisson's ratio.
2.3.3 Effects Of Cell Aspect Ratio

The effect of the aspect ratio of an embedded pressure cell on its pressure response has been investigated by the US Waterways Experiment Station (1944). It was concluded that a limiting aspect ratio of less than 0.2 was sufficient to minimise the errors in cell registration for a cylindrical WES pressure cell. These results have been subsequently verified by Taylor (1945), Monfore (1950) and Tory and Sparrow (1967).

2.3.4 Effects Of Soil/Cell Stiffness Ratio

It has long been recognised that the response of a pressure cell is highly dependent on the soil/cell stiffness ratio. It has been found that an embedment pressure cell which is stiffer than the soil will over-read, whereas both embedment cells and boundary cells will under-read if they are softer than the soil. This effect has been investigated theoretically by Monfore (1950) and Tory and Sparrow (1967) and has revealed that the error in cell response is less susceptible to changes in soil stiffness when the cell is stiffer than the soil. Weiler and Kulhawy (1982) have recommended that a soil/cell stiffness ratio, F (or flexibility factor), less than 0.5 should be achieved in order to obtain consistent results, where

\[
F = \frac{E_{\text{soil}} d^3}{E_{\text{cell}} t^3}
\]

where

- \(d\) = diameter of diaphragm
- \(t\) = thickness of diaphragm
- \(E_{\text{soil}}\) = elastic modulus of soil
- \(E_{\text{cell}}\) = elastic modulus of material

The US Waterways Experiment Station (1944) investigated the effect of projection of a cell from a rigid surface. It was found that if the ratio of the cell diameter to its projection was greater than 30, then the discrepancy in cell response as compared to a flush mounted cell would be negligible. However, for a diameter/projection ratio less than 30, the discrepancy was found to increase rapidly.
2.3.5 Effects Of Diaphragm Flexibility Of A Pressure Cell

It is widely accepted that as the diaphragm of a pressure cell deflects, the soil tends to arch across onto the relatively stiff material that surrounds. As the deflections become larger, the pressure cell tends to increasingly under-read the actual pressure exerted by the soil. However, it should be noted that the tendency to under-read for large deflections must be superimposed on the effects of other parameters such as the soil/cell stiffness ratio. The US Waterways Experiment Station (1944) and Taylor (1945) have carried out tests to determine the range of the deflection/diameter ratio over which a linear response could be obtained. The limiting values of the ratio were found to be 1/2000 and 1/1000 for embedded and flush mounted cells respectively. A recent report by Weiler and Kulhawy (1982) has suggested that a limiting value of the deflection/diameter ratio of 1/5000 may be necessary for dense soils.

2.3.6 Influence Of The Method Of Installation

The installation method for a pressure cell has been generally shown to have a marked effect on the accuracy of the pressure measurements. The main requirement is to place the pressure cell within a soil mass or against a rigid structure without affecting the properties of the soil. Taylor (1945) has shown that a region of soil with a relatively low compressibility adjacent to the cell will cause it to over-read, whereas a region of soft soil will cause the cell to under-read. Trollope and Lee (1961) have suggested that the recorded pressures may be affected by up to $\pm 15\%$ of the true value, as a result of density variations and Hadala (1968) has suggested the error may be as high as $\pm 40\%$. The problem is not so severe for cells mounted onto a rigid structure because they can be installed so as not to interrupt the backfilling operation. However, the cells embedded within the soil must be installed whilst the backfill is being placed. The US Waterways Experiment Station (1944) and Weiler and Kulhawy (1982) have recognised the need for a cell to be strong enough to withstand the temporary high stresses caused during compaction. This may require placing the cell in a small excavation in the backfill. However, the cell response is then
likely to be affected by a difference in density between the soil replaced in the excavation and the surrounding material.

2.3.7 Effects Of Surface Friction For Flush Mounted Cells

The influence of the friction between the soil and a rigid structure for a flush mounted cell has been investigated by Carder and Krawczyk (1975). They found that the recorded pressures, for pneumatic and hydraulic diaphragm type pressure cells mounted onto a frictionless base with a sand specimen, were up to 20% less than those on a frictional base, although no difference was recorded for the strain gauge cell with a rigid top plate. It was suggested that the frictional base caused a reduction in the lateral stress and thus changed the nature of the stress distribution in the soil immediately above the cell face. This appears to indicate the effects of lateral stress rotation due to wall friction as have been described earlier by Weiler and Kulhawy (1982). In addition, Carder and Krawczyk suggested that the diaphragm cells were greatly affected by the lack of base friction because it allowed the soil to strain laterally, thus changing the deflected shape of the diaphragm.

2.3.8 Effects Of Temperature

The change in cell response caused by a temperature fluctuation of a strain gauge pressure cell has been investigated by Williams and Brown (1971). They observed a zero drift of the pressure cell of about 3.5kN/m²/°C but this was attributed to an asymmetrical layout of the four arm strain gauge bridge circuit. The gradient of the calibrated cell response was found to be unaffected by changes in temperature.

2.3.9 Effects Of Soil Grain Size

The influence of the soil grain size on the performance of a pressure cell has been investigated by Kallestenius and Bergau (1956) and Weiler and Kulhawy (1982). The former found that the mean soil grain size against a rigid piston pressure cell should be less than 2% of the active face diameter of cell in order to keep the error
caused by the point loadings to less than 3%. The latter suggested a less restrictive limit of the mean grain size as 10% of the active cell face diameter for a diaphragm type pressure cell, which would still indicate a pressure equivalent to a uniform load.

2.4 Summary

The review of the literature has revealed that there is considerable uncertainty about the behaviour of spillthrough abutments, which has consequently resulted in rather vague methods of estimating the loadings caused by earth pressures. However, consideration of some of the characteristics relating to other structures, such as retaining walls and piled foundations, has led to a broader insight of several factors which may influence the performance of spillthrough abutments.

A large number of methods have been found for measuring the density of laboratory specimens of dry sand. Although the majority of the methods have been reported to be fairly accurate, their applications are often restricted to the particular conditions for which they were developed. Consequently, the method of density determination should be chosen after considering the type of soil and the requirements of the test.

Finally, the review of the performance of earth pressure cells has revealed a number of factors which should be considered in order to obtain the 'best' interpretation of the data. These have been found to be primarily related to the structural characteristics of the pressure cell and of the type of soil. Consequently, it has been shown that calibration tests are required to establish the response of each particular type of pressure cell in a representative sample of soil, if the data from monitored structures is to be analysed effectively.

The following chapter describes the details of the experiments that were carried out during the course of the present work.
Figure 2.1 Strains required to reach active and passive states in a dense sand (After Lambe and Whitman (1979))

Figure 2.2 Relationship between wall displacement to develop active failure based on $\tan\theta_{max}$ definition and wall height for various angles of internal friction of backfill soil (After Sherif et al (1982))
Figure 2.3 Initial earth pressure during compaction (After Broms (1971))

Figure 2.4 Earth pressure distribution for a rigid unyielding wall (After Broms (1971))

Figure 2.5 Notation for spacing of stabilising piles (After Ito and Matsui (1975))
Figure 2.6 Failure mechanism for centrifuge model tests on a row of embedded piers (After Ah-Teck (1983))

Figure 2.7 Illustration of pressure distributions on row of embedded piers (After Ah-Teck (1983))

(a) Wide spacing ($S>2N$)

(b) Close spacing ($S<2N$)
Figure 2.8 Design envelopes for row of piers embedded in dense sand (After Ah-Teck (1983))

Figure 2.9 Notation for spacing of anchor plates (After Ovesen (1964))

Figure 2.10 Load settlement curves for piles (After Burland and Cooke (1974))
EXPERIMENTAL DETAILS

The performance of spillthrough abutments has been investigated by two different forms of experimentation. Firstly, a pair of full size structures have been monitored and secondly, a series of model tests have been performed to study individual aspects of the behaviour. The details of the two forms of investigation are discussed separately below.

3.1 Monitoring Of Full Size Spillthrough Abutments

3.1.1 Introduction

In 1981 the University of Surrey began monitoring two full size spillthrough bridge abutments, funded by the Department of Transport. The aims of the investigation were to determine the soil pressures and the internal strains in the concrete columns of the abutments and relate them to the construction events and the movements of the abutments. The enormity of the task required the work to be split up to permit each of several researchers from the Department of Civil Engineering to investigate a distinct aspect of the problem. The writer was primarily involved in the soil mechanics aspect of the project and therefore, the basis of the subsequent details will be so related.

The Wisley abutments were monitored by means of several different forms of instrumentation. Earth pressure cells were installed to record the horizontal and vertical pressures exerted by the backfill and strain gauges were installed in the abutment columns to record the bending effects. In addition, the abutment movements were monitored by precise surveying techniques and by inclinometer tubes. To obtain the best results from the pressure cells, it was necessary to perform a series of calibration tests in the laboratory. Considerable efforts were also made to obtain reliable measurements of the in-situ density of the backfill.
3.1.2 Details Of The Wisley Abutments

The spillthrough abutments that were chosen for this investigation were the north and south abutments of the bridge that carries the A3 London to Portsmouth road over the new M25 motorway at Wisley, Surrey, (Plate 1.1). The bridge was surrounded by four other smaller, single span bridges which formed the remainder of the three-level diamond shaped interchange, as shown in Figure 3.1.

The monitored abutments stood at either end of a 76m long, four span continuous voided deck slab, as shown in Figure 3.2. The deck slab was supported on fixed bearings at the intermediate columns and on free sliding bearings at the abutments. The bridge was divided into two halves by a longitudinal joint along the centre line, which enabled the two carriageways to act independently. The only means of connectivity between the two halves was via a 24m long by 5.5m wide continuous base slab at each abutment, as shown in Figure 3.3. The deck was built at a skew of 3° to the abutments. Each abutment supported the full height of the embankment at the rear up to the finished road level of the A3 London to Portsmouth road and at the front, the embankment sloped down at a gradient of 1 in 2 to the verge of the M25 carriageway. The abutments towards the London and Portsmouth ends of the bridge have been described as the A and E abutments respectively. Both abutments were founded below the existing ground level, (Figure 3.2), on varying beds of granular and sandy soils. The total height of the A abutment was typically 9.4m, consisting of a 1m thick base slab, a row of 5.3m high columns and a 3.1m deep capping beam. The overall height of the E abutment was approximately 0.5m shorter due to 0.8m shorter columns and a 0.3m deeper capping beam. Furthermore, the heights of the columns changed across the width of the abutment to produce the road camber. Each abutment consisted of six columns (three to each carriageway), spaced at 4m centres, (Figure 3.3). However, the instrumentation was only installed in the vicinity of the two most westerly columns of both the A and E abutments. The outer and inner columns have been referred to as being on lines 2 and 3 respectively. Each column was rectangular in cross section, 800mm wide and with a side length of 1300mm. The
dimensions of the instrumented sections of the abutments are shown in Figure 3.4. A 7m long cantilever wing wall was hung at 90° from each end of the capping beams, (Figure 1.2).

The backfill around the columns was a medium to fine sand obtained from a local cutting of the M25. Samples were obtained from around the columns before it was compacted and the grading curves are shown in Figure 3.5. A variation in backfill material was observed between the two abutments because the A abutment was backfilled during March 1982 and the E abutment during May 1982. The embankment was generally compacted with a vibratory roller and the weight was increased to 72T after a 54T was found to produce unsatisfactory densities. The backfill immediately adjacent to the columns was compacted by a whacker plate and a Bomag BW90S hand roller.

The backfilling behind the capping beam commenced in October 1982 but was temporarily halted during the wet winter months and was finally completed in April 1983. The backfill material was no longer the locally obtained sand but instead was an imported sand with a considerable clay and gravel content. This material was used to form the majority of the embankment from the base of the capping beam up to road base level. The backfill within a 2.5-3.0m vertical band close to the capping beam, however, was a selected well-graded naturally occurring granular material containing occasional lumps of sandstone and clay. A 0.5m thick vertical layer of coarse gravel was placed between the rear of the capping beam and the selected backfill to allow drainage to underlying porous drains. The grading curves for the coarse granular and selected granular backfill are shown in Figure 3.6. Compaction of the backfill was performed in approximately 500mm layers with up to ten passes of a Stothert and Pitt Vibroll T182A. The backfill was compacted to within 500mm of the capping beam by the large roller and closer compaction, particularly around the wing walls, was achieved with the Bomag BW90S hand roller and whacker plate as before.

A detailed account of the dates of the various stages of construction is presented in Table 3.1. A common time base was
introduced to simplify the processing of the data so that day 1 was defined as 1st December 1981.

<table>
<thead>
<tr>
<th>Construction operation</th>
<th>Abutment A</th>
<th></th>
<th>Abutment E</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Date</td>
<td>Day number</td>
<td>Date</td>
<td>Day number</td>
</tr>
<tr>
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<td>3</td>
<td>12.02.82</td>
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<td>09.03.82</td>
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<tr>
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<td>11.03.82</td>
<td>101</td>
</tr>
<tr>
<td>Column pour A3</td>
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<td></td>
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<td></td>
</tr>
<tr>
<td>Column backfill</td>
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<td></td>
</tr>
<tr>
<td>Start</td>
<td>10.03.82</td>
<td>100</td>
<td>29.04.82</td>
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<tr>
<td>Completion</td>
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<td>Road surfacing complete</td>
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<td>530</td>
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<tr>
<td>Road open to traffic</td>
<td>25.05.83</td>
<td>541</td>
<td>25.05.83</td>
<td>541</td>
</tr>
</tbody>
</table>

Table 3.1 Dates of major construction events

3.1.3 Instrumentation

A total of 54 vibrating wire soil/structure boundary pressure cells containing temperature coils and 12 vibrating wire Maihak soil pressure cells were installed by the University of Surrey. The boundary cells were installed into the upper surface of the base, at various levels in all faces of the two most westerly columns of each abutment and at two levels in the back face of the capping beams and the exact locations are defined in Appendix D. The linear range of a pressure cell was governed by the stiffness of the diaphragm and this was varied according to the predicted earth
pressures at various positions on the abutments by using diaphragm thicknesses of 2.5mm, 3.0mm and 4.0mm. This meant that the 4.0mm cells were positioned into the upper surface of the base slab to record the high vertical pressures. The 3.0mm cells were installed into the rear face of the columns and at the bottom of the front face. The remaining positions in the front and sides of the columns and in the rear of the capping beam were occupied by the 2.5mm cells. The Maihak cells were embedded into the backfill at various levels, 1m from the rear face of the abutment, in an attempt to record the vertical stresses in the soil. In addition, 42 pneumatic pressure cells were installed by the TRRL in a number of orientations around the line 2, 3, 4 and 6 columns of the A abutment only. The positions of the pneumatic cells whose results have been used for comparison with those of the vibrating wire cells are shown in Appendix D. No pneumatic cells were placed at capping beam level. The vibrating wire and pneumatic pressure cells are shown in Plate 3.1

The action of vibrating wire cells is to deflect away from the soil, enabling the soil to arch across onto the more rigid concrete that surrounds them, thus causing the cell to underestimate the true pressure. Conversely, the pneumatic cells operate by inflating the diaphragm into the soil, thus causing an overestimate of the true value. Consequently, by using both types of cell, an upper and lower bound to the true pressure can be determined. In order to further narrow the range of uncertainty, a series of soil calibration tests were carried out on each type of cell, as described in section 3.1.4 of this report. All the cells were designed to measure the normal stress only and no direct attempt was made to record the shear stress exerted by the soil on the structure.

The vibrating wire pressure cells needed to be mounted flush with the surface of the concrete so as to minimise their influence on the soil pressures that they were intended to record. A special set of shutters were used to form the columns, with a 300mm square opening at the required level of the pressure cell. The cells were loosely wired to the reinforcement cage before the shutters were positioned. The cell could then be removed and bolted to the box-
out panels and replaced in the opening prior to concreting. The bolts were removed before striking the shutters, thus revealing the cell mounted flush with the surface of the concrete, as shown in Plate 3.2. The wires were ducted through the column to emerge at base level and were then ducted, via 100mm PVC pipes, through the embankment to a remote survey manhole where a junction box was located. However, the above technique was unsuitable for positioning the cells in the top of the base and the rear face of the capping beam. Consequently, in these instances, it was decided to use box-outs to create voids in the surface of the concrete. The cells were later grouted into the void, flush with the surrounding concrete. The wires were run along grooves in the surface of the concrete and were finally ducted to the survey manhole as before. The pneumatic pressure cells were installed by TRRL staff. They were fixed to the surface of the concrete with a bed of plastic padding and the tubing was ducted away to a remote hut containing a compressor and recording equipment.

The surface of the abutment was coated with bitumen before backfilling. The sand backfill was placed and compacted in the usual manner, adjacent to the cells in the base and the columns. However, for the cells in the rear face of the capping beam a modified technique was required. In this case, the adjacent material was a coarse granular material and it was felt that large stones adjacent to the cell diaphragm may cause a series of point loads on its surface and thus give a misleading cell response. Therefore, it was decided to replace the coarse granular material adjacent to the cell, by a 600mm diameter by 200mm thick pocket of sand, similar to the sand used at the lower levels. It was realised at this stage that the pocket might cause either an arching effect or a load concentration effect but this was considered preferable to having large stones pressing against the cell face.

The Maihak pressure cells needed to be embedded within the soil mass itself. The method of installation was to excavate a shallow trench in the surface of the backfill and evenly bed the flat surface of the cell onto the soil at the bottom. The replaced soil above was recompacted by hand tamping so as to reduce the risk of
damage to the cell. When placing the cells in the soil behind the capping beam, the cells were embedded within a thin layer of fine sand at the bottom of the trench, as shown in Plate 3.3. This again prevented large stones from creating point loadings on the cell face. The compaction of the subsequent layers was continued in the normal way. A length of heavy duty cable was connected to the cell to eliminate the need for a PVC duct and so reduce any adverse effects on the pressure readings caused by excessive interruption of the stress field. At a remote distance, a finer cable was ducted across to the manhole. The junction box at the survey manhole enabled all the readings from the pressure cells (and strain gauges) to be read with relative ease.

An inclinometer tube was cast inside the abutment at the E3 column to record the deflected profile of the abutment over its entire height. A second tube was installed in the backfill 1m from the rear face of the abutment, midway between the E2 and E3 columns and was fixed at its lower end to the base slab of the abutment. This tube was installed with the intention of measuring the lateral movement of the soil. A series of ring magnets at various depths, placed around the inclinometer tube, provided a means of measuring the settlement of the backfill by passing a Soil Instruments magnetic settlement probe down the tube. The lateral deflections of the abutment and soil were obtained by lowering a Soil Instruments slope indicator (of the strain gauged type) down the tubes to record the angles of tube inclination at predetermined depths. The lateral movement could then be deduced from the change in successive deflection profiles. Unfortunately, the accuracy of the inclinometers was considerably reduced due to undesirable drifting of the zero reading of the probe and this resulted in the need to change the probe during the course of the study. In addition, a mindless act of vandalism caused irreparable damage to the tube situated within the backfill and consequently, this did not provide any useful data.

Two other forms of instrumentation (with which the writer had only limited involvement), were strain gauges within the columns of the abutment and a precise surveying technique for monitoring displacements. A series of vibrating wire strain gauges were
installed at various levels to record both concrete and steel strains within the columns. These were installed so as to provide an indication of the distribution of bending moments. The precise surveying was carried out from a network of six permanent reference stations located away from the influence of the construction. Two of these stations, D and E, (Figure 3.1) were installed within the survey manholes used for recording the pressure cell and strain gauge readings. Horizontal and vertical deflections were determined by measuring angular displacements and distances from stations perpendicular to the expected direction of movement. A Kern DKM-2A one second theodolite was used to measure the angles by sighting onto demec pips attached to the wing walls of each abutment. A Geodimeter 120 EDM was used to measure the distances to 10mm diameter acrylic reflectors located 100mm above the demec pips. These surveying techniques have been described in detail by Kennie (1984). Vertical settlements were recorded using a precise level and a network of studs across the carriageways.

3.1.4 Pressure Cell Calibration

As described in the previous section, the action of pressure cells in soil is much different to that in a fluid. It was therefore decided that fluid calibrations would give an inadequate estimate of the true soil pressures and consequently a 500mm deep, 1000mm diameter rigid steel tank (Figure 3.7) was constructed for carrying out calibrations in soil. The depth of soil within the tank could be varied by inserting a series of concrete discs to create the required depth. A vibrating wire pressure cell was installed flush with the concrete surface by grouting it into a void at the centre of the disc. Several pneumatic cells were stuck onto the surface with plastic padding at varying radial distances, as shown in Plate 3.4. This format allowed for several cells to be calibrated in a single test and also gave an indication of the pressure distribution across the base.

Unfortunately, the Maihak embedment cells were not calibrated in soil as part of this study. However, the US Waterways Experiment
Station (1944) have shown that only minimal errors occur in cell registration for an embedded cylindrical cell with a thickness of less than 20% of the diameter. Therefore, it can be assumed that the errors would be minimal for a Maihak cell which has a rectangular pressure pad with a thickness of less than 5% of the shortest plan dimension. In addition, the Maihak cells are stiff compared to the soil and Monfore (1950) and Tory and Sparrow (1967) have found that the cell registration is approximately equal to zero for a cell which is stiff relative to the soil and has a low thickness to plan width ratio.

The soil was a medium grained sand as used for the backfill around the columns of the A abutment on site, (Figure 3.5). A wooden skip with a sloping base was specially constructed to transport the sand from the bin to the calibration tank. This enabled the total quantity of sand that had been placed to be determined by weighing the skip before and after each load had been deposited. The bulk density was determined from the total quantity of sand that had been placed and an accurate measurement of the volume of the calibration tank. The sand was placed in layers and was compacted either by hand tamping or by Kango hammer, to create average dry densities of 1507kg/m³ and 1586kg/m³ respectively. The moisture content was allowed to vary as a result of a gradual drying out process within the laboratory. Having levelled off the surface of the sand, a neoprene membrane and gasket were placed on top and the steel lid was then lowered into position. After bolting the lid down securely, the air/water bladder pressure system was connected to a tap in the lid and the void between the membrane and the lid was flooded.

A uniform water pressure was applied in increments to the upper surface of the sand by an air regulator connected to the bladder, thus imposing a K₀ loading condition on the sand (ie no lateral strain). Readings were taken on each cell, in a similar manner to that on site, after each increment of loading. For the vibrating wire cells, this involved recording the period of vibration of the wire caused by electronically plucking the wire. This enabled an equivalent fluid pressure to be calculated from the theoretical
inverse square proportionality to the period of vibration. The pressure exerted on the pneumatic cells was determined as being the air pressure required to inflate the diaphragm such that an air circulation of 80ml/min was steadily maintained. Several cycles of loading and unloading were repeated, up to a maximum pressure of approximately 220kPa which corresponded to the predicted maximum pressure recorded on site.

During the later tests, an LVDT was located at a central position in the lid to record the deflection of the soil surface during cycling and hence provide a measure of the elastic modulus of the soil.

The initial tests were carried out mainly to establish a suitable depth of soil to be used in the tank, bearing in mind that a shallow depth would prevent perfect arching of the soil and that for excessive depths, tank wall friction may become dominant. Eventually, it was decided to use a depth of 240mm, which was approximately twice the active diameter of the cell face for the vibrating wire cell. This condition satisfies the outlines laid down by Taylor (1945) who stated that "the height allowed for the pressure bulb should be at least two times the cell diameter and that the diameter of the pressure chamber should be at least four times the height of the soil mass".

Compaction in three layers, each 80mm thick, with a Kango hammer was adopted because the dry densities achieved proved to be very similar to those measured on site by the sand replacement method. This method of compaction also tended to give repeatable values of dry density which varied by no more than 2% of the average value.

The Cell Action Factors have been determined from the calibration tests with a 240mm depth of dense sand (dry density = 1586kg/m³). Each Cell Action Factor has been determined as the average cell response over an applied pressure range of 0kPa to 220kPa, for the 1st and 7th cycles and the values are given in Table 3.2, where

\[
\text{Cell Action Factor, } C = \frac{\text{Measured Pressure (based upon fluid calibration)}}{\text{Applied Pressure}}
\]
These results clearly indicated, firstly, that the more flexible the diaphragm of a vibrating wire cell, the more it will under-read and that the pneumatic cells tend to over-read, which is exactly as predicted. The reason why the 4mm vibrating wire cell tended to over-read is uncertain. However, the most likely explanation is that it was the first test to be performed with that particular array of concrete discs and consequently, they cracked during compaction, causing an uneven pressure distribution. Evidence of such cracking can be seen in the photograph in Plate 3.4. Secondly, it is apparent that repeated loading tends to reduce the Cell Action Factor for all of the cells. For the case of the vibrating wire cells, this is almost certainly due to the stiffening of the soil during the subsequent loading cycles, thus making it able to arch more efficiently over the cell face and consequently make the Cell Action Factor more different from unity. The Cell Action Factor as determined by Carder and Krawczyk (1973) for similar pneumatic cells in washed sand was 1.22. It is likely that the difference in cell response was caused by the varying test conditions (ie, triaxial and $K_0$ compression).

It was found that the loading cycles indicated a linear response with increased applied pressure. Occasionally however, the Cell Action Factor tended to reduce slightly at higher loads for the 2.5mm vibrating wire cell. This could well be explained by the fact that

<table>
<thead>
<tr>
<th>Cycle</th>
<th>Average Cell Action Factor</th>
<th>Vibrating Wire Cells</th>
<th>Pneumatic Cell</th>
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<tr>
<td></td>
<td></td>
<td>2.5mm 3.0mm 4.0mm</td>
<td></td>
</tr>
<tr>
<td>1st cycle</td>
<td></td>
<td>.607 .864 1.143 1.380</td>
<td></td>
</tr>
<tr>
<td>7th cycle</td>
<td></td>
<td>.485 .684 1.061 1.350</td>
<td></td>
</tr>
</tbody>
</table>

Table 3.2  Cell Action Factor variation with cyclic loading
soil modulus is stress dependent and the 2.5mm cell is much softer relative to the soil than the cells with thicker diaphragms, therefore causing an increased value of the Flexibility Factor, $F$, for increasing applied loads,

$$F = \frac{E_s d^3}{E_c t^3}$$

where $F = \frac{E_s d^3}{E_c t^3}$

$d = \text{diameter of flexible diaphragm}$

$t = \text{thickness of flexible diaphragm}$

$E_s = \text{modulus of elasticity of soil}$

$E_c = \text{modulus of elasticity of cell}$

Tory and Sparrow (1967) illustrated that the cell error increases for increasing values of $F$. By assuming a maximum cell diaphragm deflection to diameter ratio of 1/1000, as suggested by the U.S Waterways Experiment Station (1944), the figures in Table 3.3 are obtained. These values have been obtained using a similar approach to that described by Tyler (1976). The diaphragm deflection was expressed in terms of the strain of the vibrating wire and was substituted into the equation for the deflection at the centre of an encastre disc subjected to a uniformly distributed load obtained from elastic theory. This resulted in an equation relating the limiting pressure, $p$, to the strain of the vibrating wire, $\varepsilon$, and the thickness of the diaphragm, $t$, such that

$$p = 8.5 \times 10^{12} \varepsilon t^3 \text{ kPa for } t \text{ in metres}$$

For a limiting deflection/diameter ratio of 1/1000 the corresponding strain of the vibrating wire is $1500 \times 10^{-6}$, such that the limiting pressure becomes

$$p = 12.7 \times 10^9 t^3 \text{ kPa for } t \text{ in metres}$$

In order to reduce the cell error for the 2.5mm cell according to the above criterion, a maximum pressure of only 199kPa can be applied. This explains why the 2.5mm cell sometimes tended to increasingly under-read when the applied pressure approached a value of 220kPa.
Table 3.3 Limiting pressures for a maximum deflection/diameter ratio of 1/1000

<table>
<thead>
<tr>
<th>Cell Diaphragm Thickness (mm)</th>
<th>Max Pressure (kPa) for deflection/diameter = 1/1000</th>
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</thead>
<tbody>
<tr>
<td>2.5</td>
<td>199</td>
</tr>
<tr>
<td>3.0</td>
<td>343</td>
</tr>
<tr>
<td>4.0</td>
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</tbody>
</table>

After completing the 1st cycle, the measured pressure did not return to the zero value but a 'locked-in' stress was recorded, the magnitude of which depended on the depth of the soil and type of cell. As can be seen from Table 3.4, the locked-in stress, as recorded by the vibrating wire cells, increased for an increase in soil depth and also for an increase in diaphragm flexibility.

Table 3.4 Magnitude of locked-in stress for varying depths of soil

<table>
<thead>
<tr>
<th>Depth of Soil (mm)</th>
<th>Average locked-in stress (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2.5mm cell</td>
</tr>
<tr>
<td>88</td>
<td>-</td>
</tr>
<tr>
<td>140</td>
<td>-</td>
</tr>
<tr>
<td>200</td>
<td>-</td>
</tr>
<tr>
<td>240</td>
<td>11.3</td>
</tr>
<tr>
<td>295</td>
<td>15.0</td>
</tr>
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</table>

However, the pneumatic cells indicated a locked-in stress of approximately 30kPa regardless of the soil depth or position of the cell on the base. A similar effect was observed by the U.S. Waterways Experiment Station (1944) and Buck (1961). The WES suggested that the locked-in stress may have been due to residual pressure in the sand or due to the characteristics of the cell, and
Buck suggested that it was associated with hysteresis and non-linearity. The fact that higher stresses appear to be locked-in for the softer cells would tend to suggest that large deflections of the cell face lead to large movements of the soil particles adjacent to it. Thus, when the load is removed, the elastic energy stored in the softer cells is required to produce larger deformations of the soil in order to achieve full recovery. Furthermore, according to Lambe and Whitman (1979), when a granular soil is loaded for the first time the frictional interparticle forces act such that the lateral stress is less than the vertical stress (i.e., \( k_0 < 1 \)). During unloading, the lateral stress may exceed the vertical stress and larger interparticle forces are maintained than at the same vertical stress during loading. Therefore, after the first complete cycle there remains larger interparticle forces than before, which in turn resist the recovery of the cell diaphragm to its original position. This would tend to suggest that the locked-in stresses are recorded due to the lesser stored elastic energy available for the softer cells to obtain a full recovery. Also, the high lateral stresses during unloading tend to cause the frictional effects on the tank walls to increase and hence provide further resistance to the vertical recovery of the soil. This effect is further confirmed by the observation that when a greased liner was used around the tank walls, the locked-in stresses were reduced almost to zero. The increase in locked-in stress for larger soil depths can be attributed to the increased amount of sidewall resistance resulting from the larger surface area.

The locked-in stress experienced by the pneumatic cells was probably due to the increased soil density and corresponding increased soil modulus directly above the cell face, as a consequence of it being a high spot on the base. The weight of the soil above would thus tend to be concentrated onto the cell face rather than onto the more compressible soil that surrounds.

From Figure 3.8 it can be seen that the softer vibrating wire cells tend to produce larger hysteresis effects for the unloading cycle, whereas the pneumatic cell shows only a very small effect. According to Lambe and Whitman (1979) "The hysteresis is caused by
the elastic energy stored within individual particles as the soil is loaded". Thus, the larger deformations of the soil particles above the softer cells cause more elastic energy to be stored. Weiler and Kulhawy (1982) state that the hysteresis effect is magnified by the reduction of the tangent Poisson's ratio of the soil during initial unloading under $K_0$ conditions.

It was observed that repeated loading causes the cell response to become approximately repeatable after about the 6th cycle. This agrees well with the recorded soil deflection curve, shown in Figure 3.9 as obtained by the LVDT in the tank lid, which indicates that after the first few cycles the soil modulus varies repeatably for successive loadings. This confirms the results obtained by Kallestenius and Bergau (1956).

When incomplete loading and unloading cycles were carried out, it was observed that on subsequent re-loading, the Cell Action Factor would always approximately return to its initial value when the pressure was equivalent to the maximum pressure previously experienced by the soil specimen, as shown in Figure 3.10.

The pneumatic cells were positioned at varying radial distances across the base in an attempt to give an indication of the variation in pressure distribution. The results obtained showed an irregular pressure distribution with a typical measured pressure variation across the surface of the base of up to 70kPa at an applied pressure of 140kPa. The reasons for the variation could be due to density variations within the soil mass or is more likely due to excessive cracking of the base, (Plate 3A). The consequence of the cracking is to encourage load shedding onto the stiffer areas of the base. It is therefore suggested that for such an investigation, an extremely rigid base should be used and an exact knowledge of the factors affecting the performance of the chosen pressure cell should be appreciated.

A fluid calibration was also carried out for each cell so that the reading from the vibrating wire cell could be related to an actual known applied pressure, thus enabling the cell response from the soil
calibration tests to be related to an equivalent fluid pressure. The fluid calibrations proved to be linear both for loading and unloading, which was exactly as desired.

As a result of the relatively large hysteresis effects experienced by the pressure cells, it was decided that such effects should be included when interpreting the cell response data obtained from site. To this effect, a computer program was written (by the writer) to convert the cell readings into soil pressures. The program assumed that the loading response was always linear and that all hysteresis loops for unloading were identical in shape but varied in magnitude. Subsequent re-loading was considered to be linear between the most recent point of unloading and the maximum previous load, thereafter following the initial linear response. The effects of the initial locked-in pressure due to compaction were not included in the analysis. The necessity for such a complex interpretation is due to the numerous partial cycles of loading and unloading experienced by the cells on site. The loading history for each of the vibrating wire cells installed at Wisley is given in Appendix E.

In conclusion, the calibration study has tended to suggest that the 4.0mm vibrating wire pressure cell is most suitable for installation into full size structures. This is because it is less susceptible to the effects of hysteresis and locked-in stresses than the softer cells. Furthermore, the calibration tests have indicated that the 4.0mm cell has a Cell Action Factor in soil close to unity. Also, the cell response remains linear over a pressure range which is perfectly adequate to cope with the high pressures caused by modern compaction plant.

3.1.5 In-situ Density Determination Of Backfill

The densities of the medium to fine sand, used to backfill around the columns, were measured by the sand replacement method with a 4" cone. The soil was sealed within a polythene bag immediately after excavation and was taken to the University laboratory for moisture content determination. As can be seen from Figure 3.11(a), the dry densities were somewhat lower than desired, with
lower values being recorded before the introduction of the 72T roller. The average bulk density was calculated as 1782kg/m^3 with an average moisture content of 10.7%.

The determination of the density of the backfill subsequently placed at capping beam level did not prove to be a simple task. Both the 4" and 8" cone sand replacement techniques proved unsuitable for measuring the in-situ densities of the fill materials. The excavation of the holes was unsatisfactory due to the disturbance caused by removal of the large elements of gravel or sandstone. Consequently, it was decided to perform large water replacement tests in an attempt to minimise the errors due to hole disturbance. Holes approximately 7000cc in volume were carefully excavated by hand and were filled to the top with water in a flexible polythene liner. A total of 12 tests were performed and the results are shown as dashed symbols in Figure 3.11(b). Clearly, the recorded results must have been subjected to considerable errors because they exceeded the maximum possible density that can theoretically be achieved for a soil with a Specific Gravity of 2.65. These results were therefore totally unreliable and had to be disregarded.

An attempt to explain this discrepancy was made by carrying out a water replacement test and subsequently a sand replacement test in the same excavation. The difference in bulk density as determined from these two tests were calculated to be negligible at less than 0.5% (ie. the volume of hole as calculated by both methods was almost identical). The excavated soil was immediately enclosed within a polythene bag and was taken to the University laboratory for weighing and for moisture content determination. The accumulative error due to a negative 1% error in the measurement of the mass of soil and a positive 2% error in the measured value of moisture content was calculated to only account for a 4% error in the dry density. This in no way accounted for the maximum error of 14% of the dry densities determined from the water replacement tests compared with the theoretical maximum dry density. The latter was based on total expellation of air from a soil with a Specific Gravity of 2.65. Consequently, the reason deduced for the large error was that the excavation must have
deformed significantly, thus making the measured value apparently small.

This could have been caused either by expansion of the soil into the void due to stress relief or by unnoticed deformation of the soil surrounding the excavation due to the weight of the person performing the test. At this stage, it was felt that an alternative technique should be adopted promptly before the backfilling operation was complete. The core cutter method was unsuitable, again due to the content of large particles within the backfill. It was therefore decided to use a nuclear density probe, as is commonly used for measuring the in-situ density of asphalt road surfaces. Fortunately, Wykeham Farrance Engineering Ltd kindly offered to supply a Troxler 34-11B nuclear probe and a qualified operator and so reduce any additional delay due to satisfying the numerous regulations required for the use and transportation of radioactive substances. The nuclear gauge was used on two occasions, separated by a period of over two months. Unfortunately, in both cases the surface of the backfill had been subjected to wet weather conditions due to the halting of the backfilling operation for the same reason. Consequently, the tests were performed by excavating a shallow hole, 75-100mm deep, to remove the overlying material that had clearly been affected by water penetration. The new surface was levelled with minimum disturbance by filling the surface voids with Leighton Buzzard sand. A 10mm diameter metal stake was driven vertically to a depth of at least 200mm to create the cavity for the nuclear density probe. The nuclear gauge was firstly calibrated on a standard block of plastic material with known density. The probe was used in the direct transmission mode such that 8 millicuries of a radioactive source of Caesium 137 was lowered 200mm vertically downwards into a pre-drilled hole. The gamma radiation then transmitted through the soil and was detected by a Geiger-Muller tube on the surface and a value of bulk density was displayed automatically. In addition to recording the bulk density, the gauge was capable of measuring the moisture content of the soil by using a neutron emitting Americium 241/Be source which is detected by a Boron Tri-fluoride (BF3) tube. This operation could only be carried out in the back scatter mode, which involved placing
the device onto the soil surface. The count rate in the BF3 tube is proportional to the amount of hydrogen present, which should in turn be directly related to the moisture content.

In addition to the nuclear moisture content tests, a sample of material was removed from a depth of about 100mm to 200mm and was used to determine the moisture content in the laboratory by the standard oven drying method. A comparison of the two sets of results indicated that the nuclear gauge consistently overestimated the values obtained in the laboratory by a moisture content of about 1% to 2%. A similar observation was reported by Kaderabek and Ferris (1979). The moisture contents determined in the laboratory were thought to be the most reliable because of the possible errors involved in the nuclear values. These were due to, firstly, the hydrogen content of any organic material within the soil and secondly, the possible increase in moisture content near the surface due to rain.

When using the direct transmission mode, a series of readings were taken in different radial directions from the same hole so as to give a better average of the readings. The results are shown in Table 3.5 which gives the average bulk densities and the average dry densities based on the moisture contents determined by the oven drying method. The range of dry densities recorded for each soil type is quite high but this appears to be due to having taken readings on two separate occasions and it was generally observed that the densities were larger for the first visit. The variation was probably due to the varying degrees of compaction. The individual tests results are shown as solid symbols in Figure 3.11(b).

<table>
<thead>
<tr>
<th>Material Description</th>
<th>No of Readings</th>
<th>Av Bulk Density</th>
<th>Av Dry Density</th>
<th>Range of Dry Densities</th>
</tr>
</thead>
<tbody>
<tr>
<td>Medium/coarse gravel</td>
<td>5</td>
<td>1865</td>
<td>1767</td>
<td>1740-1822</td>
</tr>
<tr>
<td>Selected granular backfill</td>
<td>12</td>
<td>2160</td>
<td>1959</td>
<td>1873-1999</td>
</tr>
<tr>
<td>Common backfill</td>
<td>6</td>
<td>2060</td>
<td>1801</td>
<td>1731-1893</td>
</tr>
</tbody>
</table>

Table 3.5 In-situ densities (kg/m$^3$) of soils behind capping beams measured with nuclear probe
The advantages of the nuclear density probe over the other standard tests that have been mentioned are as follows:

(i) It does not require large destructive holes to be dug into the backfill material.

(ii) Testing is rapid, (ie, up to 30 test locations, each with four readings can be achieved in one seven hour day).

(iii) The test does not require large quantities of sand or water to be carried across site.

(iv) The test can be carried out while backfilling is in progress because it is quick and unaffected by vibration unlike the sand replacement test.

A control test was carried out in a compacted sand specimen contained within the 1000mm diameter tank already described. The bulk density was accurately determined by weighing the sand contained within the tank and was subsequently measured using the nuclear density probe. The value obtained from the nuclear probe only differed by 0.8% of the calculated value, therefore justifying its accuracy. Checks on the accuracy have also been carried out by Reid (1983), by comparing the results obtained from a nuclear density probe with those from core cutter tests. Reid concluded that the nuclear probe provides a safe, fast testing method but that the results must be examined in a representative context. The precision of the nuclear technique was also investigated by Arora and Sacena (1979) who concluded that the method compared favourably with the conventional methods provided that a laboratory calibration was first carried out for the particular soil and equipment geometry.

Possible sources of error observed from the tests at Wisley could be due to inadequate surface preparation, lack of fit of the probe within the pre-drilled hole and poor calibration of the device against the standard density block.
The experimental details that have so far been described have all been concerned with the monitoring of the full size abutments at Wisley. However, in order to obtain a clearer understanding of the behaviour of spillthrough abutments, it was necessary to perform a series of model tests in the laboratory and these are described in the following section.

3.2 Laboratory Model Tests

3.2.1 Introduction

The manner in which a spillthrough abutment acts depends on a large number of variables which together form an extremely complex problem. Only by breaking down the global problem into smaller distinct studies can a reasonable appreciation be made of the various factors involved. Thereafter, having obtained an adequate knowledge of the individual mechanisms, their relative significance can be assessed and a better understanding of the problem as a whole be achieved.

Consequently, a series of model tests were performed under controlled laboratory conditions in order to investigate some selected characteristics of a spillthrough abutment. The tests were specifically designed to investigate the factors which were thought most likely to influence the performance of the structure. The most characteristic features of a spillthrough abutment are the columns connecting the capping beam to the base slab, which are buried within the soil embankment, (Figure 1.2). Clearly, these have a large influence on the stability of the abutment and are subjected to soil pressures on both the front and rear faces, whereas a common retaining wall abutment only experiences a soil pressure on the rear face. Furthermore, unlike a retaining wall, the complexities of soil/structure interaction cannot simply be assumed to be a two-dimensional problem. It was therefore concluded that the columns must play a substantial part in the overall performance. Thus, it was decided that the situation of a rectangular column moving laterally within a soil mass should be investigated.
A series of pull-through tests were designed to investigate the influence of the cross-section of a rectangular column (aspect ratio = face wide (B)/Side Length (D)) and the friction effects between the column and the soil. The tests were intended to provide information which would lead to a clearer understanding of the following points:

(i) The contribution of the load transmitted to the sides of the columns by shear stresses compared with that of the load experienced directly on the front face and how this varies with increasing column displacement.

(ii) The variation of the proportions of load taken by the sides and the front face of columns with differing surface roughness and aspect ratio (B/D).

(iii) The extent and mode of soil deformation caused by lateral movement of a column and the consequent influence of column spacing.

In order to understand more fully the contribution of the load on the column side faces, it was necessary to perform a number of pull-out tests and shear box tests so as to establish load-displacement relationships for the development of shear stresses.

Another important feature of an abutment is the base slab and how this may also affect the performance. A base slab is generally designed to provide an adequate bearing load for transmitting vertical deck loads safely to an underlying soil stratum. However, abutments are regularly subjected to considerable horizontal loadings and consequently, it was felt that the influence of a base slab on the nature and magnitude of resistance to lateral loading should be investigated. Column rotation tests were designed to investigate how a base slab would affect the following characteristics of a single column loaded laterally at the ground surface;

(i) Load-displacement relationships for the structure.
(ii) Centre of rotation of the structure.
(iii) Soil pressures on the structure and the corresponding induced bending moments.
With the knowledge obtained from the above experiments it was intended to then apply the general observations to the results obtained from the full size monitored abutment and hopefully provide a clearer explanation of the mechanisms involved.

3.2.2 Test Apparatus

It was necessary to design and build a tank suitable for performing all the required model tests, with a minimum number of modifications. The tank was designed primarily to be suitable for the pull-through tests because it was clear that the remaining tests could easily be catered for. The resulting design is illustrated in Figure 3.12.

The first consideration was to determine the size of tank that was required, having already chosen a maximum column size to be tested. It was felt at this stage that the plan dimensions of the tank should be at least 10 times the face width of the column so as to reduce the boundary effects on the specimen. With a maximum column size of 75mm x 122mm x 345mm deep, a rectangular tank with internal dimensions of 748mm x 814mm x 348 mm deep was chosen. The tank was constructed of solid 12.5mm thick mild steel plate, which was considered to be sufficiently rigid to produce insignificant boundary movements. It was feared that a tank of lesser stiffness may have affected the measured soil and column displacements of the model. The boundary effects due to friction of the soil against the steel tank walls were further reduced by lining the tank walls with 6mm plate glass, which according to published data (James and Bransby (1970)), has a small coefficient of friction of 0.1 with dry sand.

An alternative solution that was considered was a tank with glass walls supported within a steel framework that would allow full vision of the boundaries of the soil specimen. However, it was felt that glass alone would be insufficiently robust to withstand the considerable compaction forces exerted by a Kango hammer. In addition, the problems involved with creating a specimen which would provide a visual image of displacements at the glass/soil interface were considered to be an unworthwhile complication.
In order to be able to move the column laterally through the soil, it was necessary to control the loading or displacement at both ends. This presented an immediate problem of designing a tank base that would enable a load to be applied to the lower end of the column and yet not affect the stress regime in the adjacent soil. A solution was achieved by creating a longitudinal slot in the tank base to allow a rigid steel bar to protrude beyond the base of the column.

An overlying sheet of 6mm plate glass, with a small rectangular hole cut in it, was used to adapt the length and position of the slot to that required for the tests. The slot size could easily be further modified to cater for various column sizes by overlying another much smaller sheet of glass with an appropriately shaped hole formed in it.

The principle of operation was that a stiff steel bar should pass through a hollow rectangular column and be loaded at each end external to the model specimen itself. The hole formed in the upper sheet of glass was smaller than the column cross section so that it enabled sufficient movement of the column without creating a cavity through which the adjacent soil could flow.

The load was applied to the steel bar by a pair of hydraulic jacks which were operated by a valve system connected to an electric pump. The jacks provided a reaction loading between the steel bar and a rigid channel section bolted to the rear wall of the tank. This ensured that the measured column displacements were relative to the tank boundaries and not due to movement of the tank as a whole, as would have been the case if a remote reaction support had been used. A 2kN compression load cell was placed between each jack and the steel bar to record the applied loads. A perspex shoe was inserted to ensure that the ball and socket connection between the load cell and the jack could be centralised and therefore prevent an eccentric lateral loading due to misalignment of the column and the loading jack. A Linear Variable Displacement Transducer (LVDT) was positioned at each end of the steel bar to record the lateral displacements.
The load cells were calibrated over the full 2kN range with an Amsler dead load testing machine. The accuracy was found to be better than +1% over the full range. The LVDTs were calibrated using a digital micrometer fixed to a rigid block. Each LVDT was calibrated over a linear range of ±30mm using a set of precision steel slips which were inserted into the rigid block. The quoted output for the LVDTs was 3.65 Vrms/inch with a linearity of ±0.5%. The recorded accuracy of displacement readings over the calibrated range of ±30mm was better than ±2.5%.

The two obvious choices of material for constructing the model columns were perspex or aluminium sheet. The former material was selected because of its ease of construction and because it was readily available at the University in a number of different thicknesses without having to specially purchase large quantities.

Six columns were built, four of which were hollow in cross section, as shown in Plate 3.5(a). The two narrowest columns were solid, as shown in Plate 3.5(b). The dimensions of the models are shown in Table 3.6. A specially designed column, 37.5mm wide with a side length of 61mm, was built so that a sliding joint allowed the front face only to move forwards during loading. All of the columns were 345mm high.

<table>
<thead>
<tr>
<th>Face Width (mm)</th>
<th>Side Length (mm)</th>
<th>Cross Section (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>75</td>
<td>122</td>
<td></td>
</tr>
<tr>
<td>37.5</td>
<td>122</td>
<td></td>
</tr>
<tr>
<td>37.5</td>
<td>61</td>
<td></td>
</tr>
<tr>
<td>37.5</td>
<td>0 (61)</td>
<td></td>
</tr>
<tr>
<td>7.5</td>
<td>260</td>
<td></td>
</tr>
<tr>
<td>7.5</td>
<td>122</td>
<td></td>
</tr>
</tbody>
</table>

Table 3.6 Dimensions of model columns used for pull-through tests
In addition a rigid wall spanning the entire width of the tank was built from 25mm thick chipboard, faced with 1mm thick perspex and reinforced laterally with two perspex steel 'T' sections. The width was 764mm but a depth of only 138mm was used so as to be able to develop an uninterrupted shear surface within the boundaries of the tank. A length of hollow rectangular steel tube was buried in the lower region of the tank to create a cavity within which the steel loading bar could move laterally and yet prevent sand from flowing out through the hole in the glass sheet at the bottom.

Each model was tested twice, firstly, with a smooth perspex surface and then again after some test sand had been adhered to the perspex with Araldite to form a very rough surface.

The largest of the columns (75mm x 122mm x 345mm) was instrumented with a series of ten 120 Ohm foil strain gauges (Type N11-FA-5-120-23) supplied by Showa Measuring Instruments Ltd. They were positioned on to the internal side faces of the column at its mid-height in a line running from the front face to the back face. A dummy gauge was mounted in another perspex column and was used to complete a Wheatstone quarter bridge circuit. It was hoped that the variation of induced strain between the front and back of the column would give an indication of the shear stresses acting on the sides.

The data from all the load cells, LVDT's and strain gauges were recorded using a Solatron 3530 Orion datalogger, connected to a RDP demodulator.

The steel bar which passed through the hollow column and which was loaded at its ends was required to be sufficiently stiff so as to produce only minimal superimposed deflections of the column due to bending. For the case of the column with a 7⅓mm face width, the steel actually formed the central core of a solid column. Specially designed angle brackets were fixed onto the forward edge of the central core extension at each end so that the load cell could be secured and thus limit the tendency of the column to twist under load. The steel bar was supported by a roller bearing which allowed unrestricted lateral movement.
The whole tank arrangement was set up on rigid blocks to provide a central tunnel beneath in which the lower loading jack, LVDT and roller bearing could be located.

In addition to measuring the load displacement relationships of the columns, it was also required to record the zone of influence of the surrounding soil. This was achieved by implementing a photographic technique, similar to a method used by Ah-Teck (1983) at Cambridge University, in which a grid of 3mm diameter silver markers were photographed intermittently during testing to record soil displacements.

A gridded array of 3mm diameter carbon chrome ball bearings were positioned onto the surface of the sand by dropping them through a perspex template, as shown in Plate 3.6, similar to the method used by Ah-Teck (1983) for centrifuge tests on a row of columns embedded in sand. It was felt that the ball bearings may become partially covered by sand during testing. This would lead to inaccuracies in the measurements from the photographs because it would be difficult to locate the exact centre of a partially covered ball bearing. It was therefore decided to shine a high intensity spot lamp onto the surface of the sand, so that a well defined small point of light would be reflected from the upper surface of each ball bearing. This method enabled an accurate measurement of the ball's position to be made, even if it was partially covered by sand. The spot light was carefully adjusted to provide an even field of light across the surface of the sand so as to ensure an even film exposure, whilst producing a high intensity spot of light to be reflected, towards the camera, from the upper surface of the ball bearings.

Numerous trials were carried out to obtain the best film type, exposure and processing for producing the required accuracy and the details are given in Appendix A.

A Nikon F2 SLR camera fitted with a battery powered motor wind and 85mm Nikon Nikkor lens was attached to a rigid scaffold framework, at a height of 2.8m above the soil surface. The light
source was also supported on the same framework at a horizontal
distance of about 1m out from the camera. The effect of the
permanent laboratory lights were found to be insignificant due to
their low light intensity at larger distances and this enabled the
tests to be performed under conditions of full laboratory lighting.
The camera was triggered from below by a remote cable release
which avoided vibrations to the scaffolding. This was particularly
important because the photographs at various stages of loading
needed to be compared with each other in order to determine the
movements of each ball bearing. A series of datum points (in the
form of ball bearings) were included in the view of each photograph
so as to provide standard reference points with which each
photograph could be compared. These datum points were supported
independent of the tank so as to be uninfluenced by tank
movements.

A Hewlett Packard 85B desk top computer and HP 9872A plotter
coupled to an HP9121 disc drive was programmed (by the writer) to
digitise selected points on each print and produce a plot of
displacement vectors. A Quantamet scanner was also considered
because it was able to locate the centre position of all the ball
bearings on a photograph automatically. Unfortunately, it was not
entirely suitable because the complete scan was performed as a
series of horizontal scans progressively in a downwards direction.
This meant that the ordering of the coordinate pairs was dependent
on the orientation of the photograph with the upper points in the
field of view being detected first. Even if two successive
photographs were orientated identically, problems occurred due to the
relative movement of the ball bearings in the second photograph
which caused them to be scanned in a different order. It was
therefore impossible to perform a comparison of the coordinates of
each individual point. A computer link would have made this
technique more feasible but unfortunately such a facility did not
exist at the time.

In addition to the specialised photography already mentioned,
photographs were also taken to record the formation of failure
planes at the soil surface. This was visualised by creating black stripes across the sand which indicated the relative movement at the sand surface. The stripes were created by sprinkling a mixture of sand and black photocopier toner over a series of timber slats which were later removed. The static charge of the toner meant that a small quantity was sufficient to make a large quantity of sand become black, by adhering to the surface of the individual sand particles.

In order to perform the column rotation tests, the tank apparatus had to be modified so as to be suitable for measuring the rotational characteristics of a column, (Figure 3.13). The hydraulic jacks were removed and replaced by a single pulley at the upper edge of the front wall of the tank. A pulley and dead weight loading was thought to be better than hydraulic jacks in this instance because the lateral and vertical movements of the loading point were more easily catered for by a non-rigid load application system. The hole in the glass at the base was simply blocked by placing a sheet of glass over the top.

A hollow 24mm x 39mm x 570 mm long perspex column was built, which incorporated twenty 120 Ohm foil strain gauges glued onto the inside of each of the front and back faces for a distance of 288mm from one end. It was necessary to relate the measured strains from the gauges to the resistant bending moments within the column. It was therefore decided to perform a pair of four-point calibration loading tests to provide a direct relationship between recorded strain and imposed bending moment for each gauge. The column was calibrated for bending in two directions so as to give independent calibration factors for the gauges in both tension and compression. This technique was considered to be more accurate than simply converting the measured strain to a bending moment by using a purely theoretical value of the column stiffness. The soil pressure distribution was derived by double differentiation with respect to length of the bending moment distribution. The column was firstly tested without a base and was then modified and tested again this time with a base (120mm wide x 165mm long x 30mm thick), as shown in Plate 3.7. The dimensions and location of the column into
the base were scaled at $1:33^{1/3}$ of the size of a column in the Wisley abutment. An additional extension of the column was built above the required depth of embedment so that two horizontal LVDTs at known spacing could be used to determine the lateral movement and rotation of the column due to the lateral loading from the dead weight, as shown in Figure 3.13. A telescopic level was used to check the change in height of a datum point marked onto the column extension, so that any tendency of the column to pull out of the sand could be monitored.

The third set of tests to be carried out in the tank were some pull-out tests to evaluate the load-displacement relationship for the mobilisation of the shear stress. A 10mm thick sheet of perspex, 600mm in width with a buried depth of 345mm, was extracted vertically from the sand filled tank by a central hydraulic jack positioned on a frame directly above the tank. A 2kN tension load cell and two LVDTs (same as for the pull-through test) were used to record the loads and displacements of the sheet respectively. The test set-up is shown in Plate 3.8.

3.2.3 Sample Preparation

The model tests were carried out with the same sand as was used for the pressure cell calibration tests, except that it was firstly sieved through a 3.35mm sieve and was then dried at room temperature to a moisture content of 0.5%. The result of a sieve test before removal of particles larger than 2mm is shown in Figure 3.5. It was felt necessary to remove the large soil particles because they could adversely affect the results of the model tests due to their relatively large size compared with the perspex models themselves.

There were two main problems that needed to be overcome before a satisfactory standard of specimen preparation could be achieved. Firstly, a technique needed to be developed to accurately measure the density of dry sand within a sand specimen. Secondly, a repeatable method of soil placement was required that could produce a uniform sand specimen at a particular density.
As was mentioned in Chapter 2, many authors have reported on a wide range of techniques for measuring the density of soils under laboratory conditions. However, after some preliminary tests, it was decided that an improved method should be developed, based on a resin impregnation technique first reported by Griffen (1954). Basically, this technique involves the injection of a quick setting resin into the soil mass via a hyperdermic needle. This enables a congealed globule containing soil and resin to be removed for density determination. The immediately apparent advantages of the resin impregnation technique are as follows:

(i) Unlike the sand replacement test, it does not rely on the ability of the soil to be self supporting during excavation. This is particularly important for dry sand which tends to flow into an excavation.

(ii) It only involves a minimal amount of soil disturbance due to penetration of a hyperdermic needle.

(iii) It is a very suitable way of obtaining soil samples at depths within a specimen. It avoids the need for samples to be taken from the surface, as is the case for most techniques. Consequently, this allows the density within a completed specimen to be determined rather than at stages during its construction.

(iv) It is not reliant on the method of sand placement, as is the case for techniques which involve the raining of sand into a collecting container positioned on the sand surface. This therefore eliminates any errors in density measurement caused by the effect of the container on the way in which the soil particles arrange themselves.

The main disadvantage of this technique relative to the others is that it takes more time to actually perform the test. At least 18 hours must elapse before values can be obtained.
The main requirements of a resin suitable for impregnating a dense sand are listed below:

(i) There should be low viscosity for easy impregnation
(ii) Negligible volume change during setting and curing is required
(iii) The gel time should allow full impregnation and yet allow early removal of the sample.
(iv) Gel and cure must take place at room temperature in a short period
(v) The chemicals must be non-toxic, for use remote from a fume cupboard
(vi) No chemical reaction should take place with the soil particles
(vii) Rigidity after setting must be achieved to prevent damage during removal of the sample.

The most suitable resin found after a number of trials with epoxy and polyester resins was a formula of Stycast W19 epoxy resin and catalyst 24LV obtained from Emerson and Cumming Ltd and a silane reactive diluent obtained from Union Carbide Ltd. A non-reactive diluent such as acetone would have been unacceptable because it would evaporate and thus cause difficulties in measuring the quantity of resin injected into the soil. The mix proportions were:

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<th>Component</th>
<th>Parts by Weight</th>
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<tbody>
<tr>
<td>Stycast W19</td>
<td>100</td>
</tr>
<tr>
<td>Catalyst 24LV</td>
<td>30</td>
</tr>
<tr>
<td>Silane A1100</td>
<td>20</td>
</tr>
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</table>

The resulting resin mixture had a setting time of 6-12 hours at room temperature, and had an initial viscosity of approximately 0.05 poise, (cf. water = 0.01 poise).

The resin needed to be introduced into the soil mass at different locations with minimal disturbance. A number of stainless steel hyperdermic needles of various lengths were manufactured with an external diameter of 2.1mm and an internal diameter of 1.6mm. A
series of approximately seventy 0.8mm diameter holes were drilled around the circumference, along a distance of about 20mm from one end of the needle. The needle wall at this end was bevelled at an angle of about 10°. A 5ml polypropylene syringe casing with a tapered fitting was connected securely to the other end, as shown in Plate 3.9. In addition, a rod with a pointed end was built to fit inside the full length of the hyperdermic needle, such that it could be easily extracted after the needle had been inserted into the soil, thus preventing the needle from becoming blocked by sand.

The resin was dripped into the syringe casing from a section of polythene tubing, which was sealed at each end by a double sided heater element. This method prevented an excessive head pressure at the needle outlet, although it generally required continual attention to regulate the rate of dripping. The bag of resin was supported above the syringe by retort stands, as shown in Plate 3.10. The needle was removed from the sample and the mass of resin that had been introduced was determined by weighing the bag before and after. The samples could then be excavated after a period of 18-24 hours, which was sufficient for the resin to set rigidly. The mass of sand was determined by subtracting the mass of resin from the mass of the sample and the volume was obtained by weighing in water. Hence, the density of the sand could be calculated.

The successive steps involved in the test are given in more detail in Appendix B.

A number of preliminary tests were carried out to determine the practicability of the resin impregnation technique and also to assess its accuracy. A series of soil specimens of various densities were constructed in CBR moulds either by compaction in three layers or by pouring sand through a funnel at a set distance from the soil surface. The overall densities of the control specimens were then determined. A single resin impregnation test was carried out in each mould in the standard manner and the resulting as-measured densities were compared with the calculated control density. A plot of the results is shown in Figure 3.14 for a relative density range of 7% to 97%.
The relative density, $R$, was calculated from the equation

$$ R = \frac{\gamma_{\text{max}} (\gamma_d - \gamma_{\text{min}})}{\gamma_d (\gamma_{\text{max}} - \gamma_{\text{min}})} $$

where $\gamma_d$ = dry density of sample
$\gamma_{\text{min}}$ = minimum dry density
$\gamma_{\text{max}}$ = maximum dry density

The maximum and minimum dry densities were determined according to the methods outlined by Ackroyd (1964). The maximum density was obtained by compacting three equal layers within a CBR mould with a Kango hammer for 1½ minutes per layer. The minimum dry density was obtained by pouring the sand through a funnel into a CBR mould. The funnel was moved in a slow spiral motion with the outlet 5mm above the surface.

A least squares linear regression analysis on the data showed an overall overestimate of the control density, with the magnitude of error decreasing at higher relative density, (Figure 3.14). This is a similar trend to that recorded by Griffen (1954). The 95% confidence limits of the measured values on the regression line are $\pm 21.7\text{kg/m}^3$.

The errors observed are likely to be considerably contributed to by the uncertainty of any density variations within the CBR mould as a direct result of the methods of sand placement. It is possible that the placement of the sand in the upper region of the mould may have caused a densification of the lower region. This deduction can be justified by the density variations with depth measured within a large tank of sand, as discussed later. It was therefore decided that a correction for the as-measured densities was unnecessary.

It was found that a 40g quantity of resin would take up to three hours to fully impregnate the soil. Thus, it was decided that a 30g to 40g was an optimum range for the quantity of resin to be introduced. The corresponding resin impregnated soil sample was observed to be generally spherical in shape, with an approximate
volume of 120-160 cm$^3$. As can be seen from Plate 3.11, the resin balls have a series of peripheral ribs which coincide with the horizontal layering of the soil. The cross section shown in Plate 3.12 illustrates that the ribs correspond to the darker areas which are caused by almost total saturation of the voids, whereas the lighter areas represent partially saturated regions. This effect is probably due to layers of different permeability rather than directly due to layers of varying density, although the two must be related to some extent. This tends to indicate a layered variation in grain size distribution within the specimen. This could have been caused by the tendency of the dry sand to segregate due to minor vibrations of the scoop which was used to pour it into the mould. The regions of low permeability were likely to have a slightly higher density than the more porous regions. However, the multiple layers observed within the sample cross-section would tend to suggest that the measured density of the specimen as a whole is a good approximation of the average density of all the layers contained within it. The cross-section also shows that the lower portion tends to contain more resin than the upper portion. The upper portion must have been formed by capillary action causing the resin to rise because the peripheral holes in the needle were only present in the lower 20mm of the needle. The varying degrees of saturation do not affect the results provided that the quantity of resin is sufficient to hold the sand particles firmly cemented together. In addition, it can clearly be seen that the insertion of the needle caused minimal disturbance of the adjacent soil and that there is no evidence of the resin forcing the soil particles outwards, as was observed by Griffen (1954).

A dense specimen of dry sand was constructed in a metal tank with internal dimensions of 815mm x 750mm x 350 mm deep. The sand was compacted in three 120mm thick layers with a 200mm square steel plate attached to a Kango 628 vibratory hammer. The soil was compacted in a 16 position grid with the outer edges being compacted first in an alternating side to side manner followed by the central region. (Figure 3.15).
A total of 38 resin impregnation tests were carried out at depths of 90mm, 185mm and 280mm (Figure 3.16) on a sample compacted in three layers, with each position in the grid being compacted for five seconds, once only. The results, shown in Figure 3.17, indicated an increase in density towards the bottom of the tank. This increase was due to the additional compaction of the lower layers from the subsequent layers above. The drop in density towards the edges was due to the outer regions receiving compaction whilst the adjacent soil towards the centre was still loose and was thus unable to provide substantial lateral confinement. In addition, the side friction on the walls may have had some influence and the subsequent compaction of the central layers may have caused the outer regions to later become loosened.

A soil specimen consisting of densities ranging by 120kg/m\(^3\) cannot be considered as being uniform and would be unacceptable for a series of model tests. Consequently, it was decided to attempt to obtain a more uniform specimen by altering the compaction program.

After several unsuccessful variations of compaction, a modified compaction program was eventually formulated which showed a distinct improvement in the uniformity of the specimen. It again involved compacting with the vibrating hammer in the same 16 position grid but this time additional compaction was applied to the upper layers as indicated below:

- **bottom layer** - 5 secs per position in the grid
- **middle layer** - 5 secs per position in the grid, twice in succession
- **top layer** - 5 secs per position in the grid, twice in succession followed by 2½ secs per position in the grid

This specimen was tested with 27 resin impregnation tests, again at similar locations to those chosen for the first test. The results, shown in Figure 3.18, indicate that the densities achieved for each layer were very close and that the drop in density towards the edge was somewhat reduced. The density in the central region was
measured to be $1662\text{kg/m}^3 + 16\text{kg/m}^3$ for the three depths tested. The density variation at the edges only varied by $33\text{kg/m}^3$ from top to bottom. The drop in density from the centre to the edge of the tank was found to be a maximum of $50\text{kg/m}^3$. These results indicated a considerable improvement on the first test and were considered to be acceptable for carrying out model tests.

The resin impregnation technique proved to be a useful method for determining the density variations within a dry sand specimen created under limited laboratory conditions. The major advantage of the resin impregnation technique is that it can establish the density variation within a completed soil specimen, whereas the box density device (Sloan (1962)), plastic cylinder (Kolbuszewski and Jones (1961)), brass cylinder (Walker and Whitaker (1967)), vacuum technique (Christensen (1961)) and wedge, tube, sand funnel and water balloon (Griffen (1954)) are all limited to determining the densities on the temporary soil surface at various increments of construction. The importance of this has been illustrated by the first test in which it was seen that subsequent compaction of overlying layers caused a considerable increase in the density of the lower layer.

### 3.2.4 Procedure

The test procedure for each type of test is described separately as follows.

(a) **Pull-Through Test**

The perspex column was clamped into position in the tank with the steel bar passing through it. The small sheet of glass at the base of the tank was positioned to allow adequate lateral movement of the column and yet block off any opening through which the sand could escape. It was necessary to rigidly clamp the column into position because large movements of the column during compaction would be undesirable for two reasons. Firstly, it would affect the density and stress state of the adjacent sand and secondly, it may cause misalignment between the loading jack and the load cell on the steel bar.
The sand was compacted in three layers in a modified grid to that described in the previous section. This allowed for the sand to be compacted alternately on each side of the column but a general trend of compacting the outer regions first was again adopted so as to be as similar as possible to the original grid pattern. Thereafter, the surface was carefully levelled flush with the top of the tank with a steel rod. A series of black stripes were then created by sprinkling a mixture of photocopier toner and sand over a series of timber slats which were later removed to reveal the stripes of natural coloured sand. An array of ball bearings were then placed onto the surface by positioning them in a drilled perspex template which was later lifted away to leave the balls in place.

The loading jacks, load cells and LVDTs were positioned top and bottom, (Figure 3.12). The loading jacks were aligned by inserting a perspex shoe seating and following this the LVDTs were initialised on the datalogger. A series of ball bearings attached to retort stands were located around the tank to be used as datum points for the photography.

The camera was positioned into the overhead frame and the spotlight was switched on so that the lens could be accurately focused onto the sand surface.

A preliminary test was carried out with the 75mm x 122mm section column and with a sheet of 6mm plate glass, with a hole cut in it, clamped onto the upper sand surface. The load was then applied in equal increments to the top and bottom jacks and the datalogger recorded the lateral movements, loads and column strains. The camera was triggered remotely after each of the load increments. Several useful points emerged from this preliminary test and these are given below, along with the resulting modifications applied to the subsequent tests:

(i) The sheet of glass on the upper surface experienced large pressures from beneath due to the tendency of the sand to heave in front of the column. As a result, the glass cracked in several places radiating outwards from the central hole.
Consequently, it was decided not to use the upper sheet of glass for future tests because it did not adequately fulfil its purpose of providing a restraint to vertical movement of the sand. This in fact was a reasonable modification to make because it is unlikely that total vertical restraint is representative of the field conditions of a buried column, except at very great depth. Furthermore, movements of the ball bearings recorded by the photography may have been slightly reduced due to friction against an overlying sheet of glass.

(ii) The peak load on the top load cell was found to be significantly lower than that on the bottom load cell because the cracked glass allowed unsymmetrical failure conditions. It was therefore impossible to reach the peak load at the base by applying equal load increments because the single hydraulic circuit from the hand pump would only permit equal pressures to be applied top and bottom. Consequently, it was decided to use an electric pump to control the two jacks individually. Furthermore, it was concluded that regular incremental control of displacements rather than loadings would permit the test to be carried out until total failure occurred.

After each test, the film was removed from the camera and developed immediately to prevent any deterioration of the quality of the negatives. On completion of all the tests, a series of prints were made from selected negatives. A typical print is shown in Plate 3.13. By using a constant enlargement for each photograph, it meant that the displacements from each test could be compared without further scaling.

A program was written for the Hewlett Packard HP85 to allow the points to be digitised on a plotter. An optical pen was manually positioned over each dot, including the datum points, on the print and the coordinates were read in automatically and stored on a magnetic disc. This program was used to digitise the selected points on the initial photograph of each test. For subsequent
photographs, another program was written which firstly read in the
digitised coordinates of each datum point corresponding to the same
datum in the first photograph. Using the data from the two sets of
datum points, the shift and rotation between the two photograph
set-ups were calculated. This involved calculating the whole circle
bearing between each possible paired permutation of datum points
for both photographs. The required angle of rotation, $\Delta \theta$, was then
calculated as being the average of the differences in angle between
each corresponding pair of datum points from the two photographs.
The datum coordinates of the initial photograph were then
recalculated for a rotation of $\Delta \theta$ about the origin. The new datum
coordinates of the initial photograph were subtracted from the
corresponding datum coordinates of the second photograph for each
possible permutation of datum points. The required horizontal and
vertical translations, $\Delta u$ and $\Delta v$ respectively, were calculated as
being the average of the translations determined after considering
each corresponding pair of datum points from the two photographs.

The theoretical coordinates of the remaining points from the initial
photograph were then calculated for the new coordinate system and
the pen was programmed to automatically move to its initial
position. Consequently, only small movements were then required to
move the pen to the displaced position, and the points were again
digitised. The derived equations for the coordinate transformations
are given in Appendix C.

A third program was written which read in stored values from the
magnetic disc for the initial and displaced coordinates of each point.
Both sets of coordinates were then corrected to the same coordinate
system and a plot of enlarged displacement vectors was drawn.

(b) **Column Rotation Test**

The bottom layer of sand was compacted first and then the column
base was carefully lowered into an excavation to the required depth.
This ensured that the sand beneath the base was correctly
compacted. The column was held firmly in place during the
compaction of the subsequent layers. The base of the column was
kept approximately 60mm above the base of the tank to provide an all round sand support.

After compaction, the sand surface was levelled flush with the top of the tank and two LVDTs were positioned horizontally at different elevations against the column extension, (Figure 3.13).

The strain gauges were connected to the datalogger and were allowed 24 hours to warm up before commencing the test. The gauges were connected on a Wheatstone quarter bridge circuit and a dummy gauge was fitted to an identically sized column. The dummy was buried in an adjacent tub of sand to compensate for large temperature fluctuations. It was found that large temperature fluctuations occurred over a long time span causing considerable variations in recorded strain. However, the test duration was never greater than half an hour and so the variations were considered to be negligible.

The loads were added in 1kg increments to a weight hanger, which applied a horizontal load to the column at 5mm above the soil surface. At each load increment the strain and displacement readings were recorded on the datalogger.

A Fortran 77 computer program was written (by the writer) for the University Prime System to convert the strain readings into bending moments. A quintic polynomial was fitted to the bending moment data by using a least squares solution. The resulting expression was then differentiated twice with respect to depth to produce a soil pressure distribution which indicated the centre of rotation as being the position of zero pressure.

The stiffness (EI) of the column was evaluated from a simple three-point bending test. The deflected shape of the column was calculated by determining y, the column deflection, from double integration of the beam theory equation.
\[
\frac{M}{EI} = \frac{d^2y}{dx^2}
\]

where \( M \) = bending moment
\( EI \) = column stiffness
\( y \) = column deflection
\( x \) = distance along column

and substituting the relevant boundary conditions. This deflection was superimposed onto the linear rotation as obtained from the two LVDTs to give another check on the centre of rotation.

The test was carried out on the column alone, buried to a depth of 288mm and was then repeated after the base had been affixed. Initially, the base was connected by a tight push joint of the column into the base. However, it was suspected that this would allow some degree of column rotation within the base itself and so the column was later cemented into position and the test was repeated. A 5mm fillet was cemented around the circumference of the column on the upper side of the base in an attempt to reduce the concentration of stresses which could possibly have led to separation of the column from the base.

(c) Pull-Out Test

A 600mm x 395mm sheet of perspex resting on the tank base was held rigidly in position during sand compaction. The compaction process was similar to the other tests, except that the grid of compactor positions was modified to cater for the different shaped insert. The sheet was buried to the full depth of the tank (ie, 345mm).

A pull rod was located through the centre of an overhead hydraulic jack and was connected to a 2kN tension load cell which was in turn connected to the apex of a lifting hanger attached to the top of the perspex sheet. It was intended that this method would cause the sheet to be lifted at a uniform rate across the entire width. The electric pump was used to expand the hydraulic jack against a
rigid frame above the tank and so pull the sheet out of the sand. The load cell and the pair of LVDTs that were used to measure the upward movement were all connected to the datalogger and were calibrated in a similar manner to those used for the pull-through tests. Readings were recorded continuously at one second intervals as the sheet was extracted from the sand, initially at a rate of 0.815 mm/min but increasing gradually to give a 60 mm displacement in approximately 10 minutes. A slow initial rate of extraction was necessary to provide sufficient data during the initial first 3 mm of displacement. Tests were performed for both smooth and sand coated perspex.

(d) Shear Box Test

A square piece of perspex was cut to fit exactly into the lower half of the shear box. Its surface was mounted flush with the top of the box. The upper section of the box was secured to the lower section and the sand was poured in and compacted to an average density of 1673 kg/m³. The box was then assembled into the shear box apparatus and tested at a rate of 0.614 mm/minute in the standard manner.

Shear box tests were performed for both smooth perspex and sand coated perspex and also for the sand alone.

3.3 Summary

This chapter has described the experimental details relating to the full size monitoring of the Wisley abutments and also the model tests performed in the laboratory. As such, the methods of instrumentation have been outlined and the test procedures have been explained. Furthermore, the adopted methods of soil density determination both in the laboratory and on site have been explained and the test results have been presented. In addition, the method of calibration of the earth pressure cells has been described and the findings have been reported.

The following chapter presents the results from all the experimental investigations and discusses their implications on the methods of estimating the earth pressures exerted on spillthrough abutments.
Plate 3.1 Vibrating wire pressure cell (left) and pneumatic pressure cell (right)

Plate 3.2 Vibrating wire pressure cell mounted flush into concrete
Plate 3.3 Installation of Maihak embedment pressure cell

Plate 3.4 Pressure cells positioned on concrete disc within calibration chamber
Plate 3.5  Perspex columns used for pull-through tests
Plate 3.6 Perspex template for positioning ball bearings on surface of sand

Plate 3.7 Perspex column used for column rotation tests
Plate 3.8 Apparatus for pull-out test
Plate 3.9 Hyperdermic needles used for resin impregnation of sand

Plate 3.10 Apparatus for resin impregnation density tests
Plate 3.11  Resin/sand samples after excavation

Plate 3.12  Cross-section of a resin/sand sample
Plate 3.13  Typical photograph of array of ball bearings
Figure 3.1 Plan of road interchange at Wisley
Figure 3.2 Elevation of mainbridge at Wisley

Figure 3.3 Elevation of A-side abutment at Wisley
Vibrating wire strain gauges (Concrete strains)
Vibrating wire strain gauges (Steel strains)
Vibrating wire embedment earth pressure cells
Vibrating wire boundary earth pressure cells
Pneumatic boundary earth pressure cells

Figure 3.4 Abutment instrumentation
### Figure 3.5 Grading curves for backfill around columns

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<tr>
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<th>SEDIMENTATION</th>
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<tr>
<td>1</td>
<td>Wisley 'A' Abutment</td>
<td>Medium / Fine Sand</td>
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#### PARTICLE SIZE DISTRIBUTION

- **Clay**
- **Silt**
- **Sand**
- **Gravel**
- **Cobble**
- **Boulders**
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### B.S. SIEVES (Metric range)

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### Figure 3.6
Grading curves for backfill behind capping beams.
Figure 3.7 Diagram of apparatus for pressure cell calibration tests
Figure 3.8 Pressure cell calibration curves
Figure 3.9 Soil compression characteristics
Figure 3.10 Pressure cell response for interrupted loading cycles

2.5mm vibrating wire cell
Soil depth = 295mm
Dry density = 1575 kg/m³
Figure 3.11 Dry densities of backfill

- (a) Around columns
- (b) Behind capping beam
Figure 3.12 Apparatus for pull-through tests
12.5 mm thick steel tank lined with 6mm plate glass

Figure 3.13 Apparatus for column rotation tests
Figure 3.14 Results of control tests for resin impregnation technique
Figure 3.15 Grid of compactor positions

Figure 3.16 Position of resin balls within compacted layers of sand
Figure 3.17  Densities of sand for equal compaction per layer
Figure 3.18 Densities of sand for increased compaction to upper layers
CHAPTER 4

RESULTS AND DISCUSSION

This chapter presents and discusses the results obtained from the full size investigations concerning the performance of spillthrough abutments. The results from the monitoring of the full size Wisley abutments are discussed in relation to the effects of each of the major construction events and the long term effects. The model test results are discussed in relation to particular aspects of the performance of spillthrough abutments. Finally, the implications of the major findings from the experimental studies are considered with regards to improving the existing methods of designing spillthrough abutments.

4.1 Full Size Spillthrough Abutments

4.1.1 Introduction

During the period of construction of the two spillthrough abutments at Wisley, frequent readings were recorded by the University of Surrey, so as to provide adequate data for assessing the effects of each major construction event. After completion, the readings were taken less frequently but were sufficient to indicate the effects of traffic loading and seasonal temperature fluctuations. The pressures on the pneumatic cells were read separately by TRRL staff at a frequency comparable to that achieved by the University of Surrey. The TRRL data was later presented to the University for further processing and where appropriate has been used to provide a useful comparison with the data obtained from the vibrating wire cells.

The measured lateral earth pressures that are presented in this section have all been determined from the raw data by applying the Cell Action Factors that were obtained from the calibration tests performed in the laboratory. As such, they represent the best possible estimate of the actual pressures exerted by the soil on the structure.
Immediately prior to a cell being covered by soil, a datum reading was recorded to indicate the specific cell response representing a zero pressure. In order to prevent unnecessary complications during the period of taking a set of readings, it was decided to record values from all cells regardless of whether they were covered or not. As a result, a large number of readings were taken for some cells long before being covered by soil. It was expected that the readings for such cells should change very little during this period of dormancy. However, this was not the case because the readings for some cells were found to fluctuate considerably and clearly, the cause was not due to earth pressure. Temperature changes of the cells were thought to be the most likely cause but tests on a cell with a 2mm thick diaphragm at 0°C and 20°C under laboratory conditions indicated only a negligible variation in cell response. Furthermore, an attempt to relate the cell response to the measured temperature of the uncovered cells in the structure revealed no significant correlation. A typical plot of temperature versus cell response for a cell at the top rear face of the E2 column is shown in Figure 4.1. Each cell was surrounded by a compressible layer of neoprene rubber to debond it from the surrounding concrete. Only the rear face of the cell was rigidly in contact with the concrete, as this was necessary to prevent any bodily cell movement due to a normal stress exerted on the structure. At first, these conditions would seem to be adequate to prevent in-plane stresses within the concrete from being transmitted across the cell. However, closer inspection of the cells revealed a series of screws whose heads were projecting from the rear face of the backplate, thus enabling the stresses in the concrete to be transmitted directly to the rigid body of the pressure cell. These can been seen in the cross-section shown in Figure 4.2 and also in Plate 3.1. The lack of correlation between cell response and cell temperature was therefore due to a temperature gradient within the capping beam, which caused the recorded cell temperatures to be unrepresentative of the temperature of the concrete as a whole. The effects of temperature were excentuated for the rear face of the E abutment capping beam because it was coated in black bitumen paint and faced southwards, thus causing a large amount of heat absorption. As the level of backfill was raised, the temperatures in the capping
beam became more stable and the fluctuations in cell response were thereafter considered to be due to the effects of construction alone.

A set of pressure response versus time plots for all the vibrating wire cells installed in the Wisley abutments are shown in Appendix E, and it can clearly be seen that certain construction events have had a marked effect on the cell response. A general inspection of these plots has revealed the most influential construction events, which are as follows:

(i) Backfill to top of columns  
(ii) Capping beam pour  
(iii) East deck slab pour  
(iv) West deck slab pour  
(v) Backfilling and road surfacing behind capping beam.

A detailed description of the effects of each construction event is presented in the following sections along with a brief outline of the long term effects that have so far been observed.

4.1.2 Backfill To Top Of Columns

As the level of backfill was raised around the columns, frequent pressure cell readings were recorded so as to relate the effect of increased overburden to the change in soil pressures. As can be seen from Figure 4.3, the majority of the cells mounted into the upper surface of the base recorded vertical pressures in excess of those calculated from the measured bulk density of the soil. This appears to be partly due to an immediate overestimate of the soil self-weight after placing the first 1m of backfill, such that the vibrating wire cells at the A abutment recorded pressures up to twice the expected pressure and the pneumatic cells by up to four times. The initial vertical base pressures for the E abutment, however, were closer to the expected values. The discrepancy between the two abutments was probably due to varying amounts of compaction being performed. The consistently high initial vertical pressures at the A abutment were caused by the intensive compaction of the first few layers of backfill, which would have
created very high stresses in the vicinity of the cells. This would have caused the vibrating wire cells to overread due to locked-in deformations of the diaphragms caused by the interlocking of soil particles above, similar to that observed during the cell calibration tests. The pneumatic cells may have tended to overread as a result of being a high spot on the base, thus causing a stress concentration above the cell face. It was intended to compensate for this by applying the Cell Action Factors determined from the cell calibration tests, but this may have been inadequate in this instance to allow for the relatively large stresses created by the compaction equipment used on site. Apart from the initially high vertical pressures, Figure 4.3 indicates that the further increases in vertical pressures due to continued backfilling exceeded those predicted by the measured soil bulk density. This has been explained by the ability of the backfill to arch across a compressible wedge of soil that was backfilled between the base slab and the existing ground, thus causing the vertical overburden to be concentrated onto the more rigid base slab. Furthermore, the friction between the columns and the soil could have caused additional residual vertical stresses. These actions are further substantiated by the fact that the cells placed between columns recorded pressures somewhat closer to the expected values. These cells were over 1.5m from the columns, thus reducing the effects of wall friction and they were also farther away from the edges of the base slab. A reduction in soil density between the columns cannot be assumed to be the cause of the lower pressures because the column spacings were sufficient for the normal type of compaction plant to be used.

After the backfill had been raised to the top of the columns the vibrating wire base cells, Maihak embedment cells and pneumatic cells, all placed 1m directly behind the columns, indicated a vertical pressure profile only slightly greater than that predicted by the soil self-weight, (Figures 4.4 and 4.5). However, the pneumatic cells placed 1m away from the rear of the abutment and part way between the line of two adjacent columns (Figure 4.4) indicated vertical pressures up to 70% greater than those predicted. It is possible that these high readings may have been due to a difference in Cell Action Factor of a pneumatic cell embedded in soil to that
mounted on a structure boundary. Unfortunately, the former conditions were not investigated as part of the cell calibration study.

The lateral pressures acting on the faces of the columns were recorded when the backfill was completed to the top of the columns. If as at Wisley, the backfill is raised evenly around the columns, then a typical design would assume at-rest lateral earth pressures to act on the faces of the columns. The measured lateral pressures on the front and back faces are shown in Figures 4.6 to 4.8. It is immediately apparent that the lateral pressures exceeded the calculated values of at-rest pressure based on a design assumption of 9.4H. At-rest pressures have been compared with the measured pressures because the level of backfill was raised at a constant rate on all sides of the columns, and this theoretically suggests that the columns would not be subjected to significant lateral movements. The design assumption of an at-rest pressure distribution of 9.4H is commonly used in design offices and is derived from the assumptions that the backfill has a bulk density of 19kN/m³ and that the earth pressure against a non-yielding structure is equal to twice the value of active pressure based on an earth pressure coefficient, K_a, equal to 0.25. The pressure profiles also indicate that the pressures do not increase linearly with depth but instead show a roughly uniform to parabolic pressure distribution with values ranging between 30kPa and 55kPa on the rear face and between 20kPa and 50kPa on the front face. The measured pressure distributions were greater than those predicted by an at-rest design approach of 9.4H because of the effects of soil compaction around the columns. The reduction in pressure towards the at-rest value near the base of the columns may be partly due to the friction developed between the upper surface of the base slab and the soil. Slightly higher pressures were recorded on the rear face because of the reduced degree of restraint at the front caused by the 1:2 slope of the embankment. However, as can be seen from Figures 4.9 to 4.12 the net pressures on the rear of the columns were generally less than a typical active design value of 5H kN/m². The forward movements of the top of the columns during the period of compaction, as recorded by precise surveying were up to 2.8mm and
2.0mm for the A and E abutments respectively. The pressures on the column sides before the capping beam pour are shown in Figures 4.13 and 4.14, which again indicate pressures greater than at-rest values and which were found to vary from 25kPa to 50kPa. The only exception was cell 105 on column A2 which recorded a pressure of 120kPa but this was probably due to a malfunction of the cell caused by a large stone pressing directly against the face of the diaphragm. Although the lateral earth pressures during this stage of construction were greater than predicted, the similar profiles recorded on each pair of opposite faces indicate that the structure was reasonably well balanced. This was confirmed by the negligible concrete strains recorded at the root of the columns which indicated that the earth pressures were not causing them to bend significantly.

Precise surveying during the period of backfilling indicated vertical settlements of 5mm and 4mm for the A and E abutments respectively. This was due to the compression of the underlying strata caused by the surcharge loading from 5m of backfill.

4.1.3 Capping Beam Pour

Although the pours for the capping beams at the A and E abutments were separated by a period of 42 days, the instrumentation indicated very similar responses on each occasion. The lateral pressures on the rear and front faces of the columns were found to remain unaltered whilst the pressures on the sides experienced major variations. This was to be expected because of the transverse nature of the capping beams. The side pressures were found to vary considerably depending on the time after pouring the concrete. After a period of about two days the recorded values indicated an increase in soil pressure on the west face of each column accompanied by a decrease on the east face, (see Figures 4.13 and 4.14). The pressure variations were more noticeable for the outer (line 2) columns than for the inner (line 3) columns. At the top of the west face of the line 2 and 3 columns the pressures increased by about 35kPa and 20kPa producing maximum recorded lateral pressures of 75kPa and 60kPa respectively. Correspondingly, the
pressure at the top of the east face of the columns dropped to below 15kPa with the exception of the E3 column. The changes in pressure lower down were generally not as significant and only amounted to a change of not more than 15kPa with the exception of the A2 column.

This pressure distribution was only temporary and after a further period of five days the pressure profiles on the sides were almost reversed. The lateral pressures on the east face of all columns increased beyond the values recorded prior to pouring the concrete. As expected, a corresponding drop below previous values was recorded on the west face for the line 2 columns. However, the line 3 columns did not experience any pressure relief on the west face and instead the pressures showed a further increase.

The reason for the large changes in soil pressure was due to the lateral movements at the tops of the columns caused by the expansion and subsequent shrinkage of the capping beam. For both abutments, the eastern capping beam was poured and the formwork was stripped before pouring the western capping beam. Although a sheet of compressible material was placed between the beams to form an expansion joint, it is clear that the eastern beam provided a sufficient propping force to cause the expansion of the western beam to push the tops of the columns westward. Consequently, the abutments experienced a plane frame type of action. Under such conditions the lateral deflection of the top of the line 2 column could be as much as 4.0mm, if it is assumed that the coefficient of expansion of concrete equals $11\mu s/°C$ and the temperature change of the concrete during hydration is about 40°C. Clearly, the columns were fixed at base level, but the strain gauges within the columns indicated a point of contraflexure towards the top of the column, thus suggesting that a degree of fixity must have also occurred at the top. The subsequent shrinkage that followed the initial period of expansion was uninfluenced by the previously poured eastern capping beam because the shrinkage caused the capping beams to separate along the expansion joint. The increase in pressure on both sides of the central (line 3) columns shows that the outer (line 2
and line 4) columns were pulled inwards. Therefore, the frame action, in this case, involved a symmetrical contraction of the capping beam about the central (line 3) column with all columns being fixed at the top and bottom.

It is evident that the high lateral pressures that were exerted by the soil must have provided a substantial lateral support to the columns during the period of expansion and shrinkage of the capping beam. This was reflected by the resultant strains measured in the columns which were insufficient to induce cracking of the concrete, and were less than those calculated from a plane frame analysis for an expansion of the capping beam, (Lindsell (1984)).

4.1.4 East Deck Pour

The continuous 76m long voided bridge deck slab was constructed in two halves, with the east deck being poured six weeks before the adjacent west deck. The plywood soffit formwork to the deck slab was supported on falsework across the entire span of the bridge. Placement of the in-situ concrete commenced at the centre of the deck and proceeded outwards on two fronts simultaneously towards the abutments. During this stage of construction, the embankment only existed to a height level with the top of the abutment columns and as such was expected to provide only minimal resistance to any lateral loading. The expansion joint separating the two halves of each capping beam was expected to prevent the majority of the loading effects of the east section of an abutment from being transmitted to the west section. Even so, regular readings from the instruments installed in the west sections have clearly indicated that a significant amount of interaction between the two halves did in fact occur due to the connectivity provided by the common base slab.

Precise surveying of the east wing walls of both abutments, before and 24 hours after the deck pour, indicated movements at the wing wall tips of 8mm in a horizontal backwards direction and 10mm and 9mm in a vertical downwards direction for the A and E abutments.
respectively. The horizontal and vertical displacements at four and three other points across the face of the wing walls of the A and E abutments respectively, were also measured in a similar manner. The recorded displacements were primarily due to a rotation of the abutments as a result of the lateral loading caused by the expansion of the deck. The dead-weight of the deck slab should have been supported by the falsework at this stage, and was therefore considered unlikely to have contributed significantly to the measured vertical displacements. The position of the centre of rotation can therefore be estimated as the intersection of lines perpendicular to the direction of movement of each reference point on a wing wall, as shown in Figure 4.15. These geometric constructions do not define a unique position for the centre of rotation but they do indicate that it occurred between 1m and 3m above the top of the base slab. This is contrary to the common design approaches in which it is often assumed that rotation occurs about a point at the junction between the columns and the base slab.

The lateral thrust which caused the backwards rotation of the abutment was due to the expansion of the concrete deck slab during the first 48 hours after casting. Although the concrete deck did not come directly into contact with the abutments, it is clear that the loads must have been transmitted via the formwork. This occurred because the formwork was not designed to permit any relative movement between the abutment and the warm concrete of the recently poured deck slab. The formwork detail is shown in Figure 4.16, and it can be seen that the loading from the deck slab was transmitted to the abutment via the polystyrene packing material along the line of the expansion joint and the timber packing material which was used to prop the bearing downstand shutters off of the abutment. Furthermore, the formwork design did not allow any movement of the bearings and consequently, the concrete bearing downstands must have been subjected to considerable distortion. This effect was observed more clearly after pouring the western half of the deck slab and is discussed in more detail in the following section. The lack of freedom of the sliding bearings was illustrated by the recovery of over 80% of the horizontal displacement at each abutment after a period of only five days.
The lateral pressure profile on the instrumented line 2 and line 3 columns, recorded before and 48 hours after the east deck pour, are shown in Figures 4.18 and 4.19. Unfortunately, only the pressures recorded by the vibrating wire cells are shown due to the lack of data recorded from the pneumatic cells during this period of construction. It can be seen that the magnitude of the pressure changes was not very large and was generally of the order of 10kPa to 15kPa. However, certain trends in the form of the pressure changes can be deduced from the plotted profiles. It was observed that the east deck pour caused a general decrease in pressure down the rear face of the line 2 columns accompanied by a smaller yet noticeable increase on the front face. In contrast, the line 3 columns experienced the opposite effect, with an increase on the rear face and a decrease on the front face. The corresponding changes in side pressures on the columns, as shown in Figures 4.20 and 4.21, indicate a general decrease on both faces of the line 3 columns but indicate an increase on the west face and a slight decrease on the east face of the line 2 columns.

Initially, it was felt that a proportion of the large rotational movement of the east section of an abutment may have been transferred to the west section as a result of the torsional resistance of the common base slab. If this was the case, then the pressures on the instrumented columns would be expected to increase at the top of the rear face and decrease towards the base, with the reverse effect on the front face. However, this type of response does not explain the measured variation in soil pressures. Consideration of the measured pressure changes therefore led to the deduction that a more significant mechanism must have also occurred such that the abutment tended to rotate in plan about a vertical axis located between the line 2 and line 3 columns, as illustrated in Figure 4.17. This mechanism would result in a backwards movement of the line 3 columns and a forwards movement of the line 2 columns, thus explaining the observed changes in earth pressures.
4.1.5 West Deck Pour

After removal of the falsework from beneath the east deck slab, the full dead-weight of the deck was transmitted to the bearings. This produced a clamping action on the eastern section of the abutments and therefore, greater torsional resistance from the base slab was expected than had been experienced during the previous east deck pour. The west deck pour provided an ideal opportunity to investigate more closely some of the effects that had been observed from the earlier deck pour. The construction procedure was carried out in an identical manner, with the concrete being poured in two directions starting from the centre of the bridge.

The development of the lateral loading applied to the abutments via the formwork was investigated by recording the displacements of reference points on the abutment wingwalls and the formwork, using a precise surveying technique. The displacements of the reference points were recorded after the concrete had been placed over the two central spans and before it reached the abutments. The formwork at the E abutment was observed to move 2.5mm horizontally in the direction of the abutment, which resulted in a 1.25mm horizontal deflection of the abutment itself. The corresponding horizontal displacement of the top of the A abutment was 0.5mm. The formwork detail (Figure 4.16) was identical to that used during the earlier east deck pour, thus indicating that the differing horizontal displacements of the deck slab and the abutments must have been caused by the compression of the polystyrene packing material which separated them. The horizontal movement of the deck slab formwork was caused by the friction generated between the expanding concrete and the 20mm thick plywood soffit shutters. The soffit shutters were prevented from buckling by the supporting falsework and were therefore able to withstand large in-plane stresses. The shutters around the bearing downstands were connected to the soffit shutters of the deck slab and were consequently subjected to similar horizontal displacements. However, the lower edge of the downstand shutters were propped against the abutment with timber packing. Unfortunately, this timber packing was less compressible than the polystyrene packing.
that was used to form the expansion joint. Consequently, the lateral displacement of the lower edge of the downstand shutters was somewhat less than that of the deck soffit shutters, thus resulting in a distortion of the downstand formwork, as shown in Plate 4.1. This therefore indicates that high shearing stresses were produced in the bearing downstands, which must have resulted in a misalignment of the bearing plates.

The deflections of the abutments were again monitored 24 hours after casting the deck and indicated that the tip of the west wing wall of the E abutment had moved 6mm backwards and had dropped vertically by 8mm. The corresponding displacements measured for the A abutment continued to be less substantial, with a 4mm backwards deflection and a drop of 5mm at the tip. Furthermore, the wing walls were observed to have deflected inwards at the top, by 7mm and 5mm for the A and E abutments respectively. The restraint offered by the east deck to the abutment was therefore reflected by the smaller horizontal displacements that were recorded as a result of pouring the second deck slab. The clamping action was effective primarily due to the torsional resistance of the base slab. However, a uniform torsional loading along the western section of the base slab would allow larger rotational displacements at greater distances from the clamped eastern section. Therefore, the column displacements experienced near the centre line of the bridge would have been somewhat less than those measured at the edge. This agrees with the inward deflection of the wing walls that occurred due to horizontal bending of the capping beam. Furthermore, the measured changes in pressure and strain for the outer (line 2) columns were observed to be more substantial than for the inner (line 3) columns.

Regular readings after the pour from the inclinometer tube located within the E3 column indicated that the maximum deflection of the column occurred 36 hours after casting the deck. The abutment displacements caused during this period are shown in Figure 4.22, and it can be seen that a relative deflection of about 5.5mm occurred between the top and bottom of the E abutment. This value was interpreted after discarding the lowest reading because of
the undesirable effects of debris at the base of the tube, which would have affected the positioning of the inclinometer probe. If this relative deflection is superimposed on the absolute deflection of about 4.5mm of the top of the capping beam, as recorded by the precise surveying technique, it indicates that the base slab kicked forwards (ie towards the deck) by about 1.0mm. A similar amount of base movement can be inferred after locating the approximate position of the centre of rotation from the measured displacement vectors of the reference points on the surface of the wing walls, as shown in Figure 4.23. In both cases, the centre of rotation is revealed to be between 1m and 2m above the upper surface of the base slab. This shows good agreement with the position of the centre of rotation that was observed after the earlier east deck pour.

Inspection of the changes in soil pressures after the pour indicated that the maximum effects occurred 24 to 36 hours after casting. The pressure profiles on the front and rear faces, 24 hours and 8 days after the pour are compared with the initial pressures in Figures 4.24 to 4.26, and these include data from both the vibrating wire cells and the pneumatic cells. Where both sets of data are available for a particular column, a band of pressure containing the true pressure is inferred.

The pneumatic cells on the A6 column have indicated that the pouring of the west deck had a marked effect on the eastern section of the abutment, despite being clamped by the deck slab that it supports. This again confirms the transfer of load across the two adjacent sections of the abutment via the base slab.

Very large increases in pressure were observed after 24 hours at the top of the rear face of the columns and a slight decrease was observed towards the base. The corresponding pressures on the front faces consistently indicated changes in the opposite sense with a reduction almost to zero at the top and a slight increase towards the base. The net pressure profiles on the columns, shown in Figures 4.9 to 4.12, clearly indicate that the tops of the columns were pushed backwards, (ie away from the bridge), causing a large pressure increase at the top of the rear face of the columns.
The maximum pressure at the top of the A2 column was measured as 124kPa and 117kPa for the vibrating wire and pneumatic cells respectively. The corresponding maximum pressure at the top of the A3 column was 55kPa and 108kPa for the two types of cell. Similarly for the E2 and E3 abutments the maximum lateral pressures at the tops, as recorded by the vibrating wire cells, were 97kPa and 62kPa respectively. These results clearly indicate that the pressures at the outer (line 2) columns were generally higher than for the inner (line 3) columns, which confirms that the outer columns were subjected to larger horizontal displacements. This is in agreement with the inwards deflections of the wing walls that led to the earlier conclusion that the capping beam was subjected to horizontal bending.

A further general observation was that the pressures exerted on the E abutment were generally smaller than those on the A abutment. This does not agree with the response that would be expected, based on the observation that the E abutment experienced a 50% larger horizontal displacement than the A abutment. However, the measured lateral displacements were measured at the top of the wing walls and not at the top of the columns. Therefore, assuming that a column rotates about a point 1m to 2m above the base, then the lateral displacement at the top of the column would be only a proportion of that measured at the top of the wing wall. In particular, if a centre of rotation is assumed to be 1m above the base, the 4mm and 6mm deflections at the top of the A and E abutments only represent deflections at the top of the columns of 2.3mm and 3.0mm respectively. The lesser relative difference at column level is due to the deeper capping beam and shorter columns of the E abutment, as compared to the A abutment. This however, still fails to explain the larger pressures exerted on the A abutment and therefore, the most probable reason for the discrepancy is that a different soil stiffness was created by a variation in the degree of compaction achieved at each abutment.

An alternative means of determining the position of the centre of rotation of the abutments is to study the pressure plots (Figures 4.24 to 4.26) and define the height at which the interpolated
pressures remain unchanged on a particular face. Using this method, the centre of rotation for the A2, A3, E2 and E3 columns was consistently between 0.75m and 1.25m above the top of the base slab. These values compare favourably with a position of 1m to 2m above the base as determined from a projection of the displacement vectors measured on the wing walls (Figure 4.23) and the inclinometer results (Figure 4.22).

The changes in pressure on the sides of the columns due to the west deck pour were found to be consistent for each of the abutments, as shown in Figures 4.27 and 4.28. The increase in pressure on the east side of the columns in the A abutment were accompanied by a slight decrease on the west face. The exact reverse was observed for the E abutment, with an increase on the west face and a decrease on the east face. This type of response can be explained by the fact that the abutments were parallel to each other, with the deck being constructed at a skew angle of 3° between them. As a result, the expansion of the deck slab caused a transverse force at each abutment. The direction of the transverse force on the abutments was consistent with the observed increases in pressure on the column sides. This mode of deformation was further confirmed by the results of the inclinometer tube in the E3 column. If the base is assumed to be restrained from transverse movement then the recorded deflections from the inclinometer tube shows that the top of the E abutment moved approximately 1.75mm in a westward direction.

The abutment deflections were observed to recover considerably after the initial period of expansion of the deck slab. The recovery was similar to that previously experienced after the east deck pour. After a period of nine days, the top of the E abutment still had a horizontal displacement of 20% of the maximum, whereas the top of the A abutment had continued to fully recover and then subsequently showed a 15% increase in the maximum horizontal displacement in the opposite direction. This clearly indicates that the bridge was not acting symmetrically about the central columns. However, a more important implication is that the alignment of the bearing
plates must have been affected by the expansion of the concrete
deck slab during the first 48 hours after casting the deck, thus
causing an increase in the value of the bearing friction. In
conclusion, it is evident that the large frictional forces developed
between the bearing plates were sufficient to cause the top of the
abutments to be dragged forwards (ie towards the deck) as the deck
slab cooled and contracted.

The lateral movement of the top of the E abutment, nine days after
casting the deck, has been superimposed on the inclinometer data
that was obtained after a similar time period, as shown in Figure
4.22. It can be seen that the top of the columns tended to push
forwards but the capping beam was still tilting backwards. This
implies that a degree of permanent deformation must have occurred
along the construction joint which joins the columns to the capping
beam. The forwards deflection of the top of the columns was
reflected by the changes in soil pressure over the same period, as
shown in Figures 4.24 to 4.26. The pressure at the top of the rear
face dropped to almost zero, whereas the pressures generally
increased slightly on the opposite face. At base level, the pressures
generally increased at the rear yet again indicating that the
abutment was rotating about a point above the level of the base
slab.

The results obtained from the two deck pours clearly emphasise the
role of the compacted backfill around the columns in providing
lateral resistance to the lateral load exerted by the expansion of the
decks during the first 36 hours after casting. In addition, the
centre of rotation has been shown to occur at a considerable
distance above the base and not at base level as is generally
assumed for design purposes.

4.1.6 Backfilling And Road Surfacing Behind The Capping Beam

The backfilling behind the capping beam was carried out
intermittently over a period of six winter months from October 1982
to April 1983. The major delays experienced during this period were due firstly, to the lack of availability of suitable fill material and secondly, to the adverse wet weather conditions.

The backfilling operation was not generally carried out simultaneously for both abutments. Consequently, when work stopped in November 1982, the level of fill behind the A abutment was 0.9m below the top of the capping beam but dropped to 1.3m immediately adjacent to the rear of the beam. In contrast, the overall level of the backfill behind the E abutment was 2.1m below the top of the beam. This state of backfill remained until February 1983 when backfill recommenced, except for a ramp that was constructed behind the A abutment immediately above the line 3 column. This ramp was regularly used by plant to travel up onto the bridge deck during the winter months. The fill behind both abutments was raised to 0.8m below the top of the capping beam at both abutments by the end of February. However, a subsequent rejection of the selected granular fill material used behind the E abutment led to the top 1m of this material having to be excavated, thus leaving a trench along the rear of the abutment, as shown in Plate 4.2. The trench was eventually filled with an acceptable material at the beginning of April 1983. The construction of the road followed in the subsequent three weeks and the surfacing was completed on 14 May 1983.

The lateral pressures generated after the placement of the capping beam backfill and road completion are shown for the front and rear faces in Figures 4.29 to 4.32. Only the data from the vibrating wire cells is shown because the pneumatic cells were positioned to record the pressures around selected columns. The degree of influence of the sand buffers against the pressure cells in the capping beam is uncertain and it is therefore appreciated that the readings may be slightly in error. However, the observed trends are still considered to give a good representation of the true effects.

Initially, as the first 1m of backfill was placed and compacted behind the capping beam at the A abutment, the soil at the front was visually observed to bulge between the columns. However,
further backfilling did not continue to push soil between the columns and instead the capping beam tended to act as a retaining wall and consequently started to move forwards.

It would be expected that the centre of rotation should occur at a height similar to that determined as a result of the deck pours because the height of backfill in front of the columns had not changed. The lateral pressures exerted on the rear face of the capping beam could reasonably be considered as an equivalent horizontal point load, similar to the horizontal reaction previously exerted by the expansion of the deck slab after casting. Therefore, the conditions were similar except that rotations in this case were in the opposite direction to those which had occurred during the deck pours. However, it must be noted that the compaction of the backfill behind the capping beam tended to cause additional compaction of the soil around the columns, particularly in the early stages.

The deflections of the E abutment during the period of backfilling and road construction were recorded by the inclinometer in the E3 column. If a centre of rotation is assumed to be at 1.5m above the base, (similar to that measured during the west deck pour), then the forward displacements of the top of the abutment after completion of the backfilling and the road surfacing are 11mm and 14mm respectively, as shown in Figure 4.33. It is therefore evident that the lateral pressures exerted against the rear of the capping beam were sufficient to overcome the large value of bearing friction and cause the top of the abutment to slide forwards beneath the deck slab and partly close the expansion joint. An inspection of the expansion joint revealed that a considerable amount of polystyrene packing material had been inadvertently left in place. It is therefore likely that the forwards movement of the top of the abutment may have caused it to come into contact with the end of the deck slab via the remaining packing material and provide an additional propping reaction to that created by the high bearing friction.
As a result of the rotation, the pressures at the top of the front faces of the columns steadily increased. However, the pressures towards the base of the front face were seen to decrease, thus confirming that the abutment must have rotated about a point some distance above the base. The pressures were generally observed to increase slightly at the top of the rear face of the column as a result of the capping beam backfill. The pressures towards the base of the rear face also tended to increase, as expected, due to the combined effects of the compaction of the overlying backfill and the tendency of the base to kick backwards.

The pressures generated against the rear of the capping beam as a result of the capping beam backfill were consistently greater than the active pressures based on the value of $K_a = 0.271$ that was used in the original design of the abutments. In particular, the effect of backfilling the trench behind the E abutment caused a type of wedging action that resulted in very high pressures. The recorded pressure of 60kPa, acting on cells 145 and 162, was equivalent to 1.5 times the overburden pressure, representing a pressure over five times greater than the assumed active value. The wedging action behind the E abutment also caused the strains at the column roots to be considerably larger than those for the A abutment, even though the columns were 800mm shorter.

The construction of the road subsequently led to further increases in the pressure exerted against the capping beam. The maximum recorded pressure against the capping beam was 45kPa for the A abutment and 125kPa for the E abutment. These high pressures were caused as a result of the intensive compaction that was carried out with a Stothert and Pitt Vibroll T182A roller only 500mm from the rear face of the capping beam. It is also possible that the wing walls may have contributed in part to the large lateral pressures due to their confining action. Similar large lateral pressures after compaction of the backfill behind several rigid bridge abutments were observed by Jones and Sims (1975).

Further increases in pressure were observed at the top of the front face of the columns due to the road construction. These were
accompanied by a decrease in pressure, almost to zero, towards the base of the front face.

The resultant pressure profiles after the completion of the road construction are shown in Figures 4.9 and 4.12. These show the large compaction pressures behind the capping beam and the rotational effects of the abutment due to the increased pressures at the top of the front face and at the base of the rear face. Clearly, the resultant pressure distribution in no way resembles the active pressure distribution on the rear of the abutment that was assumed for the original design of the structure.

The placing of the capping beam backfill and construction of the road were also observed to have a slight influence on the pressures acting on the side faces of the columns, as shown in Figures 4.34 and 4.35. Without exception, the pressure exerted at the top of the side faces was seen to decrease. This was probably due to the relief of the residual stresses caused by the flow of the soil between the columns during the initial stages of compaction. Conversely, the pressures towards the base were seen to generally increase slightly as a result of the compaction applied to the backfill behind the capping beam.

The vertical pressures, as measured by the Maihak embedment cells and the vibrating wire base cells, at a distance of 1m from the rear of the abutments, are shown in Figures 4.36 and 4.37. The pressures measured after the completion of the capping beam backfill and the road surfacing show good agreement with those predicted from the bulk density of the backfill.

4.1.7 Long Term Effects

The long term effects are being monitored by Dr P Lindsell as an extension of the original contract awarded by the Department of Transport. During the first two years since the bridge has been open to traffic, all the vibrating wire pressures cells have continued to perform satisfactorily, therefore making such long term
monitoring perfectly feasible. The data from the pneumatic cells is not considered to be as reliable over such a long period of time because several of the cells have tended to malfunction as a result of damage caused by accidental over-inflation of the diaphragms.

Considerable changes in soil pressures have already been observed during the first two years of service, primarily due to the effects of traffic loading and temperature fluctuations. The variation of pressure with time for each of the vibrating wire pressure cells is illustrated by the figures in Appendix E. A general inspection of these figures tends to reveal a cyclic pressure response beginning to occur during the first two years, which corresponds quite closely to the seasonal temperature variations. This indicates that the soil pressures exerted on the abutment are partly dependent on the ambient temperature.

The lateral pressure profiles on the front and back faces of the abutments for conditions of high and low ambient temperatures and after 12 months service, are compared with those immediately after construction in Figures 4.29 to 4.32. The corresponding net pressure profiles are shown in Figures 4.9 to 4.12. It can be seen that during the warm summer months, the lateral pressures acting on the rear face of the capping beam generally increased well beyond those at the end of the construction. This was probably due to the expansion of the concrete deck slab which caused the abutments to be forced backwards at the top (ie towards the backfill) as a result of the large frictional forces developed at the damaged bearings. However, it is quite possible that an additional horizontal loading was applied to the abutment as a result of the deck expansion and the consequent compression of some packing material which had been left in the expansion joint. The contraction of the deck slab during the winter months must have caused the abutment to be dragged forwards as a result of the friction developed at the bearings and thereby relieve the lateral pressures at the rear of the capping beam. The extremely low pressures recorded towards the top of the capping beam show that the movements were sufficient to create near active conditions. Similar changes in pressures due to the expansion and contraction of a deck slab have been observed by Broms and Ingelson (1971, 1972).
Furthermore, during the summer months, small increases in lateral pressure were also recorded at the top of the front face of the columns. The cause of this increase in pressure is unclear because it does not correspond with the simultaneous backwards rotation of the top of the abutment. However, one possible explanation is that the backfill material within the sloping embankment at the front of the columns may have expanded laterally due to a rise in the ambient temperature. Unfortunately, there is no further evidence to confirm that this did in fact occur, although it is interesting to note that the pressure changes between hot and cold conditions were larger towards the surface of the backfill than at greater depths. The grounds for this explanation are based on the work of Coyle and Bartoskewitz (1977), who found that the earth pressures against an unpropped retaining wall were affected by temperature fluctuations of the backfill.

The large pressures exerted on the rear of the capping beam will undoubtedly have also been partly caused by the additional compaction due to traffic loading. For the case of bridge abutments, it is not unusual for the effects of deck expansion and contraction and of compaction due to traffic loading to occur simultaneously. Thus, when an abutment moves away from a mass of soil during the winter, the traffic tends to re-compact the soil at the rear and fill any voids that may have been created between the wall and the soil. Consequently, the expansion of the deck during the following summer causes the pressures on the rear of the capping beam to increase beyond the maximum values recorded for the previous year because greater resistance is mobilised from the re-compacted soil. The effects of compaction of the soil due to traffic loading have been reflected by the recorded net forward movements of the A and E abutments after one year's service of 1mm and 2mm respectively. Unfortunately, the inclinometers failed to provide adequate information during this period due to a blocked tube caused by vandals. This process is a likely explanation of the increase in pressure at the rear of the capping beam towards the passive value, similar to that recorded by Broms and Ingleson (1972).
The lateral pressures exerted on the side faces of the columns during the first year of traffic loading are shown in Figures 4.34 and 4.35 and again, the pressures are seen to fluctuate according to the time of year. This can be explained by the expansion and contraction of the western capping beam in a transverse direction, similar to that which occurred immediately after it was poured. During the warm season, the pressures increased on the west face of the columns and dropped slightly on the east face, indicating that the expansion of the capping beams occurred symmetrically about the central expansion joint. It is also likely that the pressures were affected by temperature fluctuations of the backfill, and the effects of compaction due to the traffic.

At the end of construction, the recorded vertical pressures behind the abutments were only slightly greater than those predicted. However, as time has passed since being open to traffic, the pressures at various levels have increased considerably, as shown in Figures 4.36 and 4.37. It is significant that the vertical pressures recorded by the cells mounted into the base have not shown similar large increases. Furthermore, the overburden pressure has remained constant, except for the temporary vertical loadings. Consequently, it is suspected that the most likely cause of the recorded vertical pressure increases is due to a deterioration of the performance of the Maihak cells. Possible causes of deterioration could be due to corrosion of the cell or to a relaxation of the tension in the vibrating wire as a result of creep effects.

This section has discussed the effects of construction and long term traffic loading on the performance of spillthrough abutments. The following section discusses the results from the laboratory model tests which were designed to investigate the factors influencing a laterally loaded column embedded in a mass of soil.
4.2 Laboratory Model Tests

4.2.1 Introduction

The model tests have revealed a number of interesting points relating to the choice of the rectangular cross-section of a column, and its interaction with the surrounding soil. This has involved a study of the development of the soil resistance and the mode of deformation for a structure that is representative of a column in a spillthrough abutment. The major findings are presented and discussed in the following sections.

4.2.2 Development Of Shear Stress At Soil/Structure Boundary

One of the major aims of the laboratory study was to investigate the development of soil resistance at the sides of a laterally loaded rectangular column and assess its importance relative to the resistance offered by the soil at the front. Consequently, the shear stress generated at a soil/structure boundary has been investigated by shear box tests and pull-out tests. The two surface textures that were used for the models were smooth perspex and perspex with a coating of sand bonded by Araldite onto the surface.

The shear box tests were carried out at varying normal stresses for each of the soil/structure boundary conditions and for the soil alone, and the resulting peak angles of friction are shown in Figure 4.38. As expected, the smooth perspex indicated a low angle of friction of only 21°. However, the shear stress at the rough surface boundary was shown to be slightly higher than for the sand itself. This contradicted the generally accepted design assumption that the angle of friction developed at a soil/structure boundary could not exceed that of the soil alone. This assumption would seem reasonable because if the soil/structure friction is very high, then the soil immediately adjacent to it would be expected to shear instead. This may not have occurred in the shear box test because the shear plane was enforced to be at the soil/structure boundary. Therefore, the adjacent soil was restricted from shearing due to the confinement offered by the upper half of the box. In the case of
the model tests, no such shear plane was enforced and therefore, it is reasonable to assume that the peak angles of friction of the smooth and rough boundaries were 21° and 36° with residual values of 18.5° and 35° respectively. These peak values correspond to coefficients of friction of 0.38 and 0.73 respectively, which means that the rough surface generates almost twice the shear stress of that of the smooth surface at a similar normal stress.

The relative displacement required to fully mobilise the shear stress was determined from both the shear box and pull-out tests. A plot of the displacement required to develop the peak friction versus the normal stress, as determined from the shear box tests, is shown in Figure 4.39. It can clearly be seen that the displacement required to develop the peak friction at a soil/structure boundary is approximately half that required for the soil alone. Furthermore, it appears that the displacements required to develop the peak friction tend to increase for an increasing normal stress. At a very low normal stress, the required displacements were found to be about 1mm for a soil/structure boundary and 2mm for the sand alone.

The development of the shear stress with progressive displacement at a normal stress of 150kN/m² is shown in Figure 4.40. It can be seen that the shear stress increases more rapidly at a soil/structure boundary than within the soil mass as a whole. Furthermore, in all cases, a further movement of 2mm was sufficient to reduce the shear stress to its residual value. The decrease in shear stress from its peak value to its residual value was found to be about 14% of the peak value regardless of the shear plane conditions.

The pull-out tests for the smooth and rough perspex indicated different shear stress characteristics. The average shear stress was calculated after allowing for the reduction in surface contact as a result of extraction of the sheet. The variation of the shear stress with continued extraction is shown in Figure 4.41. It can be seen that the shear stress against the smooth perspex tended to increase for the first 30mm of relative movement and thereafter remained constant. In contrast, the peak shear stress against the rough surface was developed after a displacement of only 0.5mm. The shear stress was then seen to decrease rapidly for a further displacement of about 10mm and thereafter decreased only slightly.
The discrepancy between the shear box and pull-out tests results can be explained by a tendency of the sand to dilate during shearing. In the shear box test, the normal stress would not have altered due to dilatation of the sand because the loaded top platen would simply accommodate the expansion. Conversely, in the pull-out tests, soil dilatancy would have tended to increase the normal stress on the perspex sheet because the sand was confined by the rigid walls of the tank, albeit at a considerable distance from the sheet. For the case of the smooth perspex boundary, the shear plane occurred at the soil/structure interface and was detected in both the shear box and pull-out tests to occur in repeated slipping actions. This involved the stress increasing to a critical value at which stage the boundary slipped to reduce the shear stress to a lower value. The failure continued in a stepping fashion within the limits as indicated (in Figure 4.41) for the pull-out test. The steady increase in the shear stress can be explained by the dilatation of the soil, thus causing an increase in the normal stress against the perspex sheet as it was extracted. The slipping action prevented large permanent deformations of the sand and therefore minimised the reduction of the normal stress. In contrast, the extraction of the rough sheet caused an immediate dilation of the sand but the normal stress was soon relieved by the excessive rolling of the sand particles adjacent to the rough surface. This was confirmed by a tendency for the rough sheet to deposit sand at the surface as the extraction progressed, which was not observed for the smooth sheet.

4.2.3 Effect Of The Column Aspect Ratio On The Magnitude Of Soil Resistance

A number of columns with different aspect ratios were translated horizontally through sand in the pull-through tests. The aspect ratio has been defined as the ratio of the face width to the side length.

The columns were translated in increments and the loads were recorded on the top and bottom load cells, (Figure 3.12). The load displacement relationships at the top and bottom load cells of the smooth 122mm x 75mm column are shown in Figure 4.42. This type
of response was typical of all the tests in which it was generally found that the load at the bottom was proportional to the load at the top. The loading at the bottom was always greater than that at the top because of the difference in the boundary conditions. Firstly, at the bottom the vertical overburden of the soil above contributed to an increased soil resistance. Secondly, the movement of the soil at the base of the tank was resisted by the friction generated against the glass sheet. Thirdly, the resistance of the soil at the top of the tank was reduced as a result of its ability to heave at the free surface. The smooth perspex columns indicated an average ratio of the bottom load to the top load of 1.54, whereas a value of 1.62 was obtained for the rough perspex models. The higher ratio for the rough models can be explained by the additional locked-in lateral stresses that may have occurred towards the bottom of the tank during compaction due to the restraining action offered by the highly frictional surface of the model. The free upper surface reduced the tendency for locked-in lateral stresses to occur at the top. For such conditions, the larger proportion of load experienced at the bottom would therefore be due to the increased frictional resistance on the column sides. The constant proportionality between the upper and lower loads has enabled the total loads to be considered without significant error and will therefore be used for simplicity in the remainder of the discussion.

As expected, the peak soil resistance on the rough models was always higher than that on the smooth models of the same cross-section. However, it was noticed that the length of the column side had a significant effect on the ratio between the loads. It was found that the ratio of load for a rough model to a smooth model was generally higher for the columns with the longer side lengths, as indicated in Table 4.1.
<table>
<thead>
<tr>
<th>Side Length</th>
<th>Face Width</th>
<th>Height</th>
<th>Peak Load Ratio (Rough/Smooth)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>37.5</td>
<td>345</td>
<td>1.097</td>
</tr>
<tr>
<td>61</td>
<td>37.5</td>
<td>345</td>
<td>1.180</td>
</tr>
<tr>
<td>122</td>
<td>7.5</td>
<td>345</td>
<td>1.364</td>
</tr>
<tr>
<td>122</td>
<td>37.5</td>
<td>345</td>
<td>1.252</td>
</tr>
<tr>
<td>122</td>
<td>75.0</td>
<td>345</td>
<td>1.354</td>
</tr>
<tr>
<td>260</td>
<td>7.5</td>
<td>345</td>
<td>1.595</td>
</tr>
</tbody>
</table>

Table 4.1 Ratio of peak loads for rough and smooth columns with similar cross-sections

In conclusion, it is evident that increasing the surface roughness of a column causes a significant increase in the soil resistance, primarily due to the additional load transmitted to the soil via the column sides.

The contribution of the load from the column sides to the total load was also deduced from a series of tests on columns with identical face widths but with various side lengths. Tests were performed for a 37.5mm face width and side lengths of 0mm, 61mm and 122mm for both rough and smooth conditions. The intention was to compare the peak loads and deduce the load carried by the sides. However, each set of three tests failed to indicate a consistent value of the shear stress at the sides and therefore, an overall average value was determined from the individual average values determined at column displacements of 5mm, 10mm, 15mm and 20mm. A similar procedure was carried out for the columns with a face width of 7.5mm and side lengths of 122mm and 260mm. The overall average values of shear stress that were deduced are shown in Table 4.2.
<table>
<thead>
<tr>
<th>Surface Texture</th>
<th>Face Width (mm)</th>
<th>Av Shear Stress (kN/m²)</th>
<th>Range (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Smooth</td>
<td>7.5</td>
<td>2.4</td>
<td>2.3 - 2.8</td>
</tr>
<tr>
<td></td>
<td>37.5</td>
<td>0.6</td>
<td>0.3 - 1.3</td>
</tr>
<tr>
<td>Rough</td>
<td>7.5</td>
<td>5.7</td>
<td>4.4 - 7.1</td>
</tr>
<tr>
<td></td>
<td>37.5</td>
<td>5.2</td>
<td>3.7 - 5.9</td>
</tr>
</tbody>
</table>

Table 4.2  Average values of friction developed on column sides

Initially, it was expected that the 7.5mm and 37.5mm wide columns should produce the same value of shear stress on the sides. However, the 37.5mm column had a hollow section whereas the 7.5mm column was solid. This meant that the 37.5mm was more flexible and this has been reflected by the lower recorded values of shear stress.

The overall average values of shear stress in Table 4.2 do not show particularly good agreement with the peak values of shear stress of 2.4kN/m² and 3.6kN/m² for the smooth and rough perspex respectively, as were measured from the pull-out tests. Furthermore, the average values of shear stress were not consistently observed to decrease as the column moved laterally, unlike the reduction that was observed during the pull-out test on the rough perspex. Consequently, it is most likely that the shear stress would vary in a similar manner to that observed during the shear box tests, in which the shear stress reduced to a residual value after having attained its peak value.

An attempt at measuring the shear stress on the column sides was also made by using strain gauges on the inner face of the 75mm x 122mm column. Unfortunately, the method proved to be unsuccessful for this purpose but it did show up some other interesting features. Firstly, at a column displacement of only 1mm, the gauges generally recorded tensile strains on the inner faces, as shown in Figure 4.43. This meant that the sides were
bowing inwards, which is consistent with the earlier conclusion from the pull-out tests that the sand was dilating. The second interesting feature to emerge from the strain gauge data was that at large displacements, the column sides buckled in an almost random manner. This was due to the inevitable bending of the column about its vertical axis as a result of the load being applied at each end of the central steel bar. The column deformation would have been further complicated by an imperfect seating of the bar onto the inner surface of the front face of the column.

Having obtained an estimate of the loads transmitted via the column sides, it enabled the load on the front face to be calculated. The ratio of the side load to the front load at ultimate conditions has been plotted against the column aspect ratio in Figure 4.44. As expected, the effect of side load becomes less significant for high aspect ratios. However, for the case of columns in a spillthrough abutment, it is more relevant to consider the resistant loads corresponding to working conditions rather than ultimate conditions. The development of the soil resistance at distinct displacements for the different column widths is shown in Figure 4.45. At a displacement of less than 0.5mm, the face width has very little effect on the total load. It would therefore seem likely that the shear stress on the column sides contributes considerably to the total load at small displacements. This would seem reasonable because it has already been shown that the full shear stress is mobilised for a displacement of only 0.5mm to 1.0mm. If the average values of fully mobilised shear stress for the 7.5mm wide columns are used from Table 4.2, then the loads transmitted onto the 122mm long column sides are calculated to be of the order of 200N and 480N for the smooth and rough models respectively. These values correspond to approximately 28% and 45% of the total loads recorded at a displacement of 0.5mm.

For the case of the full size columns in the abutments at Wisley, it has been possible to obtain an estimate of the relative contributions of the side load and front load from the lateral pressures recorded 24 hours after the west deck pour. The forwards movement of the top of the E3 column during this period has been estimated, from
the inclinometer and precise surveying data, to be approximately 1mm (Figure 4-22). The contribution of the front load has been taken to equal the increase in pressure at the top of the rear face of the column, acting uniformly across the face width. The side load has been estimated by applying a coefficient of friction for the soil/concrete interface of 0.59 (as given by Potyondy (1961)) to the lateral pressure measured at the top of the side faces of the column. Again, this normal pressure has been assumed to act uniformly along the side length. If the lateral pressures are assumed to also act uniformly over a 1m depth of column, the front load and side load are calculated to be 30kN and 91kN respectively. This means that the side load was equivalent to 76% of the total soil resistance at this level, when the column moved laterally by approximately 1mm. The corresponding proportions of the side load to the total load calculated in a similar manner for the A2, A3 and E3 columns are 43%, 71% and 55% respectively. This clearly indicates the importance of the friction of the soil against the sides of a column on the ability of an embedded column to resist applied lateral loading.

Unfortunately, there are a number of reasons why the results of the model study are not suitable for predicting the relative proportions of the front and side loads for a full size structure. Firstly, in the laboratory tests the lateral stresses after compaction were very small, whereas on a full size structure this would certainly not be the case if heavy compaction plant was used. Consequently, the locked-in stresses caused by compaction around a full size structure would create much larger shear stresses on the column sides, therefore increasing further still the significance of the side loads for small lateral displacements. Secondly, it should be noted that the magnitude of the side load is also affected by the friction characteristics of the soil/structure interface. Thirdly, as is shown later in this section, the soil resistance against the front face of a column is not directly proportional to the width of the front face. Nevertheless, despite the differences in the conditions experienced between a model and a full size structure, the results from the laboratory investigation have succeeded in providing a good indication that the soil resistance against the sides of a laterally
The displacement of the column required to mobilise the peak soil resistance did not show any consistent trends with regards to the surface roughness of the model. However, the face width of the column was found to have a major influence on the displacement. The usual design criteria for the displacement required to produce the full passive resistance of a retaining wall is based on a fraction of the height of the structure. If the value suggested by Terzaghi (1934) of 0.01H for a wall translation is applied to the 345mm high models, then the required displacement for full passive conditions would be about 3.5mm. The corresponding value based on 0.05H, after Wu (1977), is 17.2mm. Clearly, these values do not show good agreement, nor do they accurately predict the range of displacements from 2.8mm to 22.0mm that were recorded from the pull-through tests for the narrow and wide columns respectively. It is therefore clear that such an approach based on the height of the structure is totally inadequate for columns of limited width. The relationship between the displacement to produce the full passive...
resistance and the column face width as derived from the pull-through tests is shown in Table 4.3. These results were all obtained from columns 345mm high and with a side length of 122mm.

<table>
<thead>
<tr>
<th>Face Width, B (mm)</th>
<th>Displacement at peak load, $\delta$(mm)</th>
<th>$\delta$/B</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Smooth</td>
<td>Rough</td>
</tr>
<tr>
<td>75</td>
<td>22.4</td>
<td>14.8</td>
</tr>
<tr>
<td>37½</td>
<td>15.0</td>
<td>15.8</td>
</tr>
<tr>
<td>7½</td>
<td>2.8</td>
<td>3.6</td>
</tr>
</tbody>
</table>

Table 4.3 Relationship Between Displacement At Peak Load And The Face Width For Columns With A Side Length of 122mm

The results in Table 4.3 show that the displacement is not directly related to the face width either. This is likely to be a result of different failure mechanisms that can occur depending on the front width and height of the column. This is discussed in more detail in the following section (4.2.4).

If the load displacement relationships for the top and bottom of the columns are considered independently, it reveals a consistent trend for the displacement required to mobilise the peak load to be less at the top than at the bottom. This type of response shows similar trends to those observed from centrifuge tests on vertical anchor plates by Dickin and Leung (1983), in which the failure displacement was found to increase for greater depths of embedment.

One of the simplest ways of estimating the passive resistance of a retaining wall is to assume that the soil pressures increase linearly with depth. An investigation on the passive pressure exerted on a model retaining wall was reported by Narain et al (1969). Their study concluded that a triangular pressure distribution along the height of a wall was realistic if the wall was translating towards
the soil. Consequently, it is felt that the assumption of a triangular pressure distribution can reasonably be applied to the case of translating columns, as were used in the pull-through tests. The magnitude of the triangular pressure distribution against a single laterally loaded column has been deduced by considering the load on the front face from the tests on columns with a side length of 122mm. Hence the values of the coefficient of passive earth pressure, $K_p$, have been determined, based on a soil density of $1675\text{ Kg/m}^3$ and a column height of 345mm. Figure 4.46 illustrates that the deduced value of $K_p$ increases for narrower columns. As a comparison, the values of $K_p$ obtained from the retaining wall tests were 15.9 and 19.7 for the smooth and rough surfaces respectively. Clearly, the values cannot be accurately predicted by assuming them to be similar to those for a retaining wall. This is further emphasised by considering a value of $K_p$ of 7.3 that is the best estimate for similar conditions using CP2. The vast underestimate of the passive resistance of a column using retaining wall theory is not surprising because it in no way accounts for a three-dimensional failure zone within the soil.

The influence of the surface roughness of a model has been reflected by the larger pressures that were recorded on the rough models than for the smooth models. In Table 4.1 it was shown that for a 37.5mm wide column with a negligible side length, the total peak load for the rough model was about 10% greater than for the smooth model. However, Figure 4.47 tends to indicate that if the front loads are calculated by subtracting the estimated side loads from the total loads on the 122mm deep columns, then the rough models record 13-24% higher loads than the smooth models. This can be explained by the observation that the shear stress on the sides tended to drag the adjacent soil forwards, particularly for the rough models, thus effectively increasing the width of the loaded area at the front. A similar type of action was proposed by Kezdi (1957) to explain the bearing capacity at the bottom of a friction pile. It should also be noted that the recorded load for the rough 75mm wide column may have been overestimated because the soil deformation was restricted by the front wall of the tank. This accounts for the relatively high value of the front load ratio of 24%
that was recorded for this size of column. The influence of the surface roughness on the ultimate capacity of shallow anchor plates has been investigated by using a finite element analysis by Rowe and Davis (1982). It was found that an increased surface roughness can significantly increase the capacity of an anchor at the surface but that the effect diminishes with increased depth of embedment. If the model columns are considered to act similar to an anchor plate at the surface, then the ratio of load capacity between a perfectly rough and perfectly smooth surface is 1.67, according to Rowe and Davis. However, this cannot be directly compared with the model test results because the extremes of surface friction are not equivalent.

4.2.4 Zone Of Influence Of A Single Laterally Loaded Column

Specialised photography was used to record the deformation of the top surface of the sand during all of the pull-through tests. Additional information with regards to the formation of rupture surfaces was obtained by observing the deformation of black stripes on the surface of the sand. Together, these methods provided a useful indication of the mode of deformation at the soil surface. However, it must be noted that this information can only be used to tentatively predict the deformations at lower levels in the sand.

According to Weissenbach (1961), there exists two types of soil deformation that can occur due to the lateral translation of a retaining wall of limited width. Firstly, a narrow wall can cut into the soil and displace it laterally, or secondly, a wider wall can create a failure shell within the soil. He concluded that the former mode of deformation would occur for walls that were narrower than a critical width of 0.3H. If this is applied to the width of the front face of the translating columns used in the pull-through tests, it would suggest that in each case failure should occur by the column displacing the soil laterally. Indeed, this appeared to be the case for the 7.5mm and 37.5mm wide columns. However, there is no doubt that the peak resistance of the 75mm wide columns was reached immediately prior to the formation of a rupture surface within the soil. This was observed as a distinct rupture line at the
surface caused by the uplift of the soil within its bounds. Plate 4.3 shows the failure surface as a shadow line created by using a low level overhead lamp. The zone was elliptical in shape at the surface and it is suspected that the rupture surface formed a roughly conical shape with the apex near to the base of the column. This deduction is based on the knowledge that the rupture surface in the soil at the front of a translating retaining wall extends from the base of the wall. The size of the failure ellipse at the surface was found to be larger for the rough model than for the smooth model. The extent of the failure zone was surprising and this is reflected by the fact that the tank was insufficiently large to fully accommodate it, particularly for the rough model.

Another interesting feature of the failure of the soil was the development of apparently vertical shear planes, radiating outwards from the forward corners of the models. These failure planes were illustrated quite clearly by the stepped nature of the originally straight stripes on the surface of the sand, as shown in Plate 4.4. Such failure planes were clearly visible for the 37.5mm and 75mm models and probably existed to a small extent for the 7.5mm model, although the displacements were insufficient to provide conclusive evidence. As can be seen from Plate 4.4, these failure planes crossed one another within the soil immediately in front of the column. As a result, a triangular wedge of soil, in contact with the front face of the column, remained undisturbed by the shearing action. This was revealed more clearly during the excavation of the sand after a test. The sand was seen to remain in a stable vertical column adjacent to the front face of the model, as shown in Plate 4.5. This therefore indicates that the triangular zone of soil was merely compressed and densified by the forwards movement of the model column.

The direction and magnitude of the movement of the sand at the surface was recorded by the digitisation of successive photographs defining the positions of an array of ball bearings. A selection of the most informative displacement vector diagrams are illustrated in Figures 4.48 to 4.60. The displacement vectors provide a good visual appreciation of the soil movements. However, it is recognised
that their absolute accuracy is limited by a number of inherent errors that may have occurred due to the numerous steps involved in their development. The main sources of error that were noted are as follows:

i The increased distortion effects at the outer edges of the photographs. These were limited as far as possible by using a lens with the longest focal length that the laboratory conditions would permit.

ii The distortion of the photographic materials during the stages of developing, processing and enlarging.

iii The accuracy of the digitisation process with regards to the physical accuracy of the plotting device and the operators interpretation of the best positioning of the cursor to coincide with the underlying point on the photograph.

iv The loss of accuracy due to the small scale of the photographs compared to the actual size of the tank. This results in a magnification of any errors incurred during the previous stages of the procedure.

Despite these unavoidable inaccuracies, the technique has proved to be very successful at illustrating a number of interesting features of the soil deformation for varying test conditions.

One of the most noticeable features is that the direction of the soil movements show a similar pattern to those observed in bearing capacity problems. This means that the soil immediately in front of the column was observed to move in a similar direction to that of the column. The soil at either side of this central line was seen to have a considerable component of displacement in the transverse direction. The soil to the sides of the column was found to move in the opposite direction to that of the column. This type of movement indicates that the column was displacing the soil and forcing it to flow backwards past the column. The only other direction that the soil could flow was upwards, and this was
reflected by a considerable amount of heave at the surface, as shown in Plate 4.6. As can be seen from the plate, the effects of heave were observed at a considerable distance away from the column. The vertical movement was seen to diminish as the distance from the column increased. For the case of the 75mm wide column only, the differential vertical movement seemed to be superimposed on the general upwards movement of the failure block contained within the elliptical rupture surface.

The soil deformation plots that are presented are generally for a large column displacement of about 22mm, which exceeds that required to cause the peak soil resistance. However, several plots have been presented for the 7.5mm, 37.5mm and 75mm wide rough columns with a side length of 122mm, after a displacement of only 5mm. Apart from the soil displacements being considerably less than those for a column displacement of 22mm, it is also apparent that the directions of the soil movements are somewhat different. In particular, there is no evidence of the soil being forced backwards at the sides of the columns. This indicates that the direction of the soil movements changes as the column pushes through it. This is shown in more detail in Plate 4.7 for the 75mm wide smooth column with a side length of 122mm. This plate was created by superimposing a series of enlarged negatives taken after successive increments of column displacements for a single test. It clearly shows the tendency of the soil particles at the surface to change their direction of movement. As the column continues to push forwards, the tendency of the soil to flow outwards and backwards is made more apparent. Similar changes in direction of soil flow were recorded around an anchor plate by Hanna et al (1972).

The displacement plots also provided evidence of the tendency of the rough models to drag forward the soil adjacent to its sides. Conversely, for the smooth models, the soil was observed to move in an opposite direction to within one column width of the side of the column.
Initially, one of the main objectives of the pull-through tests was to obtain an estimate of the size of the zone of influence that a translating column caused within the soil. However, after studying the results, it has been realised that where a free surface is involved, the zone of influence cannot be assumed to be identical at every depth. This therefore indicates that the extent of the zone of influence is highly dependent not only on the face width of a column but also on its height. Furthermore, it is evident that the rougher the surface of the structure, the larger is the influence on the surrounding soil. The displacement plots were generally unable to indicate the full extent of the zone of influence because it appeared that they were limited by the presence of the tank walls. The 7.5mm wide column was the only one which indicated the zone of influence to be within the bounds of the tank. At the surface, the width of the failure region was observed to span a distance between 25 and 40 times the face width depending on the surface roughness of the model. Alternatively, the observed elliptical failure surface that occurred for the 75mm wide column was found to be nearly 10 times the face width of the column. However, this was not necessarily a true indication of the full extent of the zone of influence because the soil stresses were inevitably transmitted across the rupture surface, therefore causing the soil beyond to be affected. The influence of the column height on the width of the zone of influence is more apparent if the failure surfaces for shallow anchor plates are considered. Tests on surface anchors, by Dickin and Leung (1983) and Merkin (1951), has indicated an approximately elliptical failure surface but its width was found to be less than twice the width of the anchor plate. Tests on an 0.15m square anchor at a depth of 0.53m, by Buchholz (1930), has indicated an elliptical failure surface with a width nearly five times the width of the anchor at the sand surface. Poulos (1971b) has theoretically investigated the effect of column interaction for different spacings and depths of embedment, and it was found that deep piles have more influence on each other than short piles at the same spacing. This again tends to suggest that the zone of influence is partly dependent on the depth of embedment of a structure as well as being related to face width.
A finite element analysis on the up-lift resistance of anchors has been performed by Rowe and Davis (1982) and it was concluded that the dilative nature of soil can greatly increase the size of the zone of influence. Similar conditions must have probably contributed to the large zone of influence that was recorded in the dense sand, as was used for the pull-through tests.

The main reason for interest in the zone of influence of a column is to be able to estimate the effects of interaction between adjacent columns in a spillthrough abutment. Investigations of the interaction of piles (Williams (1979)), of circular columns in spillthrough abutments (Lee (1982) and Ah-Teck (1983)) and of anchor plates (Ovesen (1964), Akinmusuru (1978) and Hanna et al (1972)) has tended to suggest that the limiting spacing ranges between 5 and 9 times the width of the structure at ultimate conditions. The measured zones of influence as measured from the pull-through tests have indicated soil disturbance within a zone considerably wider than these values. This indicates that the soil deformations towards the edges of the zone of influence can have only a negligible effect on the passive resistance of a structure. For the case of spillthrough abutments, the columns are commonly spaced at a centre to centre spacing of 3.5 to 4.5 times the front width of a column. This suggests that column interaction is extremely likely to occur for such abutments at ultimate conditions.

It must also be noted that the interaction of adjacent columns may be different for different modes of deformation, (ie translation or rotation). It is felt that a rotating column would have a smaller zone of influence because if the centre of rotation was above the level of the base, then the effective height of the passive zone would be somewhat reduced. Furthermore, all the tests that have so far been mentioned have only considered the interaction at ultimate conditions. The displacement plots from the pull-through tests show that the zone of influence increases as the column deflections increase due to a progressive yielding of the soil in front of the column. For the case of spillthrough abutments, the design of the columns would be considered for lateral displacements much
less than the critical values, thus resulting in a lesser degree of column interaction than at ultimate conditions.

4.2.5 Influence Of A Base Slab On The Lateral Resistance Of A Single Column

The problem of estimating the performance of a laterally loaded column with a base attached to the lower end is almost unique to spillthrough abutments. Although an under-reamed pile may have essentially the same form of construction, it is rarely designed primarily to resist lateral loading and is more commonly designed as a bearing foundation.

In the case of a spillthrough abutment, the base slab is designed primarily to transfer the dead load from the deck slab safely to the underlying soil. However, as has been shown from the full size instrumented abutments at Wisley, the loading from the deck slab and the compacted backfill also create significant lateral loading conditions. As a result, it was decided to investigate, in more detail, the contribution of a base slab to the lateral resistance of a single column.

The column rotation tests have indicated that the existence of a base slab at the base of a column does indeed influence the resistance due to lateral loading. The tests were performed for a column with and without a base slab, which in both cases was buried to a total depth of 288mm. The bending moment distributions along the column were deduced from the closely spaced strain measurements and are shown in Figure 4.61. The effect of the base slab is immediately apparent by the increased value of bending moment towards the base, indicating an increased degree of fixity at this level. However, the bending moments along the upper section of the column were revealed to be very similar regardless of the base conditions. This shows that the soil was providing a consistent amount of resistance to the bending of the column. The maximum bending moment in the column was found to be 7% greater when the base slab was attached, indicating that the ability of a column
without a base to rotate within the soil causes a reduction in the magnitude of the maximum bending moment. The depth of the maximum bending moment has been found to occur slightly above the mid-point of the column for a column without a base and slightly below for a column with a base. This contradicts the common design approaches whereby the columns are designed to resist the maximum bending moment which can occur at the root of the column.

Another interesting feature to be noted from the bending moment distribution for the column without a base was the existence of a positive bending effect at the lower end. This could possibly have been due to the effects of friction from the soil on the underside of the column. However, a more likely explanation is that the lowest strain gauge provided a misleading output due to the close proximity of the column end plate.

The load displacement relationships at the surface for the columns are shown in Figure 4.62. The base slab was found to have very little influence on the deflections of the column at the surface during the initial stages of lateral loading, indicating that the column was being supported by the lateral resistance offered by the soil. However, as the loading increased and the shear strength of the soil became increasingly mobilised, the resistance to lateral movement offered by the base became more significant.

The deflected shape of the column was deduced from the bending moment distribution and was superimposed on the overall rotation of the column as measured by the LVDT's. This provided an estimate of the actual position of the column within the soil relative to its initial position. Figure 4.63 shows the lateral deflections for a lateral load of 20kgf. This figure illustrates the effectiveness of the base slab at reducing the lateral movement at base level. The corresponding pressure distributions have been calculated by assuming that the pressures are constant over the full width of the column at any particular depth. Figure 4.63 clearly shows the large pressures generated on the rear face towards the lower end of the column. The pressures generated on the column with a base are seen to be
less than those for a column alone, thus further illustrating the effect of the resistance offered by the base.

The effectiveness of the base slab at reducing the lateral pressures and deflections is due to several factors. Firstly, the backwards movement of the base slab is restricted by the large area of contact between its rear vertical face and the adjacent soil. Secondly, the self-weight of the soil directly above the base tends to provide a clamping action. Finally, the resistant vertical pressure on the base slab maybe considerably increased by the application of vertical load to the column, such as would be exerted by the dead-weight of a bridge deck.

The level of a point at the top of the column was observed to drop slightly as the lateral loading was increased. This was explained by the rotation of the column and indicates that the column was not being dragged out of the soil. The centre of rotation can be deduced from the deflection profile or as the point of zero pressure from the pressure profile. These results indicate that the centre of rotation for a column without a base occurred between 73% and 78% of the depth of the column, which agrees with the values from 75.5% to 78% as determined by Bhagat (1967), Baguelin et al (1972) and Petrasovits and Awad (1972). The corresponding range for a column with a base was between 79% and 81%. This illustrates that the centre of rotation occurred above the level of the base in a similar manner to that observed for the full size structure at Wisley. Unfortunately, a direct comparison of the relative depth of the centre of rotation between the model and the full size structures was not considered to be valid for a number of reasons. Firstly, the geometry of the sloping embankment at the front of the Wisley abutments was not reproduced in the model tests. Secondly, the measured bending of the model columns did not accurately represent the rigid columns in the full size structure. Finally, the vertical loading of the deck slab in the full size structure was not simulated in the model study.
This chapter has so far discussed in detail the results obtained from the full size and laboratory investigations. This has led to a broader understanding of some of the major factors which may influence the behaviour of a spillthrough abutment. Consequently, the following section considers the most important findings and suggests how these may be incorporated into future methods of design.

4.3  The Design Of Spillthrough Abutments

4.3.1  Introduction

It is interesting to compare the general findings from the experimental laboratory investigation and from the monitoring at Wisley, with the assumptions that are adopted in the existing common design approaches for spillthrough abutments. In performing this comparison, a number of discrepancies are revealed between practice and theory. In the majority of the existing design methods the soil pressures are only considered for the situation in which the construction of the abutment has been completed and the subsequent deformations are sufficient to create the active conditions. It appears that this may be a very simplistic idealisation of the problem for a number of reasons which will be discussed. The abutments at Wisley have indicated that significant loadings can be applied to an abutment as a result of certain stages of construction. It is therefore reasonable to assume that deformations occur continually during the construction process instead of occurring only after the construction has been completed.

It must be realised that a single case history, such as Wisley, can in no way be considered adequate to provide justifiable grounds for completely modifying the existing design theories to cater for all spillthrough abutments. It is therefore intended that the following discussion will help to provide a greater understanding of the possible actions involved with spillthrough abutments and so create a broader basis for design.
4.3.2 Summary Of Existing Design Theories

In the past, it has been common to assume that the lateral earth pressure increases linearly with depth on the sides of the abutment. For a cohesionless soil, the magnitude of the lateral pressure is usually estimated by calculating a coefficient of earth pressure from the internal angle of friction of the embankment fill. Apart from actually estimating the magnitude of the pressure, it is also necessary to determine over which faces of the abutment the pressures are likely to act. It is apparent from the many different design approaches, as found by the BRE survey (Hambly (1979)) that there exists a great deal of uncertainty as to whether the resistant earth pressure on the front faces of the columns should be included in the stability analysis. Furthermore, the effects of side friction on the columns and of soil arching between columns are very vague.

The common design approach of Chettoe and Adams (1938) is based on the assumption that the soil tends to flow between the columns towards the sloping front face of the embankment. Net active pressures are proposed to act on the rear of the abutment. This is based on the assumption that any earth pressure on the front face of the columns would be approximately equal to the increase in pressure above the active value on the rear of the columns due to the forward flow of soil. The load on the rear of the columns is increased by an arbitrary allowance of up to 100% to cater for the effects of side friction, soil arching, settlement and a slight outwards flow of the soil. Huntington (1957) recommends that no reduction in the active pressure across the gross width of the abutment should be assumed when the width of the openings between the columns is less than twice the width of the rear of the column. However, no information is provided to indicate the magnitude of the reduction in earth pressure that should be applied to columns with greater spacings. Unlike the Chettoe and Adams approach, Huntington suggests that the soil at the front may be assumed to provide up to active resistance but that a reduction should be made to allow for the descending slope. Again, no information is given as to how to assess the reduction. It is suggested that the active pressures should be increased by 25% if the crest of the abutment is
prevented from deflecting. The other common approaches which form the outer extremes of design are to assume either that no lateral earth pressures are exerted on the columns, or alternatively, that full active pressures act over the gross width of the structure, regardless of the column spacing. Another type of design approach has been suggested by the Hampshire Sub-Unit of the South Eastern Road Construction Unit (1972) whereby the abutment is assumed to be rigidly embedded below the level where a 4m width of fill exists across the sloping embankment at the front of the abutment. Above this level, active pressures are taken to act on the rear face of the abutment if it is assumed to be able to deflect. Alternatively, if the abutment is assumed to be non-yielding, then at-rest pressures are taken to act on the rear face. The pressures on the rear of the columns can be doubled in a similar manner to that proposed by Chettoe and Adams (1938).

The design of spillthrough abutments is very dependent on the prediction of the earth pressures because they constitute one of the major forms of loading. In particular, an accurate estimate of the lateral pressures acting on a structure is essential for determining the critical longitudinal bending moments likely to be created in the abutment columns. In the case of the Hampshire Sub-Unit 4m assumption, this involves designing the column section to resist the bending moments created at the level where a 4m width of embankment exists at the front of the abutment, whereas, all the other methods design the column section to resist the maximum bending moment created at the column root. However, the prediction of the lateral earth pressures is not solely required to design the columns to resist longitudinal bending, as clearly, it is also very important for determining the shear, bending and torsional resistances of the other parts of the structure, such as the base slab, the capping beam, the curtain wall and the wing walls. This emphasises the necessity for improving the methods of estimating the earth pressures against spillthrough abutments, so as to be confident of producing satisfactory designs in the future.
4.3.3 Effects Of Construction

The Wisley investigation has indicated that considerable loadings can be exerted on the abutments during the period of construction. However, the common existing methods of design generally only consider the earth pressures generated after construction has been completed. It is therefore felt that future designs would benefit by considering the conditions that can be developed as a result of particular stages of construction. The remainder of this section discusses the conditions that were observed after the most significant stages of construction at Wisley and suggests possible refinements for the design procedure.

(a) Column Fill

The placing of the backfill up to the top of the columns at Wisley was carried out such that the level of backfill was raised uniformly all around the columns. As a result, there were only minimal out-of-balance forces acting on the columns which therefore did not constitute any significant threat to the stability of the structure, or bending of the columns. This agrees with the common design approach which assumes that negligible lateral movements of the columns occur during this stage of construction thus resulting in equivalent lateral pressure distributions on all faces. However, the magnitude and distributions of the measured lateral earth pressures were found to be underestimated by the design assumption of at-rest pressures of 9.4H. It is therefore suggested that the lateral earth pressures would be more accurately predicted by a method, such as that proposed by Broms (1971) or Ingold (1979c), which take into account the effects of soil compaction. Nevertheless, the Wisley study has indicated that this stage of construction does not represent a critical condition for the design of a spillthrough abutment provided that the backfill is to be raised uniformly around the columns. However, if this method of construction is not adopted, then it would be advisable to check the bending resistance of the columns to withstand any out-of-balance pressures that are exerted by the backfill, taking into account the likely effects of soil compaction.
(b) Capping Beam Pour

The Wisley investigation has revealed that the abutment columns were subjected to transverse bending as a result of the expansion of the concrete capping beam during the first 48 hours after casting. In the original design calculations for the Wisley abutments, this type of action was not considered when designing the columns to resist transverse bending. Instead, a total transverse bending moment was calculated for a completed structure from a combination of the following loadings;

(i) Vertical Loads; due to dead load from deck
due to self-weight of beam and curtain wall

(ii) Horizontal Loads; due to active earth pressure on one side of column
due to active earth pressure and surcharge on wing walls
due to wind loading on the deck
due to braking forces and impact loading.

Although, in the case of the Wisley abutments, the transverse bending of the columns was not critical as a result of the capping beam expansion, it would be worthwhile to check for such a condition in future designs. This would be more important, if for some reason the capping beam was expected to be constructed before the placement of the backfill around the columns. In such a case, the effects would be more severe due to the absence of any soil resistance. Furthermore, it would be advisable to investigate the possibility of a transverse propping support provided by an adjacent previously cast capping beam, as this has been shown to lead to increased bending of the outer columns.

(c) Deck Slab Pours

The deck pours were found to produce large horizontal loadings at the top of the abutments, thus causing significant deformations and
variations in soil pressure. Nevertheless, such conditions are not necessarily applicable to all bridge abutments, because at Wisley the lateral loadings were found to be a direct consequence of a poor formwork detail. Therefore, it is suggested that in future designs, particular attention should be given to the detail of the deck formwork so as to minimise the effects of deck expansion on the horizontal loadings transmitted to the abutment. In particular, this would require the formwork to be designed to permit an unrestricted operation of the bearings, which basically means that the bearing downstand formwork should be isolated from the abutment. This would prevent damage to the alignment of the bearing plates and would also ensure a more efficient operation of the bearings in the long term. The other major problem of isolating the deck slab from the abutment is associated with the need to create an expansion joint. Clearly, the use of polystyrene blocks provides a simple and effective means of creating such a joint. At Wisley, however, this was found to cause a direct transfer of horizontal loading to the top of the abutment from the expanding deck slab. Consequently, unless a more suitable method of creating the expansion joint can be developed, the horizontal loading transmitted to the abutment from the expanding deck slab should be given serious attention as a unique design loading condition. This would involve estimating the lateral and vertical reactions imposed by the newly poured deck and checking the design calculations for a condition where the backfill only exists up to the top of the columns.

(d) Backfilling Behind The Capping Beam

Another significant loading condition during the period of construction, was caused by the heavy compaction of the backfill in layers behind the capping beam. As the backfilling recommenced at capping beam soffit level, the soil was observed to flow forwards between the columns, in a similar manner to that proposed by Chettoe and Adams (1938). Under such conditions, the pressure on the rear of the columns tends to increase beyond the at-rest value, whereas at the front, the soil is forced away from the columns, causing the pressures on the front face to reduce towards an active value. Furthermore, the forwards flow of the soil between the
columns creates a resultant force due to the effects of friction at the soil/structure interface, acting towards the centre of the bridge. However, at Wisley, the forwards flow of soil was seen to cease after the first 1m of soil had been placed behind the capping beam and thereafter, the abutment was pushed forwards relative to the soil. The forward movement of the abutment was found to cause closure of the expansion joint such that the remaining pieces of packing material within the joint were able to transmit a propping force from the end of the deck slab: Under these conditions, the earth pressure tends to increase on the front of the columns. Although the forward movement of the abutment might be expected to decrease the pressures on the rear of the columns, it is more likely that the compaction of the overlying backfill would counteract this effect and would instead cause the pressures to increase. Furthermore, the effects of friction at the soil/structure interface tend to cause a resultant force on the column sides, acting away from the centre of the bridge. It is therefore evident that contrary to the Chettoe and Adams approach, the compaction of the backfill behind the capping beam generally causes the soil around the columns to add to the stability of the abutment rather than act as a cause of instability.

Most of the design methods assume that the earth pressures at the rear of the capping beam correspond to active conditions. This implies that it is expected that an abutment should be capable of yielding in a forwards direction. In the case of the Wisley abutments, the assumption of active conditions at the rear of the capping beam at the end of construction was totally incorrect, as shown in Figures 4.9 to 4.12. It would seem likely that the compaction of the backfill in layers behind the capping beam caused a progressive lateral displacement of the abutment, in a similar manner to that described by Sims et al (1970) and Casagrande (1973). The heavy compaction used to compact the backfill tended to recompact the underlying layers of soil, therefore preventing the abutment movements from causing the active conditions. After backfilling had been completed, the lateral pressures against the capping beam were very substantial. Although no actual measurements were taken, it is likely that the heavy compaction also caused large lateral pressures to be exerted.
against the inner face of the wing walls. The earth pressures on cantilevered wing walls of a standard motorway bridge abutment were measured by Jones and Sims (1975). It was found that after the compaction of the backfill, a rectangular to parabolic pressure distribution of about 700lb/in² (101.6kPa) was created on the inner face of a wing wall. It is therefore suggested that the lateral earth pressures created on the rear of the capping beam and on the inner face of the wing walls would be more accurately estimated by methods which account for the effects of compaction rather than by simply assuming active conditions. Such methods have been proposed by Broms (1971), Aggour and Brown (1974) and Ingold (1979a).

4.3.4 The Stability Of The Wisley Abutments After Construction

The bending effects due to earth pressures at the roots of the columns of the Wisley abutments have been evaluated from the resultant earth pressures as measured at the end of construction, (Figures 4.9 to 4.12). The bending moments have been calculated by assuming that the pressure varies linearly between each of the pressure cell positions and that the pressure reduces to zero at the top edge of the abutment. Also, the pressure profile is assumed to remain constant across the width of the abutment being considered. The contribution of the soil friction against the column sides has been estimated by assuming that at-rest pressures of 9.4H are exerted in the transverse direction. The longitudinal friction component has then been calculated by applying a coefficient of friction of 0.59, as predicted from the work of Potyondy (1961), for a dry sand with an internal angle of friction of 35° against a smooth concrete surface. The estimated bending moments are compared in Table 4.4 with those determined directly from the average induced strains measured at the columns roots, as given by Lindsell (1984). The short term value of Young's modulus for the concrete has been taken as 31kN/m², (Abdul Razak (1985)), and the section modulus of a column equals 0.225m³.
Table 4.4 Bending moments at column root estimated from Wisley data

Table 4.4 clearly illustrates that the bending effect at the column roots was far less than that estimated from the soil pressures. This indicates that there must have been another force acting which had a significant effect on reducing the bending moment at the column roots.

In the original design, a number of additional loadings were considered such as surcharge, impact and braking forces caused by traffic. These cannot, however, account for the bending restraint because the road at this time was still not open and in any case the traffic loads would be likely to cause an increased bending of the columns. In addition to the traffic loads, the original design also accounted for the effects of bearing friction and expansion joint stiffness. It is likely that these forces would indeed have contributed to the reduction in bending. However, if the design value of bearing friction of 4% of a maximum dead load of 607kN (ie 6kN/m width) and an expansion joint restraint of 2kN/m width are assumed for the longest A3 column, then these forces should only contribute a restoring moment of 185kNm and 72kNm respectively. Clearly, forces of this magnitude do not account for the large discrepancy in bending moments.
If the mode and magnitude of the abutment deformations are considered during the period of backfilling, it is clear that the forward movement of the capping beam must have caused considerable closure of the expansion joint. This would have resulted in the full mobilisation of the bearing friction and may also have caused contact to occur between the end of the deck slab and the abutment via small pieces of remaining polystyrene packing material. It has already been mentioned that considerable damage occurred to the bearing downstands during the deck pour and it therefore seems likely that the bearing friction was considerably larger than expected, thus providing a substantial propping force to the abutment. The contribution of the compressional resistance of small pieces of packing material is uncertain but it is likely that the measured closure of the expansion joint was adequate to cause a significant propping reaction from the deck slab. It therefore seems likely that the increased value of friction from the damaged bearings and the compression of the remaining packing material within the expansion joint were together responsible for reducing the bending moments at the column roots.

Back-analysis of the Wisley data has enabled the magnitude of the total horizontal reaction provided by the deck slab to be evaluated, as the combined effects of bearing friction and propping from the end of the deck slab. Since the remaining packing material was found to be towards the top of the expansion joint and the bearing friction acted at the bottom of the expansion joint, the line of action of the total deck reaction has been assumed to act horizontally at the mid-depth of the deck slab. The bearings were located at 4m centres and the calculated values of the total horizontal deck reaction per 4m width of the abutment are shown in Table 4.5.

<table>
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<tr>
<th>Column</th>
<th>Soil pressures (kNm)</th>
<th>Bending moment Column strains (kNm)</th>
<th>Net due to mid-depth of deck reaction (kNm)</th>
<th>Lever arm (m)</th>
<th>Horiz. deck reaction per 4m width (kN)</th>
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<tr>
<td>A2</td>
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<td>495</td>
<td>1889</td>
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<td>642</td>
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<tr>
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<td>5071</td>
<td>530</td>
<td>4541</td>
<td>7.6</td>
<td>597</td>
</tr>
</tbody>
</table>

Table 4.5 Estimated horizontal deck reaction from Wisley data
The total horizontal deck reaction that was assumed during the design was calculated as the total of expansion joint restraint (2kN/m) and the bearing friction (4% of the dead load of 607kN/m equals 25kN per bearing). Therefore, the total design value of the deck reaction was 33kN per 4m width of abutment. It is evident from Table 4.5 that the deck reactions produced at Wisley were at least seven times greater than those assumed in the design. Furthermore, it is extremely unlikely that the bearing friction alone could account for such high values of deck reaction. For instance, at the E3 bearing the coefficient of friction would have to be almost equal to unity. Therefore, the large values of deck reaction confirm the assumption that some degree of connectivity must have occurred between the deck slab and the abutment. The discrepancy between the calculated horizontal deck reaction for the A and E abutments can be accounted for by the lateral restraint offered by the fixed bearings at the intermediate columns along the span of the bridge.

Although the Wisley study has shown that the large deck reaction was caused by unfortunate methods of construction, it would seem possible that similar conditions could have occurred unnoticed on many other spillthrough abutments that have been constructed in the past. If this is the case, then it implies that the satisfactory performance of such structures during construction may have been partly due to the existence of significant propping forces from the deck slab which have been inadvertently underestimated during the design. From the Wisley study, it appears that the contact between the deck slab and the abutment only lasted until the end of the first warm summer and thereafter, the expansion joint opened up due to the contraction of the deck slab during all subsequent cooler seasons. This is discussed in more detail in section 4.3.6. Therefore, it would seem reasonable to suggest that similar propping actions could have occurred in the past during the construction of other abutments and that the subsequent opening of the expansion joint after the first warm season has disguised the fact that it occurred. However, it should be noted that at Wisley, the interaction of the deck slab and the abutment continued to be significant in the long term as a result of the damage that occurred to the alignment of the bearing plates during the deck pours.
4.3.5 The Effects Of Propping An Abutment Against The Deck Slab

As a result of the Wisley study, it has been found that the propping action provided by the deck slab had a marked effect on reducing the lateral displacements of the abutment caused by the high lateral earth pressures on the rear of the capping beam. Therefore, unlike the case for the deck pours, it would be worthwhile to consider the possibility of implementing intentional propping during the capping beam backfill operation so as to reduce the lateral displacements and also provide a bending restraint. If this could be achieved on a temporary basis only, then the subsequent release of the propping force would result in a reduction in the earth pressures on the rear of the capping beam towards the active value. This would mean that the bending in the columns would be limited, firstly during the backfill operation by the propping force and secondly, after removal of the propping force by the reduction in earth pressure.

Although the temporary propping of an abutment may seem to be a good solution in theory, it is of course appreciated that implementing such a solution in practice is by no means easy. However, it is felt that if the advantages of such a method of construction can be confirmed, it may well result in a more cost-effective abutment design. The design of a propping detail is beyond the scope of this investigation but one possible solution could be to provide temporary packing along the joint between the abutment and deck slab whilst the backfill is being compacted behind the capping beam.

Consequently, the present state of knowledge indicates that two types of earth pressure distributions can develop depending on the method of construction. Firstly, if as at Wisley, the abutment deforms laterally due to the compaction of the backfill behind the capping beam such that the curtain wall comes into contact with the deck slab, then it would be advisable to consider the possibility of high lateral earth pressures acting on the capping beam. If on the other hand, it is possible to implement a temporary prop during construction, then its subsequent removal would tend to make the adoption of active pressures on the rear of the capping beam more
appropriate. In the case of the Wisley abutments, the removal of a temporary prop would probably have caused a considerable reduction in the large earth pressures on the rear of the capping beam caused by compaction, preventing the abutment from coming into contact with the end of the deck slab. Furthermore, the existence of an adequate expansion gap would also have been likely to reduce the increase in pressures on the rear of the capping beam caused by the expansion of the deck slab during the first warm season. However, it should be noted that despite the use of the temporary prop, the movements of the abutment would probably have been sufficient to develop the frictional resistance of the soil against the sides of the columns.

4.3.6 Long Term Effects

The major factors that have affected the earth pressures in the long term on the spillthrough abutments at Wisley are due to the effects of soil compaction caused by traffic loading and the effects of contraction and expansion of the deck slab caused by temperature fluctuations.

At Wisley, the temperature fluctuations of the deck slab have had a significant effect on the lateral earth pressures, particularly those on the rear of the capping beam. However, the cause of the apparently large interaction between the deck and the abutment has been shown to be a consequence of the method of construction, whereby the backfill behind the capping beam caused the deck and the abutment to come into contact with each other. The only possibility of any relief of the propping force at this time would therefore have been if the deck slab contracted. However, this was unlikely to be very significant at Wisley because the backfill operation was completed in early Spring, when the temperatures were near their lowest value. Consequently, the large pressure increases on the rear of the capping beam during the following summer months were not surprising because of the failure of the expansion joint to maintain an adequate gap. This must undoubtably have resulted in the deck expansion causing the top of the abutment to be forced backwards against the embankment. The subsequent
contraction of the deck during the following winter dragged the
abutment towards by bearing friction only and therefore caused the
pressures on the rear of the capping beam to reduce towards an
active value and so produce a relatively stable condition. Recent
inspections of the expansion joints has indicated that no contact is
now occurring between the deck slab and the abutments.
Consequently, the results of the Wisley study indicate that the
expansion of the deck slab during the warm summer months is
capable of pushing the abutments backwards (ie towards the backfill)
such that an adequate expansion gap is maintained during all of the
subsequent cooler seasons. It would therefore seem that the closure
of the expansion joint was only a temporary condition caused by the
backfilling behind the capping beam and lasting until the end of the
first warm summer season. However, it is possible that similar
conditions of joint closure may again occur during periods of warm
weather of equivalent temperatures to those previously experienced
by the deck.

If on the other hand, a temporary prop had been implemented during
the backfill stage, then the expansion joint would probably have
remained open at all times. For a condition such as this, it would
seem likely that the seasonal temperature fluctuations may have had
a much lesser effect on the earth pressures acting on the abutment.
In fact, if the bearings work as intended, then the lateral loading
from the deck slab on an abutment should not exceed the assumed
bearing friction of 4% of the dead load, plus a small contribution
offered by the expansion joint. However, it must be noted that this
would not have been the case at Wisley because large values of
bearing friction were created as a result of the shutter distortions
during the deck pours.

If as at Wisley, the expansion joint is fully closed immediately after
construction, then the expansion of the deck during the first warm
season would result in a backwards lateral movement of the
abutment. This movement would not be greater than half the
increased length of the deck slab, if the deck is assumed to act
symmetrically about the mid-span position. The lateral thrust from
the deck would be opposed by a soil resistance tending towards the passive value at the rear of the capping beam. If a coefficient of thermal expansion of $11 \mu s/°C$ is assumed for a 76m long deck, subjected to a temperature range of $40°C$, then the resulting expansion at each end of the deck would be less than 17mm. The corresponding deflection to height ratio for the shorter E abutment would then be equal to 0.002, which is widely accepted to be inadequate to generate the full passive resistance of the soil. However, although ultimate conditions within the soil may not be created, the deflections may cause significant bending moments in the columns. If the columns are assumed to act as vertical cantilevers from a fixed base slab and the change in soil resistance is ignored, then the deflection of 17mm at the top would cause very large bending moments at the column root. For instance, such conditions would create a bending moment of about 3000kNm in the longest columns of the A abutment. It should however be appreciated that this high bending moment can be considerably reduced by the ability of the abutment to rotate within the soil, as was observed at Wisley.

In conclusion, it is suggested that the long term effects of deck expansion and contraction should be borne in mind when designing spilthrough abutments. Their importance should be assessed after considering the possible combined effects of expansion joint closure and the degree of fixity of the base slab.

4.3.7 Deformation Characteristics Of Abutments

The usual approach for designing the column sections to resist longitudinal bending is to assume that the column is cantilevered from a rigid base. With the assumption of a linearly increasing pressure with depth, this means that the column section is designed for the maximum bending moment that occurs at the root.

In the case of the Wisley abutments, the major causes of longitudinal bending were either due to horizontal loading from the deck slab or by the large lateral earth pressures behind the capping beam due to compaction. In both cases, the columns were
surrounded by soil up to the soffit level of the capping beam. The results from the Wisley abutments and the model tests have indicated that the base kicked backwards in the opposite direction to that of the load applied at the top. Consequently, the assumption that the base slab remains fixed in position is incorrect. Furthermore, it is evident that the soil resistance contributed considerably to the stability of the abutment. The soil resistance consisted of large earth pressures on opposite sides of the column at the top and bottom. In addition, some resistance was experienced due to the mobilisation of friction of the soil adjacent to the column sides. This represents a similar type of action to that described by Broms (1964) for a short rigid pile embedded in soil, which tends to rotate due to lateral loading. The model tests have shown that the rotation of the abutment and the subsequent mobilisation of the passive resistance of the soil, cause the bending moment induced at the column root to be less than the maximum bending moment. However, a similar bending moment distribution was not observed for the columns in the abutments at Wisley. This was because the Wisley columns were at a relatively shallow depth of embedment such that the maximum bending moment was recorded near to the root of the columns. A reduction in bending moment towards the base would probably have been recorded if the depth of embedment of the columns had been greater.

Clearly, the rotational characteristics of an abutment are highly dependent on the column stiffness, soil properties, size and type of base slab and vertical loading. As yet, there is insufficient information to be able to confidently include the effects of rotation into the bending analysis. Therefore, the present method of assuming that the columns act as cantilevers from a fixed base should be continued to be adopted until further information is available because this provides a conservative estimate of the bending moments.

4.3.8 Soil/Column Interaction

Until now, the action of the columns in a spillthrough abutment have been considered only by very crude methods. The approach of
Chettoe and Adams (1938) has been simply to double the active load on the rear face of a column to account for the effects of side friction, arching, settlement and soil flow. Huntington (1957) suggests that, for column spacings less than twice the column width, the active lateral pressure should be considered to act over the gross rear width of the structure. The method does, however, allow for a lateral pressure contribution from the soil at the front of the columns. The other common design methods generally simplify the analysis, such that the specific action of the columns is not considered at all.

One major deficiency of the existing methods, with regards to the Wisley abutments, was the failure to account for the condition when the structure pushes forwards into the soil. Instead, the general approach that has been adopted in the past has been to assume that the soil flows forwards between the columns.

Even though at Wisley, the abutments were mainly observed to push forwards, there was no indication that the resultant deformations could ever be sufficient to cause ultimate conditions within the soil. Therefore, the analysis should only consider the conditions caused by relatively small displacements. Likewise, only small relative displacements would be expected to occur if the soil was to flow forwards between the columns due to compaction at the rear of the capping beam. Consequently, it is likely that the development of soil resistance against the columns would be similar in each case, except that the direction of the forces in the longitudinal direction would be reversed. Hence, the following discussion will hopefully provide a better appreciation of the forces acting on the columns regardless of the predicted mode of deformation. This will involve the estimation of soil resistance at the front and at the sides of the columns and also the effects of interaction of adjacent columns.

The model tests have indicated that the passive resistance of a column cannot be accurately predicted by considering it to act as a narrow retaining wall. Instead, the three-dimensional nature of the affected zone of soil has been shown to considerably increase the passive resistance. Broms (1964) considered the ultimate resistance
of the soil in front of a pile in cohesionless soil to be equal to three times the Rankine passive value. Clearly, the limited displacements of a spilithrough abutment do not enable such high values of resistance to develop. Instead, the net longitudinal soil pressure on a column may be safely taken as the at-rest value (say 9.4H) acting on the leading face. At Wisley, at-rest pressures were exerted on both faces after the column backfill operations. Therefore, this assumption allows for the reduction in pressure towards the active value on the trailing face and an increase towards the passive value on the leading face as a result of subsequent relative movement between the structure and the soil. A factor of safety is built into this assumption if the soil adjacent to the trailing face is able to arch across onto the soil at the column sides and therefore reduce the lateral pressure to below the active value. The tendency of the structure to rotate and kick backwards at the base could well be accounted for by an approach similar to that used for a rotating pile by Broms (1964) whereby the high pressures at the base are replaced by a concentrated force.

If, as mentioned earlier, the deck is propped during construction, then the consequent displacement on removal of the props may be less than that caused by compaction of backfill behind an unpropped structure. In this case, it may be safer to consider a reduction towards a net active pressure on the leading face. In addition, the possibility of excavation within the sloping embankment at the front of the abutment must be considered because this could have a marked effect on the lateral earth pressures.

The model tests have shown that, at small relative displacements, the soil resistance on the side of a column can contribute significantly to the total resistance. It was found that a displacement of less than 1mm was required to fully mobilise the full shearing resistance on the sides of a column with a rough surface. In relation to a full size spilithrough abutment the displacements would be insufficient to mobilise the full passive resistance of the soil, although they would almost certainly be adequate to fully mobilise the frictional resistance on the column sides.
A suitable method of evaluating the frictional resistance is to assume a pressure distribution normal to the column sides and deduce the stress in a perpendicular direction using a coefficient of friction for the soil/structure interface. The Wisley data has suggested that the normal stress may be safely taken as an at-rest value of 9.4H. However, if the column spacing is such that the compaction of the backfill between the columns is likely to be difficult, then an active pressure distribution may be more appropriate. The value of the coefficient of friction should be estimated for the soil properties and the surface texture of the column likely to be used for the construction. Such values can be determined from shear box tests, or alternatively, Potyondy (1961) has published data from a series of shear box tests on various combinations of materials. It should be noted that a bitumen coating on the structure can seriously affect the frictional characteristics of the interface. Packshaw (1946) has suggested that the angle of friction may be equal to the internal angle of friction of the soil, whereas the Civil Engineering Code of Practice No2 (1951) recommends an angle of friction equal to 30°.

The zone of influence within the soil ahead of the leading face of a column has been shown to be over 10 times the face width of a column at ultimate conditions. However, the limits of the zone were found to be somewhat reduced when the relative displacements between the soil and the column are significantly less than those required to develop the peak soil resistance. It has already been suggested that the relative displacements experienced by a spillthrough abutment are likely to be very small compared to the peak values. Consequently, the column spacings that are commonly used for spillthrough abutments are likely to be wide enough to avoid the effects of column interaction under serviceability conditions.

The effects of column interaction for ultimate conditions have been investigated at the University of Cambridge by Lee (1982) and Ah-Teck (1983). These tests were designed to represent the conditions of toe-washout at the front of the abutment and differential
settlement. Under these conditions, the lateral pressures would be acting primarily on the rear of the columns and consequently the major forces available to oppose the forwards movement of the structure would then be the deck reaction and the friction from the soil acting on the surfaces of the base slab. These conditions may make the abutment vulnerable to being pushed forwards due to the earth pressures on the rear face, therefore making it advisable to check the factor of safety against sliding. The limit equilibrium approach, proposed by Ah-Teck (1983), could be used to predict any interaction between the columns caused by large relative displacements, although care must be taken to determine the assumed value of the earth pressure coefficient. Ah-Teck suggested that the assumption of at-rest conditions may overestimate the pressure and therefore provide an uneconomical design. If slippage of the abutment occurs, it is possible that the earth pressures would tend towards active values and therefore it may be better to assume an intermediate earth pressure coefficient between an active and at-rest value.

4.3.9 Trial Analytical Method For Predicting Loads And Bending Moments For The Wisley Abutments

As a result of the monitoring of the Wisley abutments, it has been possible to postulate the likely forces that were developed which have lead to their stability, (section 4.3.4). Although the Wisley abutments are not considered necessarily to be representative of all abutments, it has been possible to derive a modified form of analysis to account for the mechanisms that have been observed. It is hoped that this may provide an insight into how a modified form of analysis could eventually lead to a more representative design in the future. However, it should be noted that due to the present lack of data from full size monitoring, the following study can only be used as an illustration of how the Wisley abutments actually performed and therefore must not be considered as a proposed general approach for all abutments.
The modified form of analysis to be presented has been based on the observed conditions existing for the Wisley abutments after the completion of the backfill operation behind the capping beams. Only the major observations have been incorporated which are as follows:

(i) The abutments rotated about a position located above the base. However, for this analysis the rotation of the whole structure is considered to occur about a point at the root of the columns.

(ii) The deflected profile measured from the inclinometer in the E3 column (Figure 4.33(b)) has indicated that in addition to the overall rotation, the abutment flexed along its entire height by about 3mm to 4mm. This analysis considers the columns to act as cantilevers from a rigid base slab, which flex to produce a deflection of 3mm at the free end of the cantilever (ie top of abutment).

(iii) The abutment rotations caused the curtain wall to come into contact with the end of the deck slab, therefore causing a large reaction at the top of the abutment. In addition, the misalignment of the bearing plates caused a large value of bearing friction. A total deck reaction equivalent to the combined effects of bearing friction and deck propping are assumed to act as a concentrated horizontal force at the mid-depth of the deck slab.

(iv) The effects of compaction behind the capping beam caused high residual lateral pressures to be exerted. This analysis considers lateral earth pressures on the rear of the capping beam as determined by the theory of Ingold (1979a) for a Stothert and Pitt Vibroll T182A roller.

(v) The soil resistance at the front of the columns was increased by the forwards' rotation of the abutment. This analysis considers a net at-rest lateral pressure acting on the front face equal to 9.4H.
(vi) The base of the abutment kicked backwards causing high pressures on the rear face at this level. These are replaced by a concentrated force acting at the root of a column, which also includes the resultant force caused by friction on the underside of the base.

(vii) The friction on the column sides was generated by the forwards rotation of the abutment. This analysis considers at-rest lateral pressures to act on the column sides and a coefficient of friction for the soil/structure interface of 0.59, as obtained from Potyondy (1961).

The analysis has been performed to evaluate the total deck reaction, $R_D$, and the concentrated force, $R_B$, and bending moment at the column root, $M_B$, involving three main stages of analysis. Firstly, the abutment was assumed to act as a propped cantilever and three unknowns $R_D'$, $R_B'$ and $M_B'$ were calculated for the applied loadings from the soil, as shown in Figure 4.64(a). Secondly, an abutment flexure of 3mm was assumed to be caused by a horizontal point load, $(P)$, at the same level as the deck reaction, and the values of $P$ and $M_B''$ were obtained, as shown in Figure 4.64(b). Thirdly, the resulting unknowns were calculated as follows:

- Deck reaction, $R_D = R_D' - P$
- Concentrated force at column root, $R_B = R_B' - P$
- Bending moment at column root, $M_B = M_B' + M_B''$

In Table 4.6 the values of $R_D$, $R_B$ and $M_B$ for a 4m width of abutment, as determined from the Wisley data, are compared with those calculated by the above analysis.
Table 4.6 Comparison of loads and bending moments as determined from trial analytical method and from experimental findings at Wisley

The comparison of the values for the A abutment in Table 4.6 indicates that an analysis of this form is capable of predicting the reactions and bending moments for the structure. The agreement is not so good for the E abutment but this is probably due to the extremely large pressures that were exerted on the capping beam caused by the compaction of the backfill within an adjacent trench. The largest error in the prediction of the measured value of the bending moment was obtained for the E3 column but this would nevertheless have produced a safe design value.

Clearly, the analysis is very susceptible to the prediction of the amount of column flexure. In the above calculations, the measured value of approximately 3mm was assumed. However, if such a method was to be used in an original design, then the column flexure would be rather speculative because it would depend on the ability of a structure to rotate and the stiffnesses of the soil and of the columns. Consequently this form of analysis should not yet be used for designing spillthrough abutments. However, the observed actions of the abutments from which it has been derived should be noted, and their relevance to future designs should be individually assessed.
4.3.10 Comparison Of Design Methods

It would appear from the preceding discussion of the Wisley results that the deck reaction had a significant effect on the stability of the abutments and on the longitudinal bending of the columns. The mobilisation of this reaction was caused by the deformations of the abutments which were in turn caused by the soil pressures. Whether a similar large deck reaction is generally experienced by other spillthrough abutments is uncertain, however, the generation of earth pressures from the surrounding embankment is inevitable and a contribution to the development of an improved method of prediction has been the purpose of this study. Consequently, it was considered worthwhile to carry out a comparison of the longitudinal bending moments calculated by existing theories, with those predicted from the earth pressures measured at Wisley and those calculated by a proposed modified design approach which has been formulated (by the writer) from the results of this investigation. The influence of the deck reaction and live loadings are not included so that a direct comparison can be made of the maximum design values of column bending moments, as determined from the various approaches of estimating the lateral earth pressures.

A brief description of the assumptions used in the various approaches is given below.

(i) Chettoe and Adams Approach

Rankine's active pressures corresponding to an earth pressure coefficient \( K_a \) equal to 0.271 have been assumed to act on the rear face of the abutment, as calculated for a soil with an internal angle of friction of 35°. The effective width of the rear face of the columns was doubled to cater for the effects of side friction and soil arching.

(ii) Hampshire Sub-Unit Approach

Pressures identical to those used in the Chettoe and Adams approach were considered down to a depth of 2m below the tops of the
columns, where a 4m width of embankment exists in front of the abutment. Below this level, the columns are assumed to be fixed and are therefore not subjected to bending. Consequently, the maximum bending moment in the columns is assumed to occur 2m below the top of the columns.

(iii) Huntington Approach

Rankine's active earth pressures were assumed to act over the entire rear face of the abutment. A 50% reduction in Rankine's active earth pressures was assumed on the front face of the columns.

(iv) Retaining Wall Approach

Rankine's active earth pressures were assumed to act over the gross rear width of the abutment.

(v) Proposed Modified Design Approach

A pressure distribution on the rear of the capping beam has been predicted using Ingold's approach for compaction of cohesionless soil using a Stothert and Pitt Vibroll T182A roller against a yielding structure. At-rest pressures corresponding to 9.4H were assumed to act on the front face of the columns. The friction of the soil on the sides of the columns was estimated by assuming that at-rest pressures act normal to the columns with the coefficient of friction, between the concrete and the soil, equal to 0.59, as determined from the work of Potyondy (1961).

(vi) Measured Wisley Earth Pressures

The pressure has been assumed to vary linearly between successive instrument positions forming a profile of the pressures. The contribution from the side friction on the columns was allowed for in a similar manner to that used in the modified design approach mentioned above.
The resulting values of the maximum longitudinal bending moments for designing the bending resistance of the columns as caused by the earth pressures only, are presented in Table 4.7. In all but the Hampshire Sub-Unit design approach, the maximum bending moments are assumed to be created at the column root. It can be seen that the Chettoe and Adams, Huntington and Hampshire Sub-Unit approaches all underestimate the bending moments predicted from the Wisley earth pressure data. Only the gross active assumption for the A abutment gave similar values, although they were calculated using totally incorrect assumptions. The modified approach, based on the observations at Wisley, estimated quite closely the bending moments for the A abutment. However, the large pressures caused on the rear of the capping beam of the E abutment, as a result of backfilling in an adjacent trench, caused all of the methods to underestimate the bending moments in the E abutment columns based on the Wisley data. This is because none of the methods were designed to cater for the adverse effects of construction.

<table>
<thead>
<tr>
<th>Column</th>
<th>Design Approach</th>
<th>Gross Active</th>
<th>Modified</th>
<th>From Wisley Data</th>
</tr>
</thead>
<tbody>
<tr>
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<td>Chettoe &amp; Adams</td>
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<td>191</td>
<td>1146</td>
</tr>
<tr>
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<td>Hants Sub Unit</td>
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<tr>
<td>E3</td>
<td>Gross Active</td>
<td>1418</td>
<td>236</td>
<td>1130</td>
</tr>
</tbody>
</table>

Table 4.7 Comparison of maximum longitudinal design bending moments (kNm) for columns, as determined from various assumptions of earth pressure distribution

The influence of the method of backfilling is very hard to predict during the design stage of an abutment. If the maximum estimate
of pressure was to be used for all designs, then it would result in a
gross overestimate of the required column section. It would
therefore seem more sensible to enforce strict controls on the
method of compaction, so that the risk of developing extremely high
pressures, as were observed on the Wisley E abutment, can be
minimised.

If this is the case, then it is felt that an approach similar to the
modified approach used above, may well produce a more accurate
estimate of the bending effects caused by earth pressures.

In addition to estimating the bending moment at the column root, it
may also be desirable to estimate values farther up the columns so
that a reduction in reinforcing steel can be applied. A comparison
of the bending moment profile calculated from the Wisley data with
that estimated using the modified approach is shown in Figure 4.65.
It is evident that this modified method would have resulted in a
more accurate design of the Wisley abutments than was obtained
from the original analysis based on the Chettoe and Adams
approach. Furthermore, if the bending moments due to the deck
reaction are added to those caused by the earth pressures, it can be
seen that the design bending moments closely estimate those
actually measured at Wisley.

Although the Wisley study has shown that the propping force from
the deck slab was a significant contribution to the stability of the
abutments, it cannot be assumed that this will be the case for all
spillthrough abutments. Consequently, even if similar methods of
construction are used to those adopted at Wisley (ie no temporary
propping between the deck slab and the abutment during the
construction of the embankment behind the capping beam), it is not
safe to assume that the abutment rotation will necessarily be
sufficient to cause it to come into contact with the end of the deck
slab, as this will be highly dependent on the soil properties and the
geometry of the abutment. If contact does not occur between the
end of the deck slab and the abutment, then the bending moments
at the columns roots would be considerably larger than those
measured at Wisley because the bearing friction alone would not
provide a significant resistance to the bending of the abutment caused by the earth pressures. This of course assumes that unlike Wisley, the bearings would have been undamaged due to the method of construction or otherwise. Therefore, from the point of view of a designer, it would be safer to consider the conventional deck reaction caused by bearing friction (ie 4% of dead load) and assume that the bending moments in the columns are caused by earth pressures estimated from the modified design approach that has been presented within this thesis. As a result, the main tension reinforcing steel would need to be positioned towards the rear face of the columns. This would provide an adequate design even if the abutment became propped by the deck slab because, as at Wisley, the bending of the columns under these conditions was still found to cause a tensile stress in the rear face. This was because the closure of the expansion joint was a result of the large compaction pressures on the rear of the capping beam which had caused the abutment to flex and rotate forwards at the top. However, it should be noted that the deck pours at Wisley caused the top of the abutment to rotate in the opposite sense, thus creating a tensile stress in the front face of the columns. Clearly, a loading such as this must be catered for by providing tension reinforcing steel in the front face. Similarly, if as has been suggested, a temporary prop was to be introduced during the period of compacting the backfill behind the capping beam, then the resultant earth pressures on the rear face of the structure at this time would also cause the columns to bend, such that a tensile stress would be created at the front face.

For abutments which are similar in design to those used at Wisley, it is comforting to know that an underestimate of the earth pressures is unlikely to induce a catastrophic failure of the structure. This is because the earth pressures on the rear of the capping beam will cause the abutment to rotate forwards but the movement will only continue until the expansion joint has closed sufficiently to introduce a propping force from the deck. The most likely cause of failure would therefore be the cracking of the columns towards the base but it is possible that this may never be evident at the surface. Even so, if the existing methods continue to
be adopted without further thought on the likely action of an abutment, it is possible that serious damage could occur to other parts of the structure. For instance, an incorrect assumption of the earth pressures could lead to an inadequate design of the wing walls, the curtain wall, the capping beam or the base slab.

In conclusion, it is hoped that the information and discussion that has been presented will provide future designers with a broader basis for producing a more appropriate design of spillthrough abutments.

4.4 Summary

The results from the full size and laboratory investigations have been presented and discussed. A number of important features have been revealed concerning the estimation of earth pressures during construction and also in the long term. The laboratory investigation has led to an improved understanding of the factors which contribute to the resistance of the soil against a laterally loaded column with a rectangular cross-section. As a result of the experimentation, it has been possible to suggest some factors which should be considered when estimating the earth pressures against spillthrough abutments in the future.

The following chapter outlines the main findings of the present study and also suggests some areas which may well benefit from additional research.
Plate 4.1  Distortion of formwork around bearing downstand due to West deck pour

Plate 4.2  Trench in backfill adjacent to capping beam of E abutment
Plate 4.3  Rupture surface in front of smooth 75mm x 122mm column

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Soil
Backfill (Placed after deck pours)

Curtain wall

Polystyrene pieces never removed
Polystyrene packing

20mm thk plywood*

Bearing plates

Capping beam

[^* Removed after deck pours]

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Pull-Through Test No. 2
Column Section : 75 x 122 mm
Surface Friction $\Delta = 21^\circ$
Column Displacement = 22.5 mm
Figure 4.49 Soil disturbance caused by translating column
Pull-Through Test No. 4
Column Section: 7.5 x 122 mm
Surface Friction $\delta = 21^\circ$
Column Displacement = 20.1 mm

Figure 4.50  Soil disturbance caused by translating column
Pull-Through Test No. 7
Column Section: 7.5 x 122 mm
Surface Friction $\delta = 30^\circ$
Column Displacement = 4.8 mm

Figure 4.51 Soil disturbance caused by translating column
Pull-Through Test No. 7
Column Section: 7.5 x 122 mm
Surface Friction $\delta = 36^\circ$
Column Displacement = 22 mm

Figure 4.52 Soil disturbance caused by translating column
Figure 4.53 Soil disturbance caused by translating column
Pull-Through Test No. 9
Column Section: 37.5 x 122 mm
Surface Friction $\delta = 38^\circ$
Column Displacement = 22 mm

Figure 4.54 Soil disturbance caused by translating column
Pull-Through Test No. 10
Column Section: 37.5 x 61 mm
Surface Friction $\delta = 38^\circ$
Column Displacement = 22.1 mm

Figure 4.55 Soil disturbance caused by translating column
Pull-Through Test No. 11
Column Section: 37.5 x 0 mm
Surface Friction $\delta = 38^\circ$
Column Displacement = 22 mm

Figure 4.56  Soil disturbance caused by translating column
Figure 4.57  Soil disturbance caused by translating column
Pull-Through Test No. 13
Column Section: 7.5 x 260 mm
Surface Friction $\delta = 38^\circ$
Column Displacement = 23.2 mm

Figure 4.58 Soil disturbance caused by translating column
Figure 4.59  Soil disturbance caused by translating column
Pull-Through Test No. 14
Column Section: 75 x 122 mm
Surface Friction $\delta = 38^\circ$
Column Displacement = 22 mm

Figure 4.60 Soil disturbance caused by translating column
Figure 4.61  Effect of base slab on column bending moments
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Figure 4.63 Column embedded in soil and subjected to lateral load of 20kgf
\[ M_B' = -\frac{W_c}{4} + \frac{W_b}{8} \left( 2 - \frac{b^2}{L^2} \right) - \frac{W_a}{60L^2} (3a^2 - 15aL + 20L^2) \]

\[ R_b' = \frac{3W_c}{4L} - \frac{W_b}{8L} \left( 6 - \frac{b^2}{L^2} \right) + \frac{W_t}{20L^3} \frac{W_a}{(5L-a)} \]

\[ R_p' = \frac{W_f}{4L} \left( 1 + \frac{3L}{4L} \right) + \frac{W_b}{8L} \left( 6 + 6b^2 + 8 \right) - \frac{W_a}{20L^3} (5L-a) \]

(a) Applied loadings

\[ P = \frac{3EIS}{L} \]

(b) Flexure

Figure 4.64 Trial analysis of longitudinal column bending
Figure 4.65 Comparison of measured and predicted bending moments using modified design approach.
CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK

This chapter describes the most important findings that have emerged from this experimental study concerning the behaviour and design of spillthrough abutments, with particular reference to the description of earth pressures. In addition, an appraisal is given of the most notable aspects of the experimental details that have been implemented during the course of this study. Finally, a series of recommendations for the directions of future work are presented. These are primarily concerned with the investigation of particular aspects of the behaviour and design of spillthrough abutments but also make suggestions about the need to improve and develop certain methods of instrumentation.

5.1 Conclusions

The most important conclusions that have emerged from this investigation of the determination of earth pressures exerted against spillthrough abutments are listed below:

(i) The earth pressures exerted on the sides of the abutment columns after placement of the backfill around them were found to exceed the at-rest pressures based on a typical design assumption of 9.4H. It is suggested that the earth pressures would be more accurately estimated using methods which account for the effects of soil compaction, such as those of Broms (1971) or Ingold (1979c). However, the uniform raising of the level of backfill around the columns created approximately balanced conditions and as such does not represent a critical condition for design.

(ii) The pouring of the capping beams caused a transverse bending of the columns due to the expansion of the concrete within the first 48 hours after casting. The measured increases in earth pressures on the column sides indicated that some resistance was offered by the soil. The frame action that was developed was found to be influenced by a propping action provided by an adjacent capping
beam. The transverse bending of the columns during this stage of construction should therefore be checked in the design calculations.

(iii) The pouring of the deck slabs was found to cause a significant lateral thrust to be applied to the top of the abutments. The thrust was created by the expansion of the concrete deck slab which was transmitted to the curtain wall via the deck formwork. This resulted in distortion of the formwork around the bearing downstands, a backward rotation of the abutment and an increased longitudinal bending of the columns. The soil was shown to provide considerable support to the abutment as the earth pressures on the rear of the columns were found to increase significantly during the first 36 hours after casting the deck. Such a load case is undesirable and should be eliminated in future designs by use of a modified formwork detail such that relative movement is permitted between the deck slab and the abutment. Furthermore, the formwork to the bearing downstands should be designed to allow the bearing plates to operate correctly during the deck pour.

(iv) The placement of the backfill behind the capping beam was found to cause a forward rotation of the abutment. This is contrary to the Chettoe and Adams (1938) design approach in which the soil is assumed to flow forwards between the columns.

(v) The resultant pressures on the capping beam after completion of the backfilling exceeded the active pressures and were more closely estimated by a method allowing for the effects of compaction, similar to that described by Ingold (1979a). The pressures were greater than at-rest at the top front face of the columns but were found to tend towards a passive value at the bottom of the rear face. The lateral pressures were found to be greatly increased, even beyond the Ingold pressures, if the soil was compacted within a trench immediately adjacent to the structure. It was therefore concluded that the existing design theories do not accurately predict the earth pressures and a modified approach has been proposed on the basis of the findings from the Wisley study.
(vi) The bending strains at the root of the columns after the construction was complete were not equivalent to those expected from the earth pressures. It was deduced that the large movement of the abutment must have caused it to come into contact with the end of the deck slab. This consequently produced a supporting reaction to oppose the resultant force created by the earth pressures on the rear of the capping beam. An additional horizontal supporting reaction was produced by a large value of bearing friction.

(vii) The abutments were found to rotate within the soil as a result of large lateral loadings applied to the upper region of the structures. The rotation occurred about a point up to 2m above the top of the base slab, thus indicating a similar type of action to that of a short rigid pile. The magnitude of lateral displacements at the top of the capping beam at no stage exceeded 14mm. This value represents $1/570$ of the abutment height and can be compared with typical ratios for developing the active and passive conditions of $1/1000$ and $1/20$ respectively. However, active conditions were not developed at the rear of the capping beam because the structure yielded progressively as the backfill was compacted behind it in successive layers.

(viii) At small lateral displacements, the frictional resistance of the soil against the side faces of a column is fully mobilised and thus contributes significantly to the total soil resistance. An improved method of evaluating the forces acting on the column sides is to assume a pressure distribution acting normal to the sides and apply a coefficient of friction for the soil/structure interface. The Chettoe and Adams (1938) approach of simply doubling the active pressure on the rear face of the columns is, firstly, inaccurate and secondly, assumes a mode of deformation which incorrectly results in the frictional force being considered to act as a cause of instability.

(ix) The earth pressures acting on the leading face of the columns were found to be greater than those calculated using a conventional two-dimensional approach as is used for retaining wall
design. This is because the limited width of the column enables the shear strength of the soil to be developed in three-dimensions.

(x) The zone of influence within the soil at the front of a column was found to increase in width with increasing displacement of the column. At ultimate conditions, the width of the zone at the surface was found to be at least ten times the width of the column. For the case of spillthrough abutments, the lateral displacements were found to be very small, thus making the effects of column interaction an unnecessary consideration for design.

(xi) The lateral pressures were found to vary considerably in the long term due to the expansion and contraction of the base slab caused by the seasonal temperature fluctuations. High lateral earth pressures were recorded on the rear of the capping beam during the summer when the expansion of the deck caused the abutments to be pushed backwards, as a result of the closure or blockage of the expansion joint. In winter, the pressures were relieved due to the deck contraction dragging the abutments forwards on the sliding bearings, which in turn caused the expansion joint to open. The pressure variations were exaggerated by the compaction effects of the traffic.

(xii) The vibrating wire pressure cells used at Wisley have all continued to perform perfectly during the first three years since being installed. The vibrating wire boundary cell with the 4.0mm thick diaphragm was found to be most suitable for use in full size monitoring of structures. It has been shown to perform linearly for pressures up to 250kPa at least and suffers only slightly from the effects of hysteresis and locked-in stresses. Also, it has a Cell Action Factor near to unity, thus reducing the effects of soil arching.

(xiii) The nuclear density probe should seriously be considered as an alternative method of measuring the density of backfill during placement. It provides a fast, accurate and simple method of density measurement and unlike the other methods, such as sand
or water replacement tests, it involves a minimum of equipment and causes minimal disturbance to the backfill. Its main disadvantage however, remains the safety aspect of using radioactive materials on a busy construction site.

(xiv) The resin impregnation technique has proved to be an accurate method of determining the density of dry sand specimens within the laboratory. Its advantages are that it causes minimal errors due to disturbance of the soil and it is capable of measuring the density throughout a completed sand specimen and not just at the surface.

5.2 Recommendations For Future Work

As a result of this experimental study, it has become apparent that further investigation is required to improve the understanding of various features of the behaviour of spillthrough abutments and also to improve and develop some of the experimental techniques used in this study. Brief outlines of the suggested courses of further work are listed below:

(i) In order to extend the findings from the instrumentation of the Wisley abutments, it is necessary to perform additional similar full size investigations, so as to eventually provide sufficient data upon which to establish a modified general design approach for spillthrough abutments. In a further full size investigation it would be necessary to repeat all the measurements that were taken for the Wisley abutments. However, this study has revealed certain areas in which the instrumentation was deficient and subsequent improvements or additions could well lead to a more complete understanding of the problem. Firstly, it would be very informative to measure the earth pressures acting on the base slab by installing pressure cells on the front and rear vertical faces and also on the underside. Secondly, a more detailed elevation of survey targets on the bridge and its abutments would produce a more accurate interpretation of the lateral and vertical displacements. Finally, an improved measurement of the profile of earth pressure could be easily obtained by using an increased number of pressure cells.
It has been mentioned in this study that it may be advantageous to prop the abutments during compaction of the backfill so as to prevent excessive bending of the columns and to reduce the lateral earth pressures after its subsequent removal. It is therefore necessary to investigate the options for providing such temporary propping and whether it can produce a beneficial reduction in earth pressures and abutment deformations.

The size and shape of the zone of influence within the soil at the front of a column could be further investigated by a series of model tests. It would be interesting to establish the effects of the column width and the column height on the width of the failure zone and to determine how the zone varies with depth. In addition, the influence of the mode of deformation of the column (i.e., translation or rotation) requires further investigation.

The effects of the base slab on the rotational characteristics of a column should be investigated for varying soil conditions and geometries of the base slab. Furthermore, it would be interesting to study the influence of the depth of embedment on the ability of a column to rotate.

The vibrating wire boundary pressure cells that were used at Wisley were found to be susceptible to the in-plane stresses of the concrete. This was due to the exposed screws on the rear of the pressure cell which were not debonded from the concrete. It is probable that a laboratory investigation could evaluate this type of action and subsequently lead to an appropriate modification.

The effects of installing a pocket of fine grained soil adjacent to the face of a pressure cell were not investigated in this study. It is likely that such a technique may well cause the cell to indicate a misleading pressure. This aspect of pressure cell performance could be investigated by a series of calibration tests with similar apparatus to that used in this study.

The resin impregnation technique that was developed was found to be very suitable for measuring the density variations within a
mass of dry sand. Further research may reveal an alternative resin which may be suitable for use in damp or saturated soils.

(viii) The nuclear density probe has been found to be a suitable method for measuring the in-situ density of freshly compacted backfill. In order to justify its use in the future it would be useful to perform a series of controlled tests to verify its accuracy. It would also be worthwhile to consider the safety aspects of using a nuclear device on site and to develop a modification such that its usage would be less restricted.
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### APPENDIX A

Details of high contrast photography for producing prints for digitising soil displacements

<table>
<thead>
<tr>
<th>Component</th>
<th>Details</th>
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<tbody>
<tr>
<td>Make of Camera</td>
<td>Nikon F2</td>
</tr>
<tr>
<td>Lens</td>
<td>Nikon Nikkor 85mm f2</td>
</tr>
<tr>
<td>Motor Wind</td>
<td>Nikon MD-2</td>
</tr>
<tr>
<td>Battery Pack</td>
<td>Nikon MB-1</td>
</tr>
<tr>
<td>Film Type</td>
<td>Kodalith Ortho Type 3 (2 ISO)</td>
</tr>
<tr>
<td>Exposure</td>
<td>1 sec at f5.6 - f8</td>
</tr>
<tr>
<td>Developing</td>
<td>Kodalith Super Orth (equal parts of A and B)</td>
</tr>
<tr>
<td>Processing</td>
<td>2 minutes at 20°C with continuous agitation. Stopped and fixed in normal way.</td>
</tr>
<tr>
<td>High Intensity Light Source</td>
<td>240v, 1000w studio projector lamp</td>
</tr>
</tbody>
</table>
APPENDIX B

Test procedure for the resin impregnation technique for determining the density of dry sand

The test procedure was as follows:-

(i) Determine mass of empty syringe (including hyperdermic needle).

(ii) Insert rod into the needle so that its point is just visible beyond the end of the needle.

(iii) Carefully push syringe and rod combined into soil mass to required depth and then remove inner rod.

(iv) Prepare resin mixture and place into sealed bag and cut bag to provide a good drip edge.

(v) Weigh bag plus resin.

(vi) Pierce bag with a fine needle repeatedly until required drip rate is achieved and continue to regulate until flow ceases.

(vii) Weigh bag plus residual resin.

(viii) After resin has soaked away from syringe, extract syringe and weigh to account for any residual resin and determine net quantity of resin introduced into the soil.

(ix) Allow 24 hours for the resin to set and then excavate the entire sample.

(x) Gently tap specimen to shake off any loose sand particles and then weigh and subtract mass of resin to determine mass of soil particles alone.

(xi) Place sample onto a wire cradle and dip into molten paraffin wax to create a watertight coating.
(xii)  Weigh sample plus wax coating and determine mass of wax and deduce its volume by dividing by its specific gravity of 0.908.

(xiii) Weigh coated sample in water at room temperature and calculate the volume of the sample alone by subtracting the volume of wax.

(xiv) Calculate density of sand.

The syringe was washed out with acetone immediately after withdrawal from the sand to permit repeated use.
APPENDIX C

Derivation of transformation equations using digitised datum points for converting coordinates from two separate photographs into a common coordinate system

Let \( n \) equal the number of datum points common to each photograph.

The angle of rotation between the two photographs using any pair of datum points is

\[
\Delta \theta_{ij} = \theta_{ij} - \theta_{ij}'
\]

\[
\Delta \theta_{ij} = \tan^{-1} \left( \frac{a_{2j} - a_{2i}}{b_{2j} - b_{2i}} \right) - \tan^{-1} \left( \frac{a_{1j} - a_{1i}}{b_{1j} - b_{1i}} \right)
\]

Averaging for all possible combinations of pairs of datum points gives an average angle of rotation of

\[
= \sum_{j=1}^{n} \sum_{i=1}^{n} \frac{\Delta \theta_{ij}}{n}
\]
Therefore, newly rotated coordinates of photograph 1 into (u,v) coordinate system of photograph 2 are

\[ a'_{1k} = a_{1k} \cos \Delta \theta - b_{1k} \sin \Delta \theta \]
\[ b'_{1k} = a_{1k} \sin \Delta \theta + b_{1k} \cos \Delta \theta \]

Each newly rotated datum point of photograph 1 is translated to produce coordinates identical to that in photograph 2 in the (u,v) coordinate system.

The translation of each datum point is

\[ \Delta u_k = a_{2k} - a'_{1k} \]
\[ \Delta v_k = b_{2k} - b'_{2k} \]

Average translation is

\[ \Delta u = \frac{1}{n} \sum_{k=1}^{n} \Delta u_k / n \]
\[ \Delta v = \frac{1}{n} \sum_{k=1}^{n} \Delta v_k / n \]

Therefore, modified coordinates of all other points on photograph 1 are given in (u,v) coordinates of photograph 2 by

\[ a''_{1k} = a_{1k} \cos \Delta \theta - b_{1k} \sin \Delta \theta + \Delta u \]
\[ b''_{1k} = a_{1k} \sin \Delta \theta + b_{1k} \cos \Delta \theta + \Delta v \]
APPENDIX D

Positions of vibrating wire and pneumatic earth pressure cells installed on spillthrough abutments at Wisley
APPENDIX E

Time-pressure plots for all vibrating wire cells
Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 4.0mm
Pressures calculated from cell readings using: SOIL calibration:
COMPLETE TIME v PRESSURE graph.
FLUID calibration:

Pressure cell number 101

Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 3.0mm
Pressures calculated from cell readings using: SOIL calibration:
COMPLETE TIME v PRESSURE graph.
FLUID calibration:

Pressure cell number 102
Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 3.0mm
Pressures calculated from cell readings using:  SOIL calibration ;
COMPLETE TIME v PRESSURE graph.  FLUID calibration ;

Pressure cell number 103

Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 3.0mm
Pressures calculated from cell readings using:  SOIL calibration ;
COMPLETE TIME v PRESSURE graph.  FLUID calibration ;

Pressure cell number 104
Pressure cell type: GAGE TECHNIQUE  
Diaphragm thickness = 2.5mm  
Pressures calculated from cell readings using: SOIL calibration  
COMPLETE TIME v PRESSURE graph  
FLUID calibration  

Pressure cell number 105

Pressure cell type: GAGE TECHNIQUE  
Diaphragm thickness = 2.5mm  
Pressures calculated from cell readings using: SOIL calibration  
COMPLETE TIME v PRESSURE graph  
FLUID calibration  

Pressure cell number 106
Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 2.5mm
Pressures calculated from cell readings using: SOIL calibration:
COMPLETE TIME v PRESSURE graph.  FLUID calibration:

Pressure cell number 107

Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 3.0mm
Pressures calculated from cell readings using: SOIL calibration:
COMPLETE TIME v PRESSURE graph.  FLUID calibration:

Pressure cell number 108
Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 2.5mm
Pressures calculated from cell readings using: SOIL calibration;
COMPLETE TIME v PRESSURE graph.

Pressure cell type: GAGE TECHNIQUE
Diaphragm thickness = 2.5mm
Pressures calculated from cell readings using: SOIL calibration;
COMPLETE TIME v PRESSURE graph.

Pressure cell number 109

Pressure cell number 110
Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 2.5mm  
Pressures calculated from cell readings using: SOIL calibration  
COMPLETE TIME v PRESSURE graph.  
FLUID calibration:  

Pressure cell number 111

Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 2.5mm  
Pressures calculated from cell readings using: SOIL calibration  
COMPLETE TIME v PRESSURE graph.  
FLUID calibration:  

Pressure cell number 112
Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 2.5mm
Pressures calculated from cell readings using: SOIL calibration:
COMPLETE TIME v PRESSURE graph.
FLUID calibration:

Pressure cell number 113

Pressure cell type: MAIHAK MDS 18  Pressure range = 0 - 2 bar
Pressures calculated from cell readings using: SOIL calibration:
COMPLETE TIME v PRESSURE graph.
FLUID calibration:

Pressure cell number 114
Pressure cell type: MAIHAK MDS 78  Pressure range = 0 - 2 bar
Pressures calculated from cell readings using: SOIL calibration:
COMPLETE TIME v PRESSURE graph.  FLUID calibration:

Pressure cell number 115

Pressure cell type: MAIHAK MDS 78  Pressure range = 0 - 2 bar
Pressures calculated from cell readings using: SOIL calibration:
COMPLETE TIME v PRESSURE graph.  FLUID calibration:

Pressure cell number 116
Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 4.0mm
Pressures calculated from cell readings using: SOIL calibration:
COMPLETE TIME v PRESSURE graph.

FLUID calibration:

Pressure cell number 117

Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 4.0mm
Pressures calculated from cell readings using: SOIL calibration:
COMPLETE TIME v PRESSURE graph.

FLUID calibration:

Pressure cell number 118
Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 3.0mm
Pressures calculated from cell readings using: SOIL calibration
COMPLETE TIME v PRESSURE graph.

Pressure cell number 119

Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 3.0mm
Pressures calculated from cell readings using: SOIL calibration
COMPLETE TIME v PRESSURE graph.

Pressure cell number 120
Pressure cell type: CAGE TECHNIQUE  
Diaphragm thickness = 3.0mm  
Pressures calculated from cell readings using: SOIL calibration:  
COMPLETE TIME v PRESSURE graph.  
FLUID calibration:  

Pressure cell number 121

Pressure cell type: CAGE TECHNIQUE  
Diaphragm thickness = 2.5mm  
Pressures calculated from cell readings using: SOIL calibration:  
COMPLETE TIME v PRESSURE graph.  
FLUID calibration:  

Pressure cell number 122
Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 2.5mm
Pressures calculated from cell readings using: SOIL calibration
COMPLETE TIME v PRESSURE graph.
FLUID calibration:

Pressure cell number 123

Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 2.5mm
Pressures calculated from cell readings using: SOIL calibration
COMPLETE TIME v PRESSURE graph.
FLUID calibration:

Pressure cell number 124
Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 3.0mm
Pressures calculated from cell readings using: SOIL calibration:
COMPLETE TIME v PRESSURE graph.

Pressure cell number 125

Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 2.5mm
Pressures calculated from cell readings using: SOIL calibration:
COMPLETE TIME v PRESSURE graph.

Pressure cell number 126
Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 2.5mm
Pressures calculated from cell readings using: SOIL calibration: 
COMPLETE TIME v PRESSURE graph.
FLUID calibration: 

Pressure cell number 127

Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 2.5mm
Pressures calculated from cell readings using: SOIL calibration:
COMPLETE TIME v PRESSURE graph.
FLUID calibration: 

Pressure cell number 128
Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 2.5mm  
Pressures calculated from cell readings using: SOIL calibration: ——-
COMPLETE TIME v PRESSURE graph.
FLUID calibration: ——— 

Pressure cell number 129

Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 2.5mm  
Pressures calculated from cell readings using: SOIL calibration: ——-
COMPLETE TIME v PRESSURE graph.
FLUID calibration: ——— 

Pressure cell number 130
Pressure cell type: MAIHAK MDS 78  Pressure range = 0 - 2 bar
Pressures calculated from cell readings using: SOIL calibration: ———
COMPLETE TIME v PRESSURE graph. FLUID calibration: ———

Pressure cell number 131

Pressure cell type: MAIHAK MDS 78  Pressure range = 0 - 2 bar
Pressures calculated from cell readings using: SOIL calibration: ———
COMPLETE TIME v PRESSURE graph. FLUID calibration: ———

Pressure cell number 132
Pressure cell type: MAIHAK MDS 78  Pressure range = 0 - 2 bar
Pressures calculated from cell readings using: SOIL calibration :  
COMPLET TIME v PRESSURE graph.  
FLUID calibration :  

Pressure cell number 133

Pressure cell type: CAGE TECHNIQUE  Diaphragm thickness = 4.0mm
Pressures calculated from cell readings using: SOIL calibration :  
COMPLET TIME v PRESSURE graph.  
FLUID calibration :  

Pressure cell number 134
Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 3.0mm
Pressures calculated from cell readings using: SOIL calibration:
COMPLETE TIME v PRESSURE graph.
FLUID calibration:

Pressure cell number 135

Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 2.8mm
Pressures calculated from cell readings using: SOIL calibration:
COMPLETE TIME v PRESSURE graph.
FLUID calibration:

Pressure cell number 136
Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 3.0mm
Pressures calculated from cell readings using: SOIL calibration: COMPLETE TIME v PRESSURE graph.
FLUID calibration: 

Pressure cell number 137

Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 2.5mm
Pressures calculated from cell readings using: SOIL calibration: COMPLETE TIME v PRESSURE graph.
FLUID calibration: 

Pressure cell number 138
Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 2.5mm
Pressures calculated from cell readings using: SOIL calibration:
COMPLETE TIME v PRESSURE graph.

Pressure cell number 139

Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 2.5mm
Pressures calculated from cell readings using: SOIL calibration:
COMPLETE TIME v PRESSURE graph.

Pressure cell number 140
Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 3.0mm
Pressures calculated from cell readings using: SOIL calibration:
COMPLETE TIME v PRESSURE graph.

Pressure cell number 141

Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 2.5mm
Pressures calculated from cell readings using: SOIL calibration:
COMPLETE TIME v PRESSURE graph.

Pressure cell number 142
Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 2.5mm
Pressures calculated from cell readings using: SOIL calibration: COMPLETE TIME v PRESSURE graph. FLUID calibration:

Pressure cell number 143

Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 2.5mm
Pressures calculated from cell readings using: SOIL calibration: COMPLETE TIME v PRESSURE graph. FLUID calibration:

Pressure cell number 144
Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 2.5mm
Pressures calculated from cell readings using: SOIL calibration
COMPLETE TIME v PRESSURE graph.

Pressure cell number 145

Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 2.5mm
Pressures calculated from cell readings using: SOIL calibration
COMPLETE TIME v PRESSURE graph.

Pressure cell number 146
Pressure cell type: MAIHAK MDS 78. Pressure range = 0 - 2 bar
Pressures calculated from cell readings using: SOIL calibration:
COMPLETE TIME v PRESSURE graph.

Pressure cell number 147

Pressure cell type: MAIHAK MDS 78. Pressure range = 0 - 2 bar
Pressures calculated from cell readings using: SOIL calibration:
COMPLETE TIME v PRESSURE graph.

Pressure cell number 148
Pressure cell type: MAIHAK MDS 78  
Pressure range = 0 - 2 bar
Pressures calculated from cell readings using: SOIL calibration:
COMPLETE TIME v PRESSURE graph.

Pressure cell number 149

Pressure cell type: GAGE TECHNIQUE  
Diaphragm thickness = 4.0mm
Pressures calculated from cell readings using: SOIL calibration:
COMPLETE TIME v PRESSURE graph.

Pressure cell number 150
Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 4.0mm
Pressures calculated from cell readings using: SOIL calibration: COMPLETE TIME v PRESSURE graph.  FLUID calibration: 

Pressure cell number 151

Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 3.0mm
Pressures calculated from cell readings using: SOIL calibration: COMPLETE TIME v PRESSURE graph.  FLUID calibration: 

Pressure cell number 152
Pressure cell type: CAGE TECHNIQUE  Diaphragm thickness = 3.0mm  
Pressures calculated from cell readings using:  SOIL calibration:  ---  COMPLETE TIME v PRESSURE graph:  FLUID calibration:  ---

Pressure cell number 153

Pressure cell type: CAGE TECHNIQUE  Diaphragm thickness = 3.0mm  
Pressures calculated from cell readings using:  SOIL calibration:  ---  COMPLETE TIME v PRESSURE graph:  FLUID calibration:  ---

Pressure cell number 154
Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 2.5mm
Pressures calculated from cell readings using: SOIL calibration
COMPLETE TIME v PRESSURE graph.
FLUID calibration

Pressure cell number 155

Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 2.5mm
Pressures calculated from cell readings using: SOIL calibration
COMPLETE TIME v PRESSURE graph.
FLUID calibration

Pressure cell number 156
Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 2.5mm
Pressures calculated from cell readings using: SOIL calibration:
COMPLETE TIME v PRESSURE graph.  FLUID calibration:

Pressure cell number 157

Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 3.0mm
Pressures calculated from cell readings using: SOIL calibration:
COMPLETE TIME v PRESSURE graph.  FLUID calibration:

Pressure cell number 158
Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 2.5mm
Pressures calculated from cell readings using: SOIL calibration:
COMPLETE TIME v PRESSURE graph.
FLUID calibration:

Pressure cell number 159

Pressure cell type: GAGE TECHNIQUE  Diaphragm thickness = 2.5mm
Pressures calculated from cell readings using: SOIL calibration:
COMPLETE TIME v PRESSURE graph.
FLUID calibration:

Pressure cell number 160
Pressure cell type: CAGE TECHNIQUE  Diaphragm thickness = 2.5mm
Pressures calculated from cell readings using:  SOIL calibration:  
COMPLETE TIME v PRESSURE graph.   FLUID calibration:

Pressure cell number 161

Pressure cell type: CAGE TECHNIQUE  Diaphragm thickness = 2.5mm
Pressures calculated from cell readings using:  SOIL calibration:  
COMPLETE TIME v PRESSURE graph.   FLUID calibration:

Pressure cell number 162
Pressure cell type: GAGE TECHNIQUE. Diaphragm thickness = 2.5mm. Pressures calculated from cell readings using: SOIL calibration:

COMPLETE TIME v PRESSURE graph.

FLUID calibration:

Pressure cell number 163

Pressure cell type: MAIHAK NDS 78. Pressure range = 0 - 2 bar. Pressures calculated from cell readings using: SOIL calibration:

COMPLETE TIME v PRESSURE graph.

FLUID calibration:

Pressure cell number 164
Pressure cell type: MAIHAK MDS 79. Pressure range = 0 - 2 bar
Pressures calculated from cell readings using: SOIL calibration
COMPLETETIME v PRESSURE graph. FLUID calibration

Pressure cell number 165

Pressure cell type: MAIHAK MDS 79. Pressure range = 0 - 2 bar
Pressures calculated from cell readings using: SOIL calibration
COMPLETETIME v PRESSURE graph. FLUID calibration

Pressure cell number 166