A New Technique for Simulation of Collapse of a Tunnel in a Drum Centrifuge

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1. Introduction

Tunnels form an important part of day-to-day life. Whether their role is for rail transport, highway traffic, as a subway, for water conveyance, sewer system, for cable transmission or housing generating plants or for general storage/disposal purposes as arise in civil engineering applications, they serve an important function. Tunnels have particular advantages over similar structures located at the surface such as protection from the weather, overall economy of function and minimal impact on the surface environment.

The analysis, design and construction of tunnels has formed the focal point of several international symposia. Several books and hundreds of papers have been written on various aspects of tunnelling. Although great feats have been achieved in the construction of tunnels, the behaviour of tunnels and the importance of soil-structure interaction in their analysis and design is far from clear. To date, use is being made of empirical methods based on limited field or experimental observations (Schmidt, 1974; Attwell, 1978; O'Reilly and New, 1982; Mair et al., 1993) in predicting either the settlement at the surface or the forces in tunnel linings. These empirical methods assume a plane strain condition either in the cross-sectional or in the longitudinal plane (Figure 1.1) whereas the problem of tunnelling is truly three-dimensional. These methods have many other limitations such as: (1) they are specific to the type of subsoil (Heath, 1995), (2) inability to take into account the interaction between the tunnel lining and the surrounding soil, and (3) inability to model the construction sequence of the tunnel.

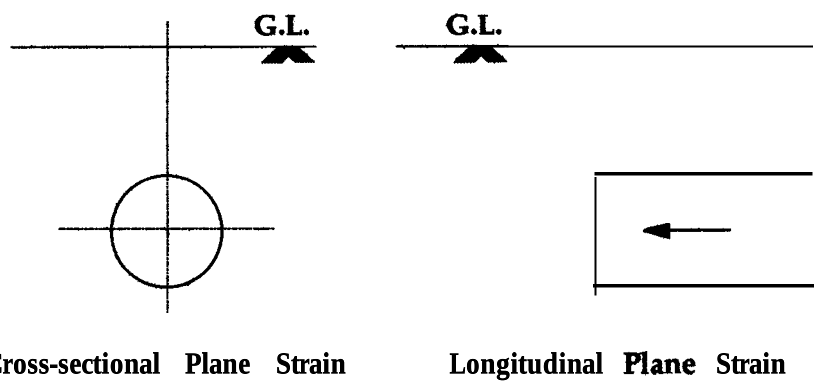

Figure 1 .1 Approximations used in the analysis of tunnel construction

A better way of analysing and designing tunnels is to use numerical methods such as the finite element method (FEM) or the boundary element method (BEM). The FEM is clearly more popular of the two. It is possible to model the soil-structure interaction and the construction sequence using the FEM. The true three-dimensional behaviour of tunnels can also be modelled. There are many examples of the use of the FEM in tunnel analysis and design (Mair et al., 1981; Rowe and Lee, 1989; Swoboda et al., 1989; Lee and Rowe, 1990; Rowe and Lee, 1992, Leca and Clough, 1992; Atzl and Mayr, 1994; Chen and Baldauf, 1994; Akagi, 1994). However, most of these examples have used the assumption of plane strain condition either in the cross-sectional plane or in the longitudinal plane. This is due to the fact that a true three-dimensional finite element analysis can be extremely time consuming and expensive in term of computing resources used. For example, the three-dimensional finite element modelling of shield tunnelling carried out by Akagi (1994) on a Fujitsu supercomputer took more time to run than the actual shield tunnelling it was trying to model. before any finite element program can be used as a design tool, it needs to be calibrated against some good quality experimental or field data.

There are many instances of instrumented projects or field trials on tunnels (e.g. Attwell and Farmer, 1974; Rowe and Kack, 1983; Harris et al., 1994; Deane & Bassett, 1995). However, the results of the field trials or instrumented projects are notoriously difficult to interpret. They tend to suffer from poor quality control at the site, hazardous environment causing malfunctioning of instrumentation, variable site conditions and high cost involved. It is almost impossible at times to correlate the results of two field trials carried out at different sites. Hence, sometimes they do not necessarily contribute towards improving our understanding of the behaviour of tunnels and may not be very useful from the point of view of calibrating new design methods. Many of the limitations of field trials can be overcome if the technique of centrifuge modelling is used in exploring the behaviour of tunnels.

Centrifuge modelling, because of its ability to reproduce the same stress levels in a small-scale model as in a full-scale prototype, can be used in assessing the behaviour of any soil structure (Schofield, 1980). Furthermore, it offers the advantages of smaller size, ease of management, greater control over the entire event, shorter consolidation time-scale and the option to continue the test up to failure. In the past, the centrifuge has contributed significantly towards improving the understanding of the behaviour of geotechnical structures leading to better design methods (e.g. Mair, 1979; Springman, 1989; Sun, 1990). Centrifuge modelling of tunnels has been very popular since the 1970s. Several examples of it can be found in the literature (Cairncross, 1973; Orr, 1976; Potts, 1977; Mair, 1979; Seneviratne, 1979; Taylor, 1984; Bolton et al. 1994; Nomoto et al. 1994; Yoshimura, 1994). Most of the tests were carried out under plane strain conditions with the exception of Mair (1979), Bolton et al. (1994), Nomoto et al. (1994) and Yoshimura (1994). All these centrifuge tests were carried out using a beam centrifuge and a strongbox of limited dimensions. Therefore, the behaviour of model tunnels may have been influenced by boundary effects.

It can be seen from these examples that there is a general lack of data on three-dimensional simulation of the tunnelling process resulting in a propagating settlement trough at the surface. Modelling the three-dimensional behaviour of tunnel construction is of paramount importance from the point of view of accurate prediction of surface settlements. Dasari (1995), after critically comparing the prediction of surface settlements from two-dimensional plane strain and three-dimensional finite element analyses, point out that the difference in magnitude of predicted surface settlements can be significant (settlements predicted by plane strain analysis almost three times those predicted by three-dimensional analysis). Also, the effect of tunnelling underneath a built-up area on the superstructures has not been widely investigated, the only example being that of Bezuijen and van der Shrier (1994). The present and future series of centrifuge tests on tunnels are aimed towards filling these gaps that exist in the database for tunnels. These tests have been or will be carried out using a drum centrifuge. Due to the nature of drum centrifuge modelling (described in section 2), the tests are relatively free of boundary effects.

This report describes the procedure and results of two preliminary centrifuge tests on tunnels in sand. A new technique for simulation of excavation of tunnel is developed and is described in section 2 which also gives description of the equipment and other experimental techniques. The results of the two tests are presented in section 3. Section 4 contains the description of some of the tests that have been planned to be carried out in near future.

2. Equipment and Experimental Techniques

2.1 Aims

The aims of the centrifuge model tests described here were: (a) to develop a technique for simulation of sequential segmental tunnel construction resulting in a propagating surface settlement trough, and (b) to design and commission a device for accurate measurement of the profile of the settlement trough. In addition, it was intended to use the results from these tests in deciding on what thickness of tunnel lining to be used in future tests. An ideal tunnel lining should be able to produce measurable strains output while at the same time it should deform sufficiently without buckling in order to produce a 1-4% ground loss. It is envisaged that the data obtained from these and future tests will be used in planning and execution of centrifuge model tests on compensation grouting to prevent excessive surface settlements. Also, the data generated from these tests would be helpful in calibrating analytical and design methods such as the finite element method.

It should be noted that the importance of the tests described herein is in "developing and proving the technology" and hence, the reader is advised not to draw any conclusions from the results presented here. It may be possible that the results may not be relevant from a practical point of view. However, the technical information presented in this technical report should help the reader either in replicating the experiments or in making use of the technique elsewhere.

2.2 Procurement and Development of Equipment

The general arrangement of the model for all the tests described in this report was same and is shown in Figure 2.1. A brief summary of each of the tests is given in Table 2.1

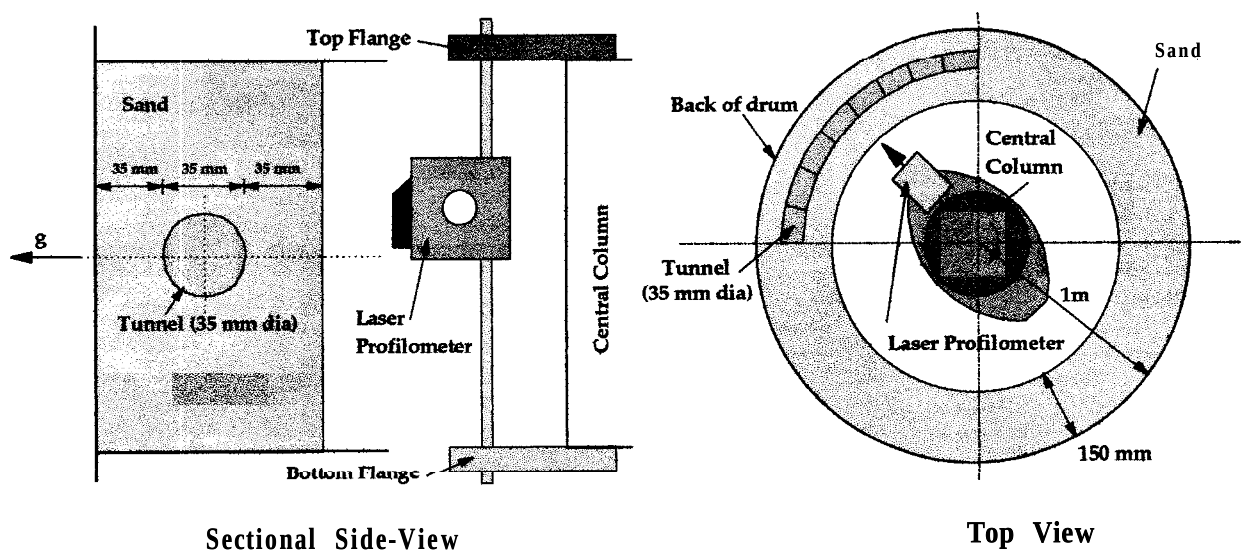

Figure 2.1 General arrangement of a typical centrifuge test

All the tests were carried out at an acceleration of 150 g at drum centrifuge wall. The system for a typical test can be divided into the following functional components:

1. CUED 2 m diameter drum centrifuge
2. Laser profilometer
3. Model tunnel sections

4. Solenoid manifold and solvent reservoir
5. Other instrumentation
   (a) Pore pressure transducers
6. Data acquisition system

In this section, each of the above-mentioned components is described.

Table 2.1 Centrifuge test programme

Test Code	Description
PRETUN1	Four tunnel sections, three unlined sections followed by a strain gauged lined section. Thickness of the brass lining 0.006" (0.1524 mm).
PRETUN2	Eight tunnel sections, two unlined sections followed by a strain gauged lined section (thickness 0.2 mm, strain gauges both on the outside and inside) followed by two unlined section followed by a strain gauged lined section (thickness 0.003" or 0.0762 mm) followed by an unlined section followed by another uninstrumented lined section (thickness 0.002" or 0.0508 mm).

2.2.1 CUED 2 m diameter drum centrifuge

Figure 2.2 shows a schematic diagram of the CUED 2 m diameter drum centrifuge.

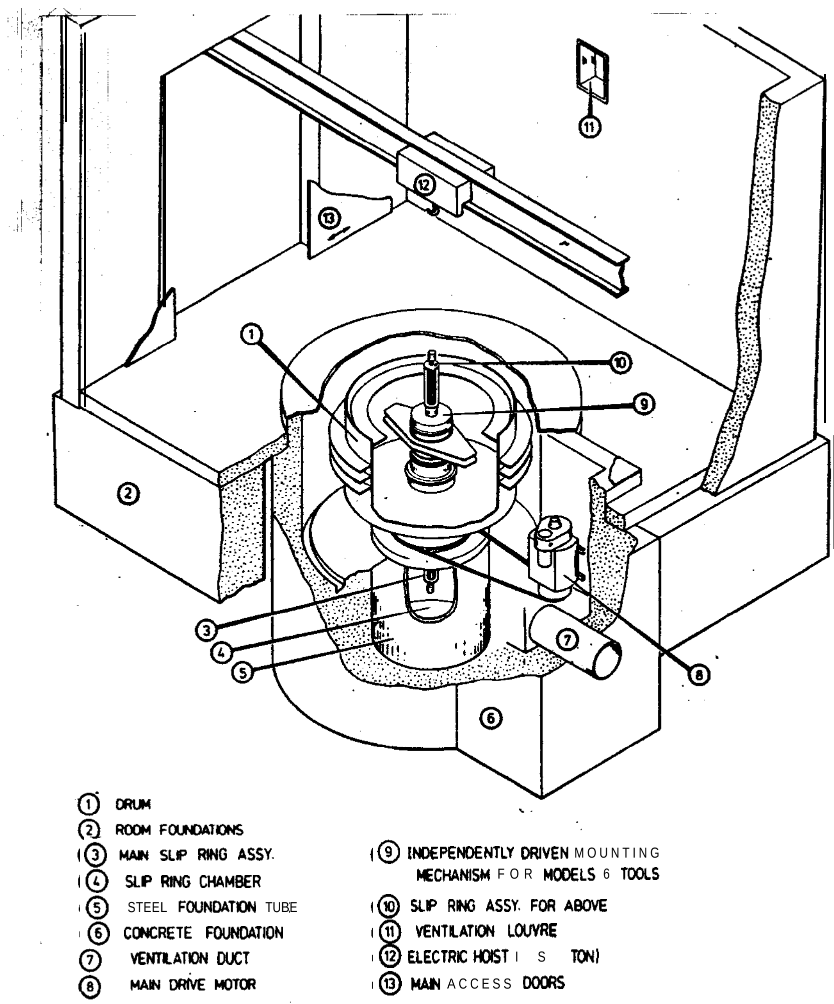

Figure 2.2 The CUED 2 m diameter drum centrifuge

The centrifuge is driven by a 100 kW d.c. motor. The centrifuge is capable of a maximum working acceleration of 500 g at the drum wall with an evenly distributed payload of 1 tonne. At prototype scale, this implies a soil specimen which is 3 km long, 0.5 km wide and 60 m deep. There is a central column in the drum to which various devices and actuators can be attached. This column rotates synchronously with the drum. However, it can also rotate differentially with respect to the drum. This facility makes it possible to do several tests at different sites without having to stop the centrifuge. The differential rotation of the central column is achieved by connecting it to a 2 kW d.c. motor via a system of gears. In the present series of test, the differential rotation capability of the central column has been used to measure the 3-D profile of the settlement trough using the laser profilometer (described later).

Before the present series of test were carried out, there was no facility to know the position of central column with respect to a fixed vertical line on the drum. Therefore, it was decided to mount a friction wheel coupled with a 10 turn rotary potentiometer at the bottom of the drum. The friction wheel is spring loaded and is in contact with the outer wall of the central column. When the central column is rotated differentially with respect to the drum, the friction wheel rotates with it. The rotation of the friction wheel is recorded by the rotary potentiometer. This facility is of utmost importance from the point of view of accurate measurement of settlement profile.

2.2.2 laser profilometer

In order to record the profile of the settlement trough accurately, it was decided to design and commission a device (called profilometer) which could be mounted with a laser displacement sensor (LDS). Figure 2.3 shows a schematic drawing of the laser profilometer.

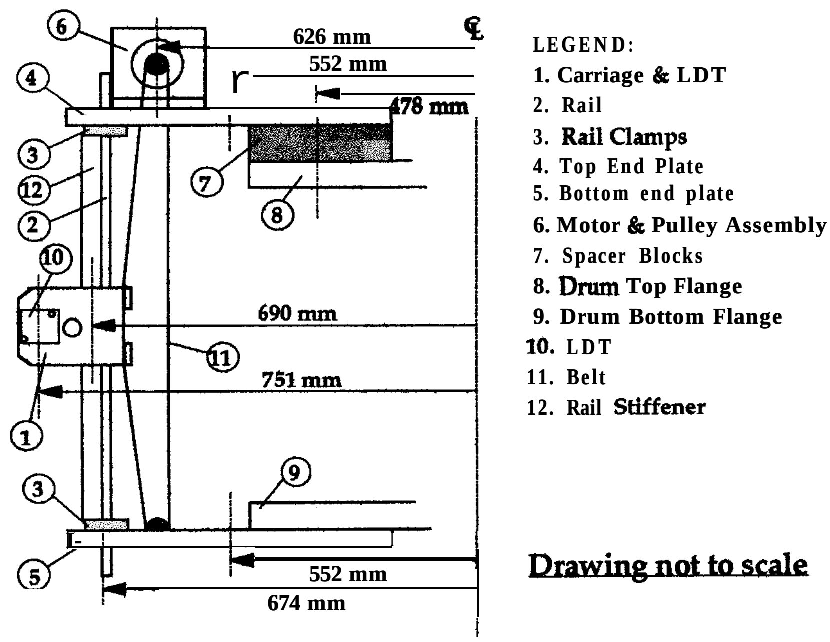

Figure 2.3 Schematic diagram of the laser profilometer

The laser profilometer consists of an LDS carriage which can roll on a vertical rail. The rail is supported at the ends by two dural plates which in turn can be supported on the flanges of the central column of the drum centrifuge. The carriage can traverse up or down the rail with the help of a timing belt and pulley system which is driven by a 12 V d.c. motor. The vertical position of the carriage is recorded by a 10 turn rotary potentiometer directly coupled with the shaft of the driving motor. The traversing of the carriage is controlled by a remote 3-way toggle switch located in the control room. The two trip switches mounted on the inside of the end plates switch off the driving motor as the carriage approaches the end plate. The direction of movement of the carriage can them be reversed by the toggle switch located in the control room.

The LDS used in the present series of tests was a Lb-72 sensor manufactured by Keyence Corporation, Osaka, Japan. The specifications of this sensor are described in detail in its instruction manual (Keyence Corp., 1994). The LB-70 LDS has a measurement range from 60 mm to 140 mm at high response speeds (maximum 700 Hz or 0.7 ms). Its resolution can be as high as 10 μm (object: white paper at a distance of 100 mm at 500 ms response speed). The LDS comes with its own junction box which has a built-in voltage regulator. The LDS was powered by a 12 V d.c. power supply. It was calibrated by pointing it at a SP200 grade beige coloured sand paper pasted on a flat perspex board while the board was gradually raised or lowered. The elevation of the top of the board from the datum was measured by a digital vernier calliper. During calibration, the response speed of the LDS was set to 20 ms. Figure 2.4 shows the calibration curve for the LDS. It can be seen from the calibration that the error in the estimation of the slope of the calibration curve is of the order of 0.2 mV/mm giving weight to the claim of high resolution by its manufacturers.

The 10 turn rotary potentiometer was calibrated by attaching the laser profilometer to a table top with the help of a G-clamp and recording the elevation of the carriage from a fixed datum (in this case the bottom end plate) with the help of a digital vernier calliper.

2.2.3 Model tunnel section

Model tunnel sections used in present series of tests were 35 mm diameter and 70 mm long cylinders as shown in Figure 2.5. At 150 g, the model section represented a 5.25 m diameter, 10.5 m long prototype section. The cores of all the model tunnel sections were cast from blocks of low density polystyrene foam. The polystyrene blocks used were similar to those used in the construction of model aeroplane wings. Figure 2.4 shows the stress-strain curve for the foam obtained by compressing a 50 mm x 50 mm x 50 mm block in an Instron MTS at a strain rate of 10% per minute.

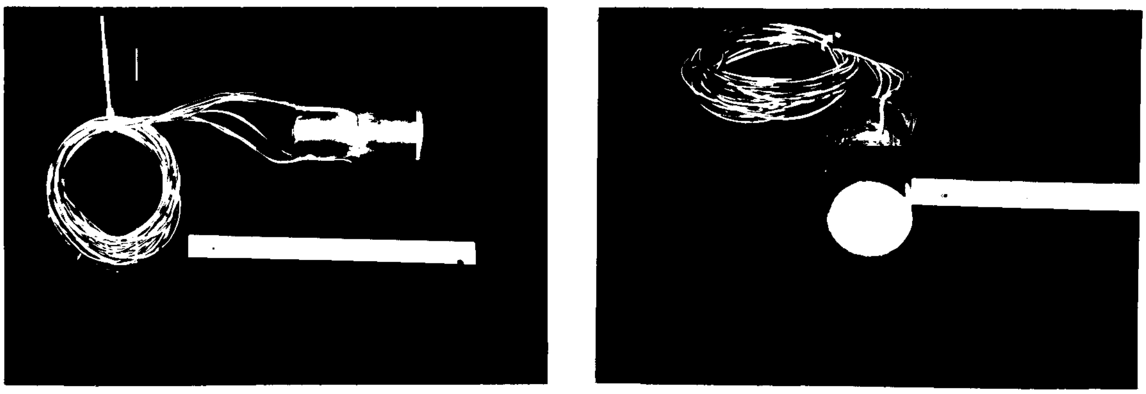

Figure 2.5 Model lined tunnel section

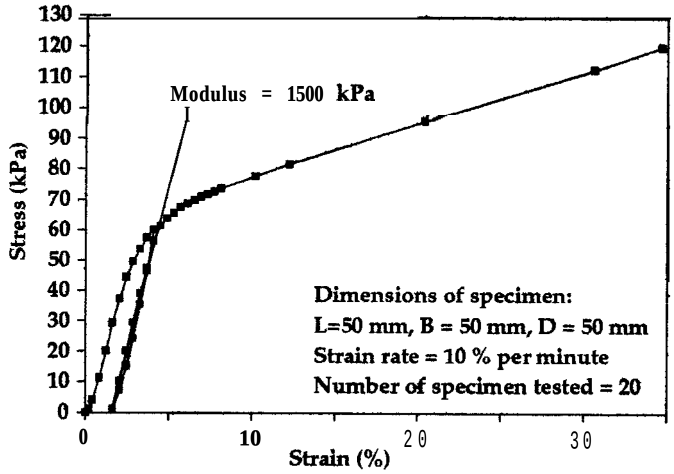

Figure 2.6 Stress-strain curve for polystyrene foam

The process of casting the cores was also similar to the one used in making model aeroplane wings. It consisted of attaching two coaxial circular guide sections (made up of stiff heat resistant material such as wood) at each end of the block. A hot wire, stretched tightly between two terminals which were approximately 1 m apart and heated electrically, was then passed over the guide sections. The hot wire "cut" the foam resulting in a cylinder of foam having approximately the same diameter as that of the guide sections. The cylinder of foam was then coated with a general purpose glue and wrapped with brown paper. The cylinder was then allowed to dry after which its outer surface was coated with wood glue to provide extra stiffness to the section. The resultant cylinder was then cut into 70 mm sections. These sections were termed as "unlined" model tunnel sections since the paper lining has negligible strength once the polystyrene core is removed.

The "lined" model tunnel sections were manufactured by wrapping a half hard brass foil around the unlined tunnel section and soldering the lap joint with the help of tin solder and an electronic soldering gun. Brass foils of different thickness were used (0.15 mm, 0.075 mm and 0.05 mm). One of the lined sections was constructed by turning down the thickness of a brass pipe from 3 mm to approximately 0.2 mm using a lathe. The polystyrene foam was then pushed into this pipe and any gaps were filled with silicone rubber sealant.

Instrumentation on tunnel lining consisted of strain gauges. The arrangement of strain gauges is shown in Figure 2.7. For the brass foil linings, strain gauges were attached only on the outside whereas for the brass pipe lining, strain gauges were attached both on the outside and inside of the lining. It should be noted that in order to calculate bending moment and thrust in the lining, strains at both outside and inside of the tunnel lining should be measured. This is because of the fact that the neutral axis is located outside the tunnel lining section. However, the aim of the tests with brass foil linings was to find out the most suitable thickness of the lining which should produce measurable strain output as well as deform enough to create 1-4% ground loss. Therefore, to make these preliminary tests economical, strain gauges were attached only on the outside of the brass foil lining. The strain gauges used were TML Cu-Ni foil type (FLA-2-350 2H-17) manufactured by Tokyo Sokki Kenkyujo Corporation Limited, Japan. These type of strain gauges have excellent self temperature compensation characteristics (-20°C to 80°C). It is important that the strain gauges mounted on thin metal foils should have good temperature compensation characteristics because the dissipation of heat generated by the passing current is poor due to small thickness of the foil. Most of the characteristics of these type of strain gauges can be found in the data sheet (TSKC Ltd., 1992). The strain gauges were covered with a silicone sealing compound to make them water-proof.

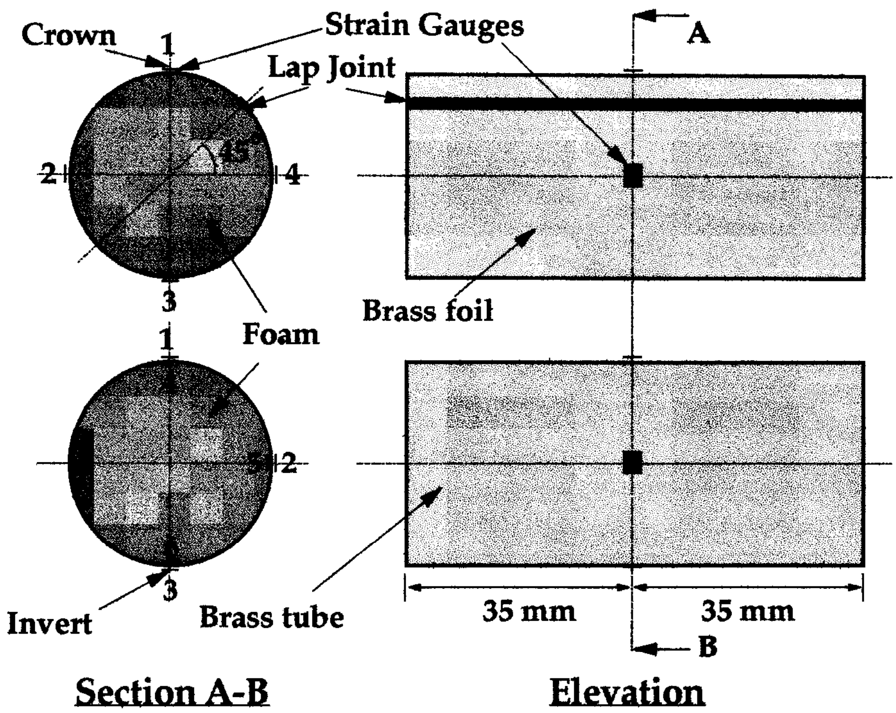

Figure 2.7 Arrangement of strain gauges on tunnel linings

After the completion of strain gauge installation, the ends of each model tunnel section were sealed by silicone rubber sealant. This was done so that each section could be collapsed independently. The ends were cured in about four hours after which a 2.5 mm diameter internal diameter flexible plastic tube was inserted from one end just below the crown of the tunnel as shown in Figure 2.8. This tube ran across the entire length of the tunnel section and had small perforations across its length inside the tunnel section. Another tube was inserted from the other end in order to ventilate the fumes produced by the process of dissolving of the foam core.

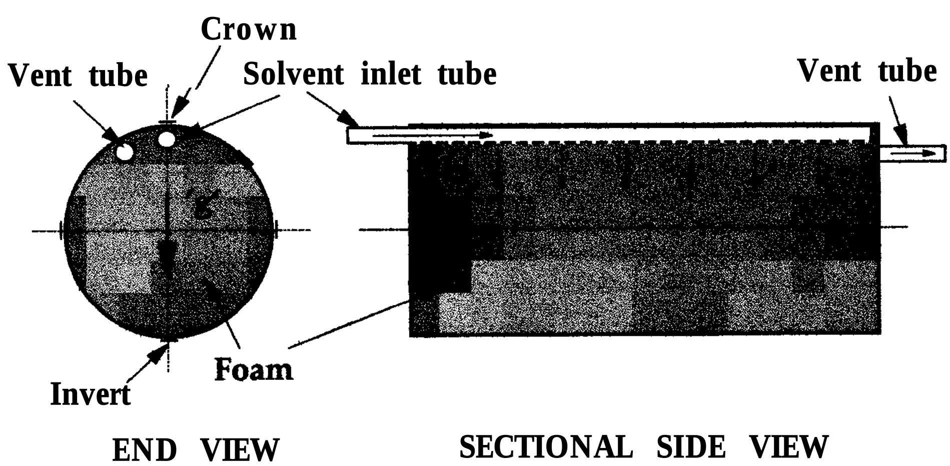

Figure 2.8 Arrangement for the inflow of the solvent into the tunnel core

2.2.4 Solenoid manifold and solvent reservoir

For the purpose of controlling the flow of solvent (required to dissolve the foam core; described in detail in section 2.3) into each model tunnel section, a manifold consisting of eight solenoid valves was built and mounted at the base of the drum. With the help of this manifold, a maximum of eight different model tunnel sections could be collapsed independently. The inflow tube attached to the model tunnel section was connected to the outlet of the solenoid valve. The inlets of all the solenoid valves were connected via a single tube to the solvent reservoir. The solenoid valves could be operated by a 24 V d.c. power supply. The power lines of each of the eight solenoid valves passed through the lower slip ring stack of the drum centrifuge and were connected to a switchboard located in the control room containing eight two-way toggle switches. Voltage was supplied to the switchboard via an auxiliary power supply unit. With this arrangement, each solenoid valve could be opened and shut from the control room. The voltage supplied to these solenoid valves was logged along with signals from the instruments so that the time at which a particular solenoid valve was opened and closed could be recorded. In future tests, it is intended to control the opening and closing of the solenoid valves using a PC via the RS232 port and a card that can convert digital code into an analogue signal.

2.2.5 Pore pressure transducers

In order to establish the elevation of ground water table using pore pressure measurements in sand, two Druck PDCR81-350 kPa miniature pore pressure transducers (ppts) were installed (one at the top of sand layer and one at the bottom of sand layer). A de-aired saturated porous stone was fitted in front of each transducer which helped in minimising the response time of the transducer to pore pressure changes and in protecting the silicone diaphragm of the transducer. The ppts were powered by a 5 V d.c. supply. The output of these transducers were amplified by a factor 10 in the junction box before it passed through the slip rings on its way to the data acquisition system located in the control room. The ppts were calibrated using a set-up which comprised of a device for applying water pressure, a junction box, a digital volt meter and a digital pressure gauge. The signals from the ppts were normalised with a power supply set to a standard 5 V.

2.2.6 Data acquisition system

A schematic diagram of the data acquisition system is shown in Figure 2.9.

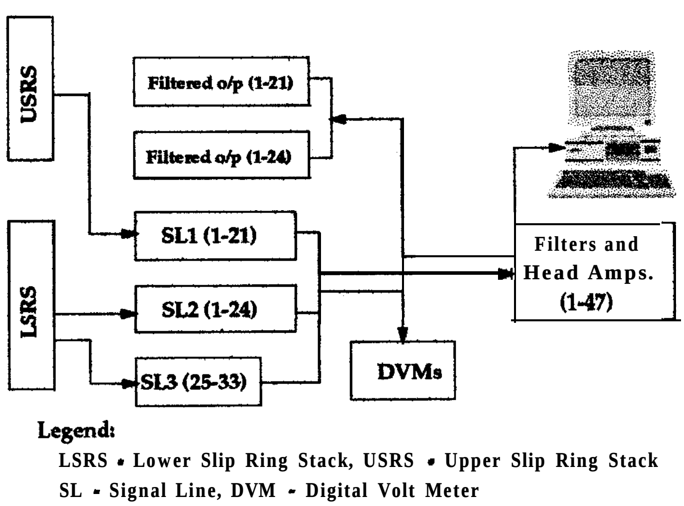

Figure 2.9 The data acquisition set-up for the drum centrifuge

All the instruments were connected to the junction boxes using either 4-way or 19-way Amphenol plugs and sockets. signals coming out of the instruments were filtered and amplified in these junction boxes using appropriate signal processing blocks. The filters in the junction boxes eliminated noise at frequencies of 1 kHz or higher. The signals coming out from the slip rings of the drum centrifuge were further filtered using filters present in the control room before they were sent to the A/D card. The filters present in the control room eliminated noise at frequencies of 2 Hz or over. Digitisation, recording and further processing of the signals was carried out using an IBM PC/AT compatible computer (HP Vectra) installed with Burr-Brown PCI data acquisition cards and LabTech NoteBook software.

2.3 Experimental techniques

2.3.1 Deposition of sand in the drum centrifuge

The Leighton-Buzzard 100/170 sand specimen (105 mm deep) was already in-place before the tests in present series. It was used for carrying out tests on single leg spud can foundations. As mentioned earlier, the tests in the present series were intended to "prove the technology" more than anything else. Therefore, it was decided to conduct them in the used sand specimen to save time and money. However, for future tests, it is intended to bury the tunnel section during the process of deposition of sand in the drum centrifuge. The actual process of deposition of sand in the drum centrifuge has been described by Tan (1990) and Dean et al. (1992) in great detail and hence it is not described here.

2.3.2 Installation of tunnel sections in sand

A clear area, relatively free of the sites used in spud can foundation tests, was selected for the installation of model tunnel sections. A faint centreline for the model tunnel section was marked on the sand surface using the top and bottom flanges of the drum as guides. A trench was then excavated using a U-shaped dredging tool made of stiff plastic (Figure 2.10).

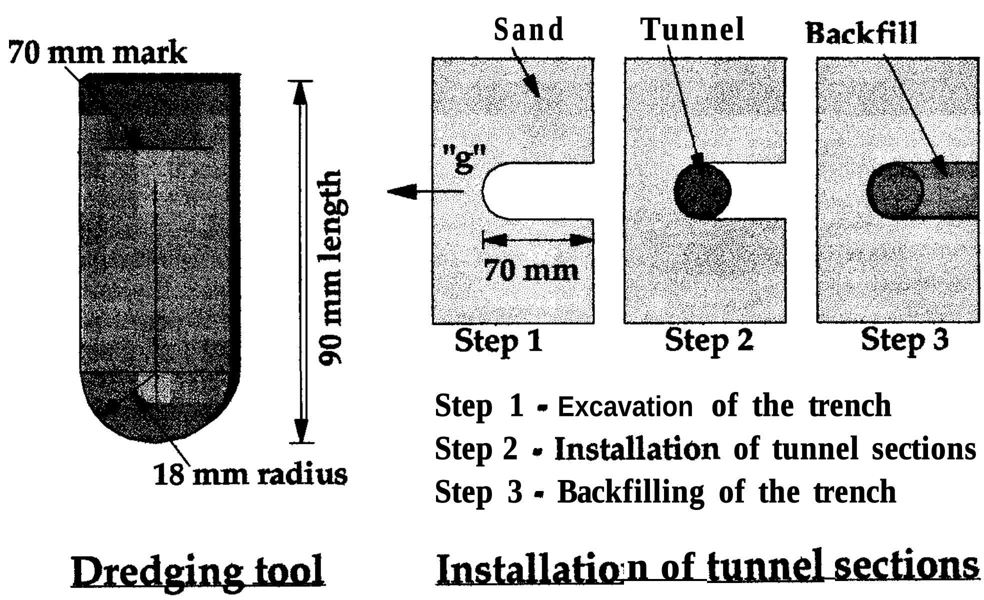

Figure 2.10 Installation of model tunnel sections in sand

The lowermost point of the trench was 70 mm deep from the sand surface. The total length of the trench along the periphery of the drum was slightly greater than the sum total of lengths of individual tunnel sections. The model tunnel sections were then placed next to each other in this trench as shown in Figure 2.10. The trench was backfilled with the excavated damp sand and compacted by hand. The flat end of the dredging tool was used to level off the surface after backfilling. The solvent inlet and vent tubes for each section were taken out near the bottom of the drum. The inlet tubes were then connected to the solenoid manifold. In case of instrumented lined sections, the cables were also taken out near the bottom of the drum and connected to the junction boxes located at the base of the drum.

2.3.3 Simulation of tunnel excavation by dissolving a polystyrene foam core

Before the commissioning of the tests described here, several trials were carried out to select appropriate material for the core of model tunnels. In the past, use has been made of heavy liquids such as zinc chloride paste by Mair, 1979 and compressed air by Britto, 1979. Such an arrangement requires the ends of the tunnels effectively sealed so that there is minimum reduction in the pressure before the simulation of the collapse. However, during the trials, it was discovered that a piece of polystyrene foam readily dissolves when it comes in contact with any organic solvent. Hence, it was decided to make the cores of the model tunnel sections from polystyrene blocks and dissolving these cores using an organic solvent during the centrifuge testing, thus simulating tunnel excavation. This technique has several advantages over the other techniques. Firstly, the ends of the model tunnel section need not be effectively sealed since there is no pressure involved. Secondly, making cores from polystyrene blocks is very easy as described above. It also facilitates excavation of one half of the core making it possible the simulation of NATM tunnelling (described in section 4). The core is stiff enough to withstand overburden pressure of soil without causing any settlement at the surface before it is dissolved.

The organic solvent used in the present series of tests was Inhibisol. Chemically, it is known as 1,1,1- Trichloroethane (CH_3CCl_3). It is also known as methyl chloroform. It is a relatively harmless substance with a fairly high flashpoint. However, it has been known to be harmful to the ozone layer and therefore, it may not be allowed to be used after 1995 following the Montreal protocol (Merck Ltd., 1995). The search for an another suitable solvent continues.

2.3.4 Measurement of the surface settlement trough

After the collapse of a model tunnel section, the profile of the settlement trough was measured using the laser profilometer. The way in which the profile was measured is shown in Figure 2.11.

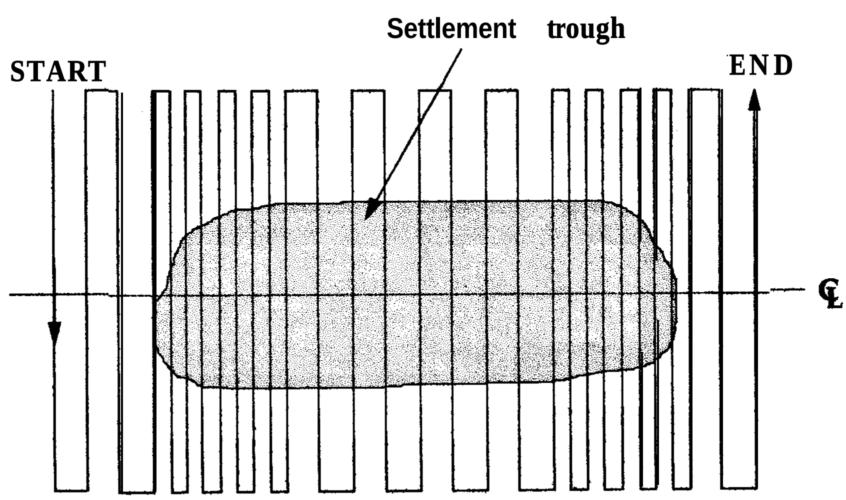

Figure 2.11 Pattern for the measurement of the profile of the settlement trough

The LDS on the profilometer was pointed approximately on the centreline of the tunnel. The central column was then differentially rotated from left to right at very slow speed. After completing the scanning of the profile at the centreline, the LDS was taken to the uppermost position and the central column was moved to a section just to the left of the beginning of the settlement trough. The overall scanning of the ground topology was carried out in a vertical zigzag fashion as shown in Figure 2.11. The whole operation was manually controlled from the control room using the switches and a closed circuit television connected to a video camera mounted on the central column. A complete scan took approximately 12 minutes. One vertical scan took approximately 20 to 30 seconds. For future tests, it is intended to automate the entire process by controlling the movement of both the laser profilometer and the central column using a PC.

2.4 Test Procedure

Before starting the centrifuge, the initial ground profile was scanned. The centrifuge was then started and speeded up in steps of 50 g until an acceleration of 150 g at the centre of model tunnel sections was achieved. At this point, another scan of the ground profile was done in order to assess any surface settlements caused by the compression of the foam core. The process of excavation of model tunnel cores was initiated by opening the first solenoid valve for approximately 2 seconds. The development of the settlement trough was observed using the CCTV system and by monitoring the output of the LDS which was pointing at the ground directly above the mid-span of the tunnel section. After approximately 100 seconds, the solenoid valve was opened for another 2 seconds and the settlement trough was observed. The scanning of the surface was started when there was no appreciable change in the settlement (usually approximately 400 seconds after the first opening of the solenoid valve). Similar procedure was observed for all the tunnel sections.

At the end of scanning for the last tunnel section, the saturation of the sand specimen was started by opening the water inlet valve. During the saturation, the output from the ppts was monitored which reached a constant value after a steady water table was achieved. Another scan of the settlement trough was done underwater. After the scanning was complete, the dump valve was opened which allowed fast draining of water. Once the surface was free of water, another scan of the settlement trough was done. The purpose of carrying out these scans was two-fold. Firstly, to ascertain whether the LDS is capable of measuring ground profiles under water and secondly, to establish (only if the first purpose is served) whether there is any difference between the measurement of the underwater ground profile and the true ground profile.

After dumping the water, the centrifuge was slowly brought down to rest and post-test investigations were carried out. All the tunnel sections were carefully excavated and checked for the presence of any undissolved foam. The lined sections were checked for any signs of buckling. The strain gauges on the lining were also checked for any damage such as breaking of connections, damage to the waterproofing seals etc..

The data was transferred to 3.5" floppy disks so that it could be analysed using softwares such as Lotus 123, Matlab and Lotus Freelance.

3. Results

In this section, some of the results from the two tests are described. Emphasis is placed on the measurement of the profile of the settlement trough (both dry and under water), the output from the strain gauges attached to tunnel linings and the post-test investigations. The reader should note that the results are presented on model scale, i.e. the prototype results can be obtained from the model results by applying appropriate scaling factors described by Schofield (1980).

3.1 Profile of the settlement trough

For both tests, the settlement trough was visible above the unlined sections a few seconds after the injection of solvent. As mentioned above, the LDS was aimed at the ground surface immediately above the mid-section of the tunnel section during the development of settlement trough. Figure 3.1 shows a plot of surface settlement vs. time for test PRETUNl.

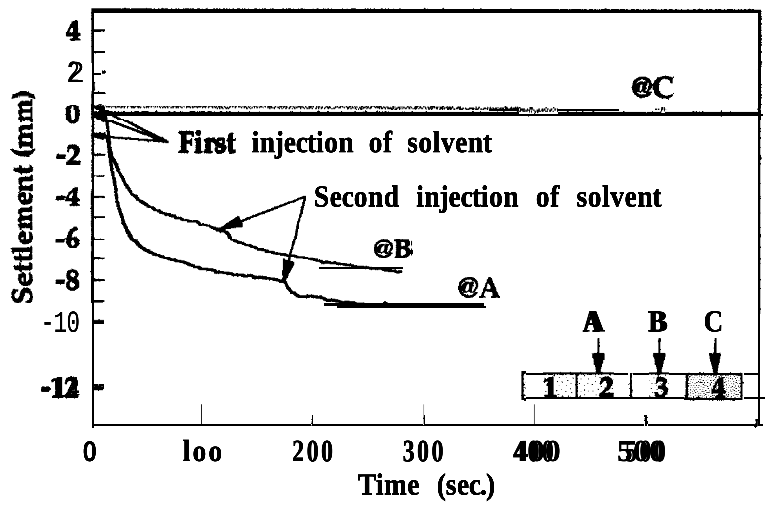

Figure 3.1 Settlements at the surface after the injection of the solvent (Test PRETUNl)

From Figure 3.1, it can be seen that the settlement of the surface was nearly complete in about 350 seconds after the first injection of the solvent (for 2 seconds). Most of the settlement was caused by first injection of the solvent. However, the second injection of the solvent (for 1 second) also caused measurable settlements. Also, there was negligible settlement above the lined tunnel section (lining thickness 0.1524 mm).

After the excavation of a section was complete (usually in about 350 seconds), a centre line scan of the settlement trough was carried out. Figures 3.2 and 3.3 show the progressive development of the settlement trough as the model tunnel sections are excavated one after another. Again, negligible settlements were recorded above the lined tunnel sections. The centre line scan was followed by a complete scan of the ground in a zigzag pattern as described in section 2. Figure 3.4 shows the threedimensional profile of the surface for test PRETUNl after the excavation of all four sections. The profile is presented as contours for test PRETUN2 in Figure 3.5. Figure 3.6 shows the settlement trough in the cross-sectional plane for test PRETUNl superimposed on the initial ground profile measured just after achieving 150 g. The dip in the initial profile can possibly be attributed to the readjustment of the tunnel section in the trench as the acceleration level increased. It is clear from these figures that the resolution of the LDS is fairly high and that it can accurately measure the profile of the settlement trough fairly quickly.

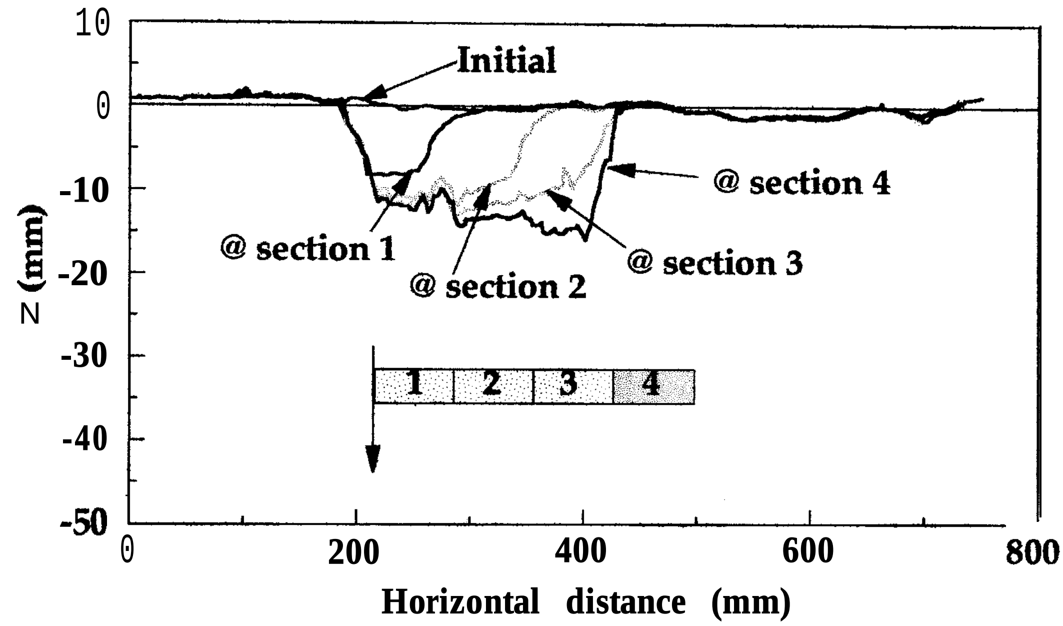

Figure 3.2 Progressive development of the settlement trough @ centreline (Test PRETUN1)

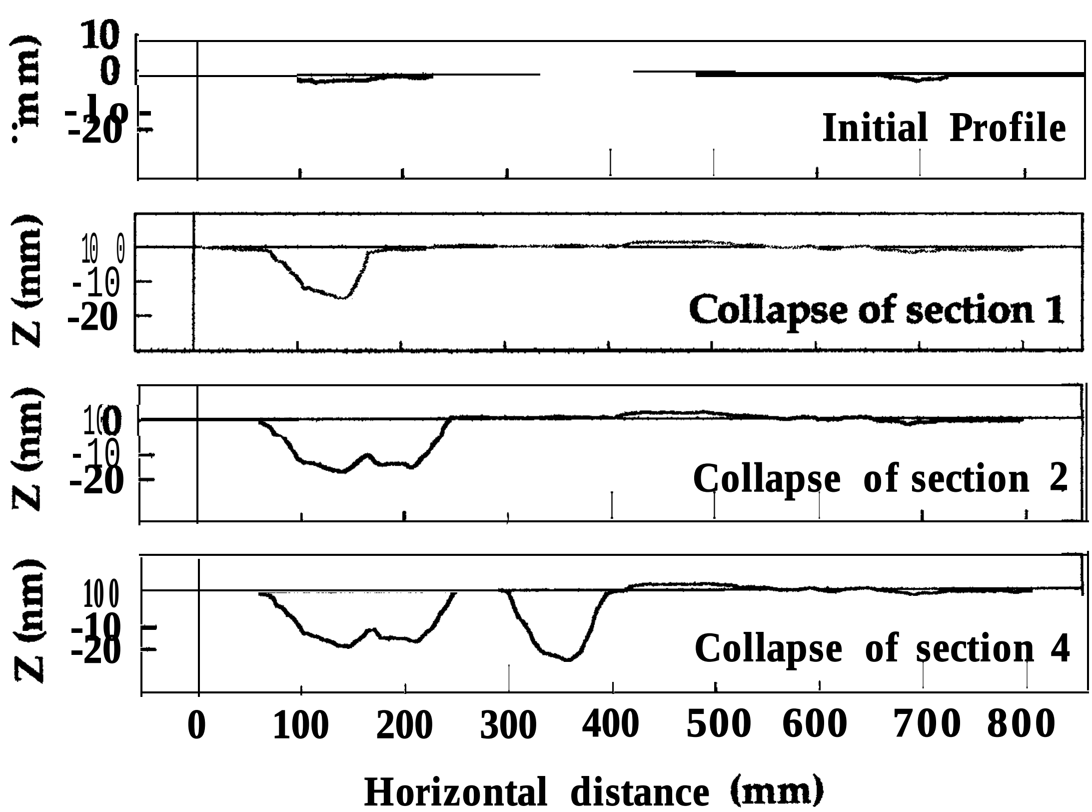

Figure 3.3 Progressive development of the settlement trough @ centreline (Test PRETUN2)

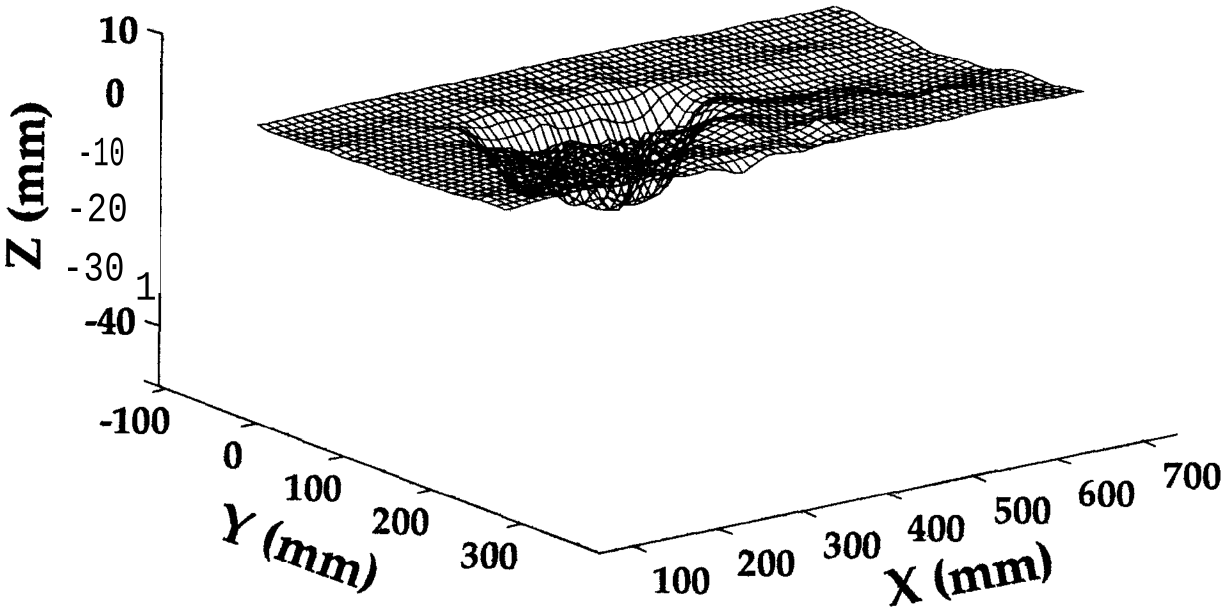

Figure 3.4 3-D presentation of the final settlement trough (Test PRETUN1)

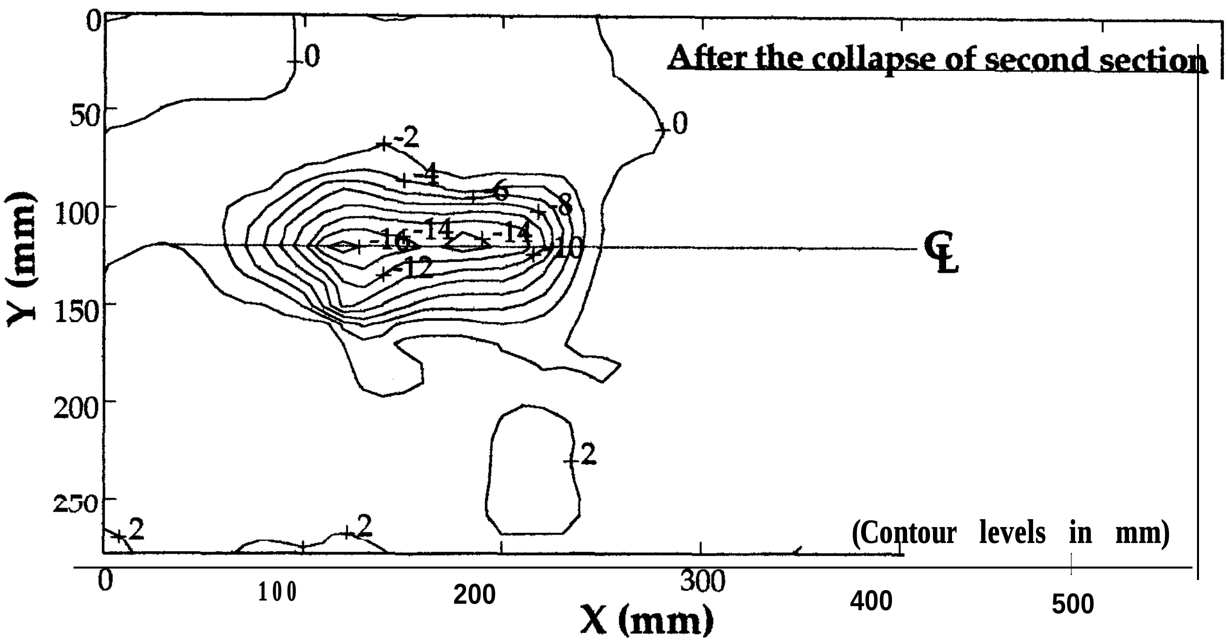

Figure 3.5 Contours of settlement trough for test PRETUN2

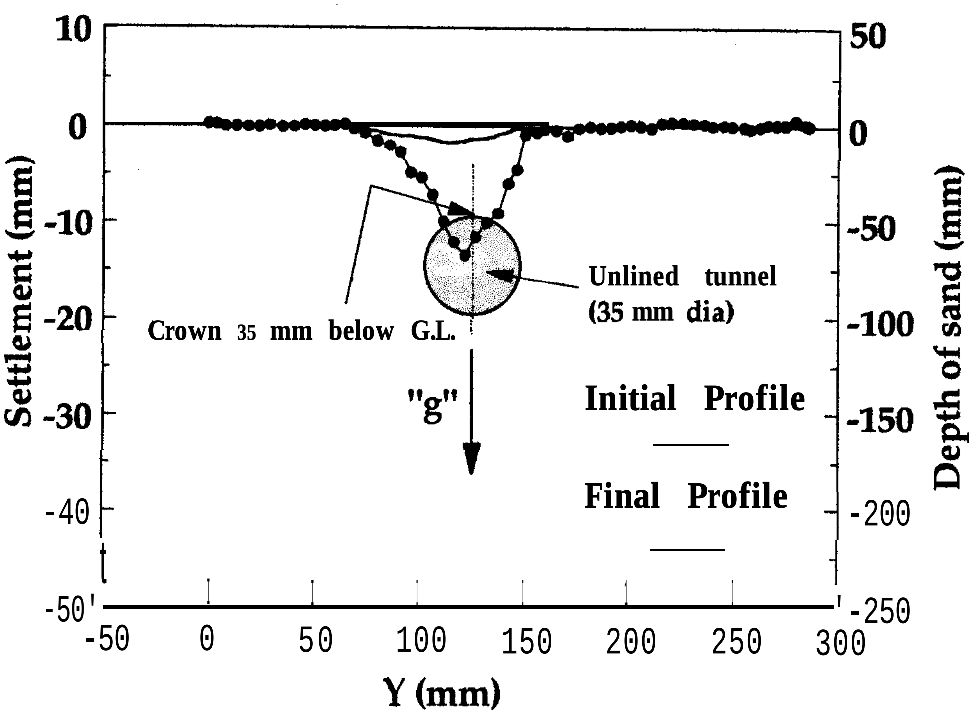

Figure 3.6 Settlement trough in cross-sectional plane (Test PRETUN1, @ mid-span of section 2)

3.2 Underwater measurement of settlement profile

Since it is intended to carry out future tests on tunnels in saturated sand rather than damp sand which was used in the present series of tests, it was decided to explore whether the LDS is able to measure the ground profile underwater. The behaviour of damp sand can be very stiff due to the presence of capillary suctions resulting in greater arching capability. The arching ability of damp sand may prevent the development of a measurable settlement trough at the surface. The evidence in support of this can be seen in Figure 3.7 which shows a picture of the buckled 0.002" lining taken after the completion of test PRETUN2. The lining was able to carry the surcharge when its core was excavated under damp sand conditions resulting in very little settlement at the surface. However, it collapsed when the sand was saturated resulting in a deep settlement trough at the surface.

Figure 3.7 Buckled 0.002" thick tunnel lining (Test PRETUN2)

As described in section 2, the sand was saturated after the end of the final ground profile scan for test PRETUN1. A steady water table was achieved with the help of an overflow pipe located about 20 mm above the surface of the sand. The height of the water table was established using the output from the ppts. A centreline scan of the settlement trough was then carried out. The dump valve was opened after the completion of the scan which allowed quick draining of the sand layer. Once the water table was significantly lower than the surface of the sand, another centreline scan of the ground profile was carried out. The results of these two scans are plotted in Figure 3.8 along with those obtained from the scan over damp sand. It can be seen that the ground profile measured underwater appears shallower than the other two profiles. The reason for this is not clearly understood yet. It may be attributed to the change in the wavelength of the laser as it passes through the water. The manufacturers of the LDS have been contacted for a satisfactory explanation of this phenomenon. There has been no reply from them at the time of producing this report. The reader is advised to keep an eye on future reports in this series for the explanation.

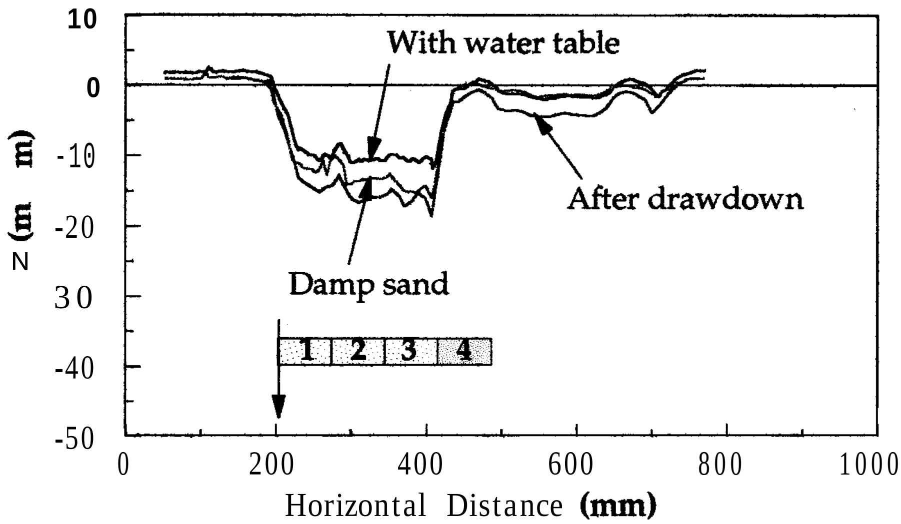

Figure 3.8 Comparison of in-air and under water measurement of ground profile (Test PRETUN1)

From the point of view of estimating the difference in the ground profile measured in air and under water as observed in test PRETUN1, a steel step gauge was placed on the surface of sand for test PRETUN2. The step gauge had several steps of 0.25 mm height and it was attached to the base of the drum so that sand deformation could be eliminated from the measurement. The profile of this step gauge was measured using the laser profilometer. After this, as in test PRETUN1, the sand was saturated and a steady water table was achieved. Similar centreline scans of the ground profile were carried out. The profile of the step gauge was also measured underwater. Figure 3.9 shows the in air and under water profiles of the step gauge. As expected, there was a significant difference between the two profiles - the under water profile being shallower. Before opening the dump valve for draining of the water, the LDS was positioned in such a way that it pointed at the mid-section of the step gauge. The dump valve was then opened and the LDS readings were recorded as the water table was being lowered. Figure 3.10 shows a plot of height of water table above the surface of the step gauge vs. the distance measured by the LDS. From Figure 3.10, it can be concluded that the discrepancy between the under water and in air measurement is higher when the height of the water table above the sand surface is higher and that there is a linear relationship between the two. The under water measurements approach the true in air measurement as the water table drops.

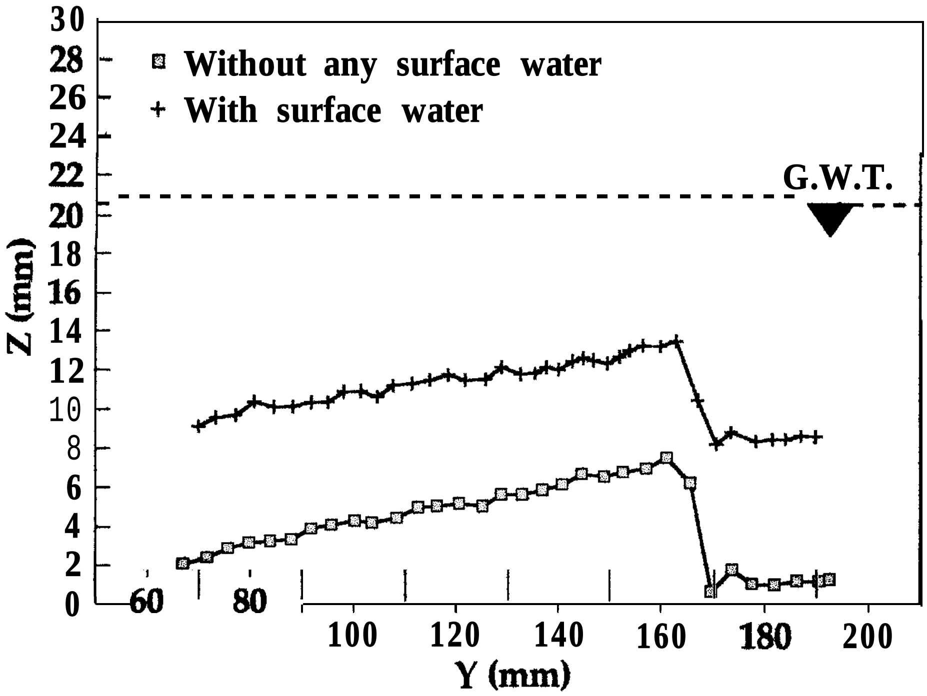

Figure 3.9 Comparison between in air and underwater measurement of step gauge profile (Test PRETUN2)

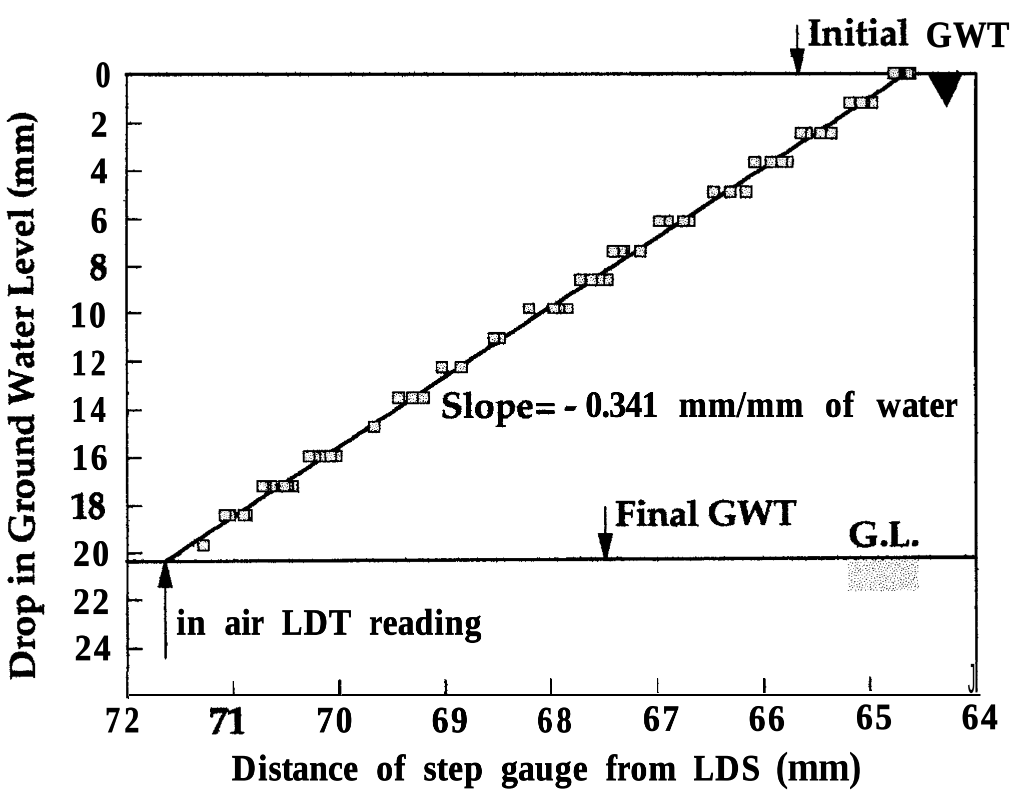

Figure 3.10 Change in the LDS reading with change in the height of water table

The linear relationship between the difference in the measurements and the height of the water table means that a correction can be applied to the under water measurements in order to get true in air measurements. The only condition is that the distance between the water table and the LDS should be known. The correction factor can be obtained as follows (refer to Figure 3.11):

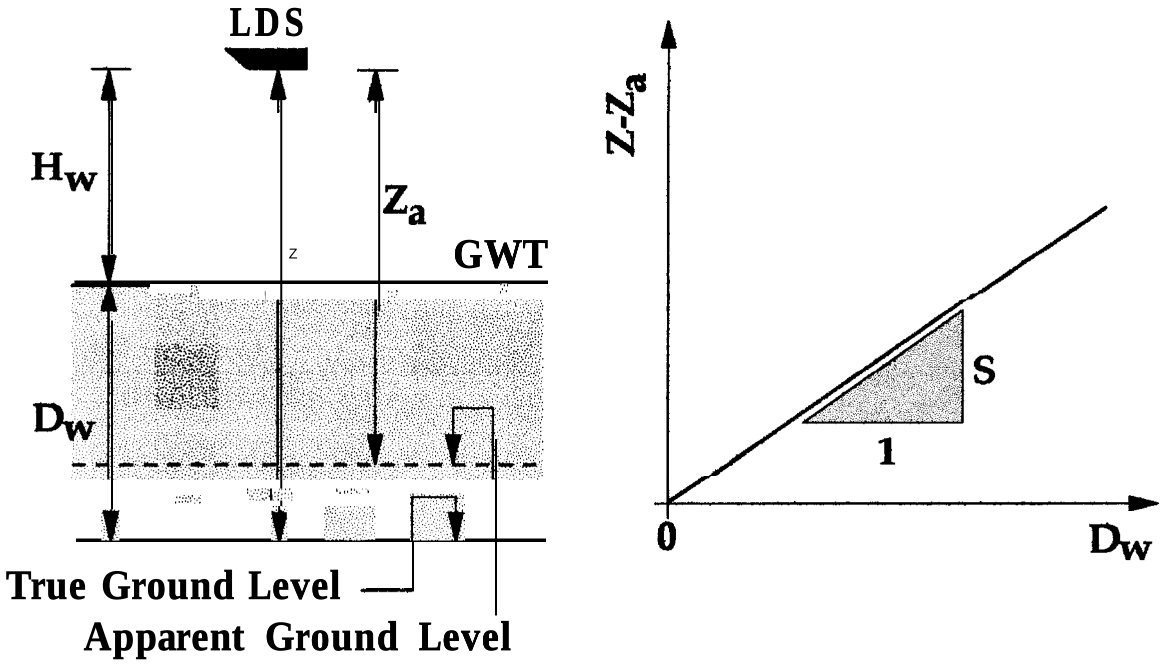

Figure 3.11 Correction factor for under water measurement of profiles

The slope of the linear relationship between the difference of the in air and under water measurements and the depth of water table above the ground surface (S) is given by:

$$S = \frac{Z - Z_a}{D_w} \tag{3.1}$$

but the depth of water (D_w) is given by:

$$D_w = Z - H_w \tag{3.2}$$

Substituting equation 3.2 in to equation 3.1 and rearranging, the following relationship can be obtained:

$$Z = \frac{Z_a - S \cdot H_w}{1 - S} \tag{3.3}$$

All the variables in the right hand side of the equation 3.3 are known. Slope S can be estimated as described above. Z_a is the apparent LDS reading and H_w, the distance of the water table from the LDS, can be computed if the height of water table above a fixed marker or object is known. Therefore, the true LDS reading, Z, can be calculated from equation 3.3.

3.3 Response of strain gauges on tunnel lining

As mentioned above, strain gauges were mounted on tunnel linings to estimate the range of voltage output for a given lining thickness so that a suitable lining thickness could be chosen for the future tests which should deform enough to produce 1-4% ground loss. Hence, in the results presented here, the voltage output from the strain gauges has not been converted into strains.

Figure 3.12 shows the response of strain gauges mounted on the outside of the lined tunnel section (section no. 4, test PRETUN1) during the swing-up to 150g and during the successive excavation of sections 1 through 4. The change in the strain gauge outputs during the swing-up may be attributed to the increase in the surcharge with increase in acceleration level. The readjustment of tunnel section may also have resulted in some change in the output. The effect of an approaching shock wave of collapsing tunnel section on the lined section is clearly seen from Figure 3.12.

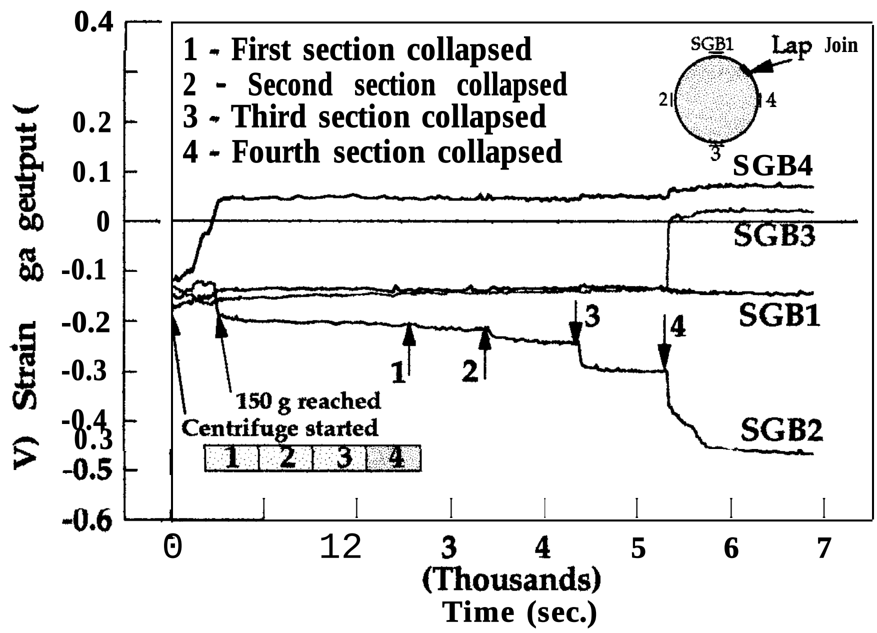

Figure 3.12 Response of strain gauges on tunnel lining (Test PRETUNI)

The change in the output got bigger and bigger as the shock wave approached the lined section, the biggest change occurring when the lined section itself was excavated. The change in outputs of strain gauges nos. 1 & 4 were much smaller than those of nos. 2 & 3. This may be due the local stiffening effect of the lap joint on the tunnel lining. Although the strain gauges on the lining (thickness gave accurately measurable outputs, the lining did not deform enough to cause the development of a measurable settlement trough. Hence, it was decided to try linings of lower thickness for test PRETUN2. The outputs of the strain gauges mounted on lined tunnel sections in test PRETUN2 were also accurately measurable.

3.4 Post-test investigations

Post-test investigations mainly consisted of excavation of excavated tunnel sections from sand and their visual examination. Examination of the unlined and lined sections for both the tests confirmed complete dissolution of the foam core. The linings for three of the four lined sections (one in test PRETUN1 and three in test PRETUN2) were found intact with no evidence of any buckling whatsoever. The thickness of these linings were 0.2 mm, 0.1524 mm and 0.075

mm. However, as mentioned above, the 0.0508 mm (0.002") thick lining buckled when the sand was saturated whereas the other three linings did not deform enough to induce a measurable settlement trough at the surface. Hence, the right thickness of the lining should be between 0.05 mm and 0.075 mm.

The visual examination of the lined sections also revealed that some of the waterproof coverings over the strain gauges were partially dissolved by the solvent. This explains the enormous drift in the output of these strain gauges when the sand was being saturated around the tunnel linings. The excess solvent inside the tunnel sections may have come in contact with the coverings after being flushed out by the rising water table, dissolving these covering and causing short circuit in the strain gauges. Learning from this experience, it has been decided to use another kind of waterproof covering which is not affected by the solvent.

4. Future research

4.1 Simulation of NATM *tunnelling*

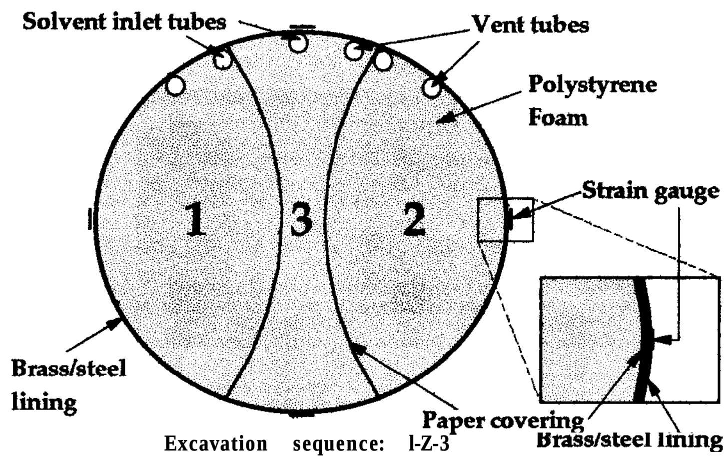

Figure 4.1 Cross-section of a model NATM tunnel

Ever since the collapse of NATM tunnels at Heathrow airport (Oliver, 1994), there is a renewed interest in the NATM technology amongst the research community. Although the problem has been widely investigated using numerical analyses (e.g. Leca and Clough, 1992, Atzl and Mayr, 1994), the 3-D nature of the excavation used in NATM makes it very difficult to model experimentally. Therefore, it is not surprising to see a lack of experimental data on various aspects of NATM such as the forces in the shotcrete lining, asymmetric settlement trough at the surface, effect of a relatively weak invert. Most of this information is available only from large scale field trials which have various limitations as mentioned in section 1.

The technique of dissolving the foam using an organic solvent described in this report can be used in the modelling of the NATM using a drum centrifuge. A typical cross-section of a model NATM tunnel section is shown in Figure 4.1. The three sections 1,2 and 3 are cast separately using the hot-wire technique described above and are glued together with paper in between two adjacent sections. The entire section is then wrapped in paper and coated with wood glue. The presence of paper between two adjacent sections makes it possible to excavate each section independently in 3-D, thus simulating NATM excavation. The lining, however, has to be installed prior to the installation of tunnel sections in soil and can be made up of thin brass or copper foil. Strain gauges can be placed at strategic locations on the lining to monitor the forces in the lining during and after excavation.

The effect of a weak invert (that can break during the excavation) on the forces in the lining and soil movements around the tunnel can also be investigated. In this case, the lap joint in the lining is located at the invert and is formed by applying a weak glue. It is also possible to explore the effect of collapse of a NATM tunnel on another NATM tunnel running parallel (situation similar to the one encountered at Heathrow Airport).

4.2 Collapse of a tunnel underneath a built-up area

Most of the tunnels are constructed under heavily built-up urban areas. In the analysis and design of these tunnels, however, a green site condition is assumed. The effect of stiff superstructures on the nature of the settlement trough can be significant (Payne, 1995). A centrifuge experiment can be planned in which a propagating wave of settlement trough caused by tunnel excavation approaches a model building (deployed in-flight) and passes underneath it. The movement of the building can be recorded by attaching displacement transducers to it. In addition, the stresses induced in the building can be recorded by mounting strain gauges at strategic locations. Various types of foundations (e.g. raft, pile etc.) can be used for the building. It is also possible to model some important structural details of the building.

5. Acknowledgement

The work reported here forms a part of an EPSRC funded research into compensation grouting. The authors wish to thank Mr. Richard Boyle for his enormous help in the development of equipment and in planning and execution of the tests described here. The authors are also grateful to Mr. Steve Chandler who carried out the strain gauging of one of the lined tunnel sections. The help and cooperation of everybody at the Geotechnical Centrifuge Centre of Cambridge University Engineering Department is highly appreciated.

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