UNIVERSITY of KENT at CANTERBURY

REPORT
on
SUBSIDENCE

JUNE 1975

PROFESSOR A. W. BISHOP,
Imperial College,
Kensington,
London.

HARRIS & SUTHERLAND,
Consulting Civil Engineers,
38-42 Whitfield Street,
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SYNOPSIS

Following the subsidence of the Cornwallis building at the University of Kent at Canterbury in July 1974, Harris & Sutherland were appointed as Engineers for the remedial works. At the same time they were asked, in conjunction with Professor Bishop of Imperial College, to investigate the cause or causes of the subsidence and to evaluate the site with regard to existing and future buildings.

It is concluded that subsidence occurred as a result of the local failure of the lining of an old railway tunnel running beneath the campus. The mechanisms of both the subsidence and the failure of the tunnel lining are examined in detail. Evidence from the soils investigation and the events of the time indicate that the ground failed in undrained shear as a single plug which moved downwards into a cavity created by local failure of the tunnel lining and surrounding clay. A mechanism for the tunnel lining failure is proposed and although the exact causes are not clear it is concluded that the tunnel walls failed in bending most probably initiated by a failure of the invert brickwork.

Further to the site investigation and the remedial grouting works, including the back filling of the tunnel, an evaluation of the site is presented. This concludes first that existing or new buildings over the backfilled sections of tunnel will not be subject to catastrophic subsidence but may be subject to differential movements which are marginally greater than would be expected elsewhere on the site; second that, with certain precautions, rebuilding over the area of subsidence can take place without delay; third that buildings adjacent to the area of subsidence have been adequately protected by the effects of the compaction grouting; and finally that no reason has been found why the remainder of the campus, remote from the tunnel, should be treated as in any way unusual.

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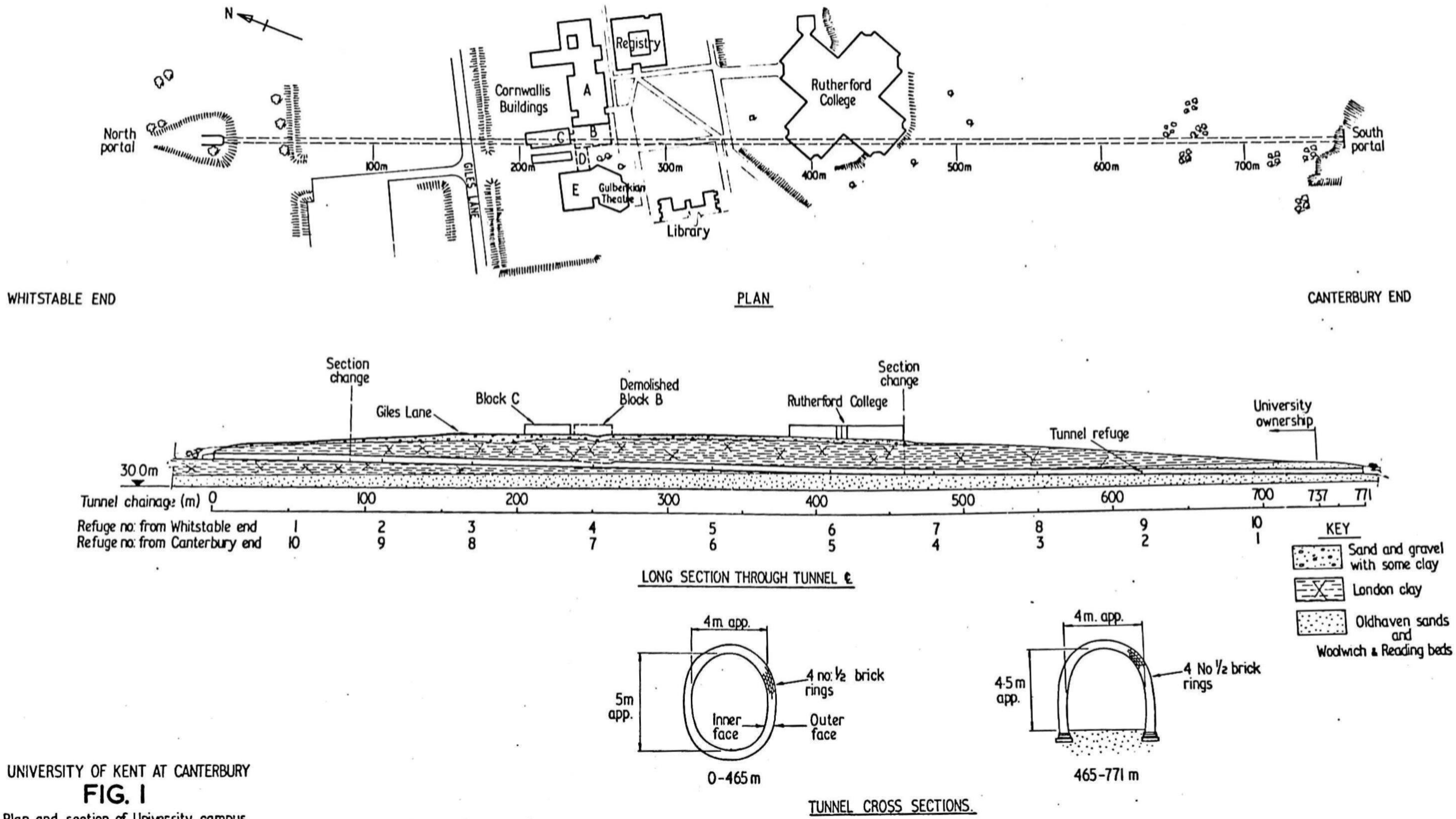


UNIVERSITY OF KENT AT CANTERBURY

FIG. 1

Plan and section of University campus

1.1 General

On 11th July 1974 Farmer and Dark, the architects and structural engineering consultants for the Cornwallis building at the University of Kent, urgently requested Harris & Sutherland to visit the University and give an opinion on several major cracks in the structure of the building. By the following day, when Harris & Sutherland visited the site, parts of the Cornwallis Block B and a link bridge, Block D, had settled by as much as 700mm. Shortly after this Professor Bishop of Imperial College was invited to a meeting with Farmer and Dark to discuss the subsidence before inspecting it on 21st July.

Blocks B and C of the Cornwallis building and Rutherford College are built over a disused tunnel on the former Canterbury to Whitstable railway line (see Fig. 1). This tunnel was built in 1827 and had been sold by British Railways to the University of Kent in 1963. The brick lining had been disintegrating over two short lengths below the Cornwallis building for several weeks prior to the major subsidence leading to a series of breaches with the ingress of clay on 3rd and 11th July. At the same time the Cornwallis building had been showing serious signs of distress, culminating in the subsidence.

Although at the time of subsidence there was no proof that a third agency had not caused both the collapse of the tunnel and the subsidence of the building, it was clear that every effort should be made to prevent the collapse of further sections of the tunnel which were located beneath the Cornwallis Block C and Rutherford College. Farmer and Dark, who had previously been negotiating a contract for gunite lining the old tunnel, had already requested Messrs. Intrusion Prepakt Ltd. to start filling the tunnel by drilling and pumping grout from the surface; beginning with the length beneath the Cornwallis Block C which was adjacent to the collapsed section.

Block B had suffered severe damage and about 70 per cent of the structure was in danger of collapse (see Fig. 1). The link bridge and the damaged portion of Block B were also causing considerable distress to adjoining buildings on which they were now imposing large unexpected loads. There was therefore an urgent need to disconnect Blocks B and D from the adjoining buildings. At this stage it was impossible to estimate how much further subsidence might take place and what area of ground this might affect. Since 30 per cent of Block B and the plant room were thus far only slightly damaged, it was considered essential to support their foundations on jacks so that in the event of further movement the buildings could be separated from their foundations and held in position by means of the underpinning. Accordingly Messrs. Pynford (Southern) Ltd. were instructed to carry out this support work.

On 23rd July 1974 Harris & Sutherland were appointed to supervise the measures already taken by Farmer and Dark and to recommend for approval and execute any measures required to restrict further damage. As an immediate action a resident engineer was put on site to supervise these works and Cementation Construction Ltd. were invited to co-ordinate the works in the role of Main Contractor.

Harris & Sutherland were also asked to investigate the soil and foundation conditions under the remainder of the Cornwallis building, Rutherford College (including the proposed extension) and the remainder of the tunnel from the south side of Rutherford College to the north side of Giles Lane, recommending for approval and executing all works found necessary to ensure stability against further subsidence. They were to report on the effect of damage to the Cornwallis building with a view to establishing the feasibility of remedial measures or demolition. Further they were asked to report on the cause or causes of the subsidence and apparently simultaneous failure of the Cornwallis building and the tunnel, in-so-far as could be established from the task defined in the terms of reference.

Finally they were asked to survey and report with recommendations on any safety or remedial measures required to the foundations and structures of other existing buildings and underground services in the central area of the University.

In addition to Harris & Sutherland, Professor Bishop of the Imperial College of Science and Technology was asked by the University to advise them on the soil mechanics aspects of the remedial works on the "cause or causes" as defined above. This report has been written in conjunction with Professor Bishop and represents the joint conclusions of both Harris & Sutherland and Professor Bishop in accordance with the above brief.

1.2. Aims of the Investigation

The technical questions raised by the foregoing events and the remedial grouting measures executed can be summarized as below:

i) What were the mechanisms and causes of the ground subsidence beneath the Cornwallis building?

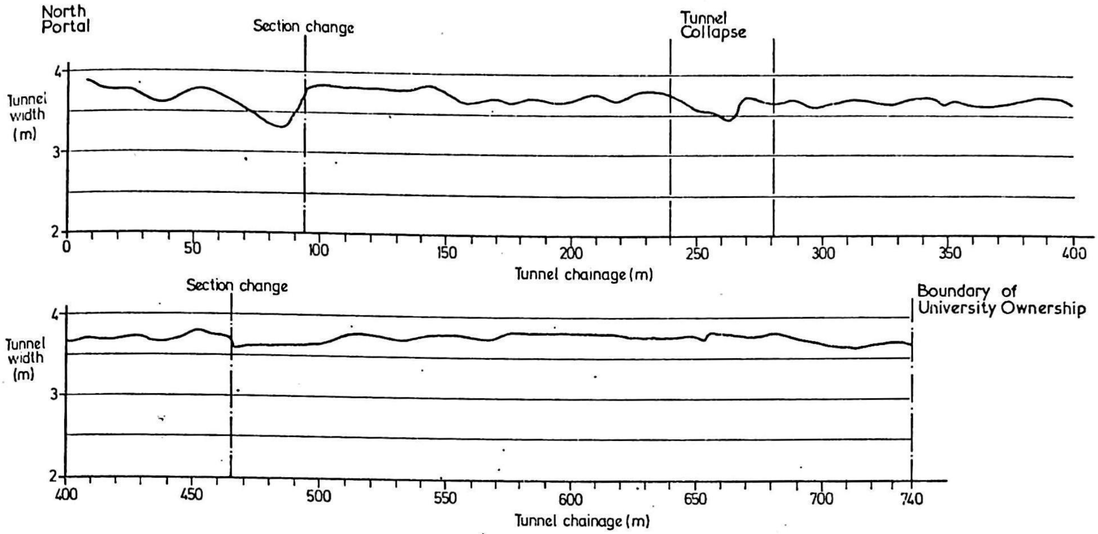
ii) What were the mechanisms and causes that resulted in the section of the old railway tunnel within Chainage 240 to 270 collapsing?

iii) Is the remaining length of the tunnel a danger to the existing buildings over it or to future developments?

iv) Is the subsidence area capable, now or in the future, of supporting the loads imposed by a new building?

v) Are the buildings adjacent to the area of subsidence secure and will they be subject to further movement?

vi) Is the remainder of the site, remote from the tunnel centre-line, substantially safe from similar movements to those which caused the collapse of the buildings in the Cornwallis block?



NOTE:
The values are taken from the Farmer & Dark report of NOV. 1973.

FIG: 2 A
Variation in tunnel width.

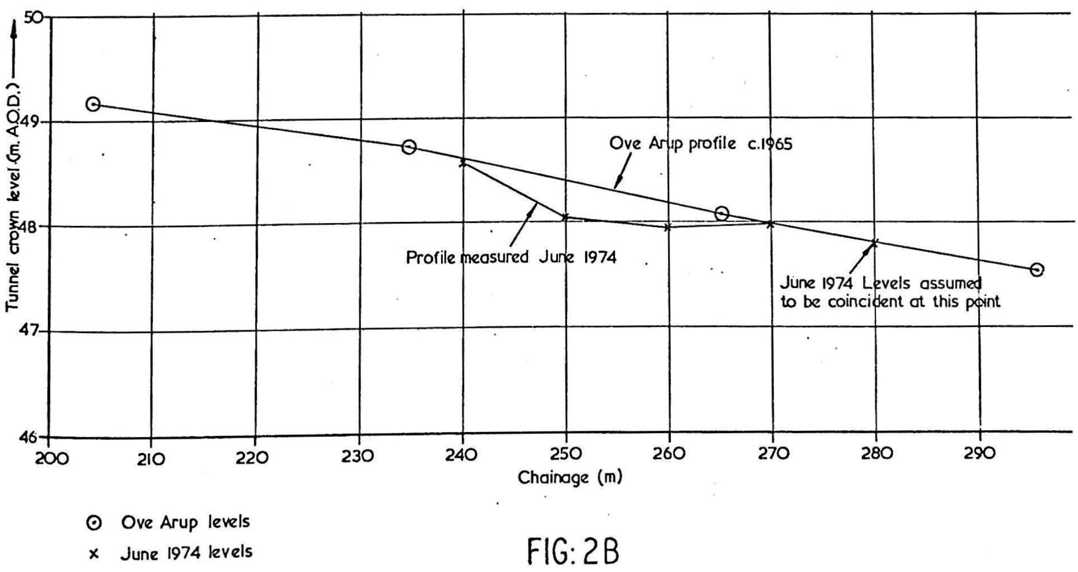
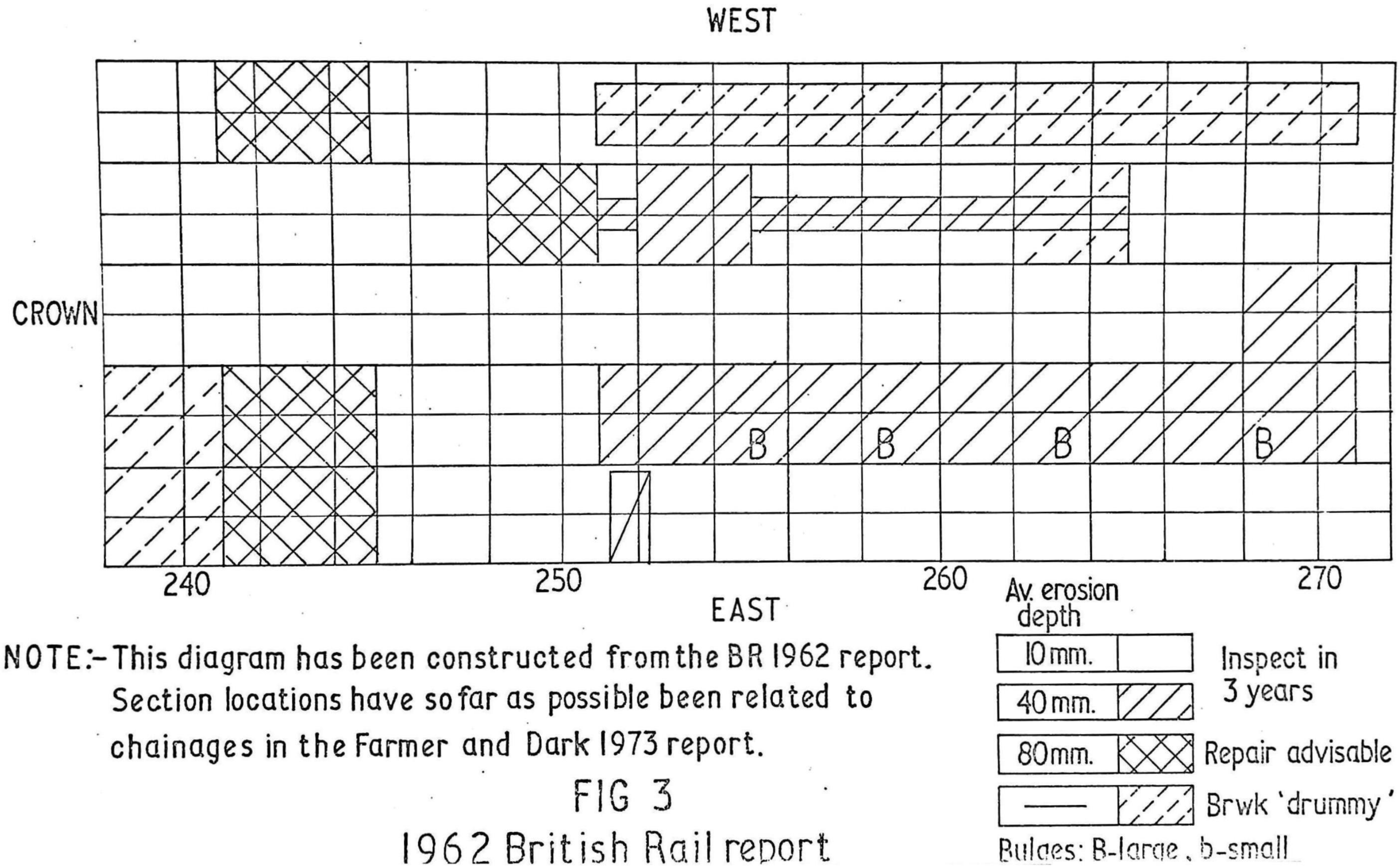


FIG: 2B
Tunnel crown profiles



NOTE:- This diagram has been constructed from the BR 1962 report. Section locations have so far as possible been related to chainages in the Farmer and Dark 1973 report.

FIG 3

1962 British Rail report

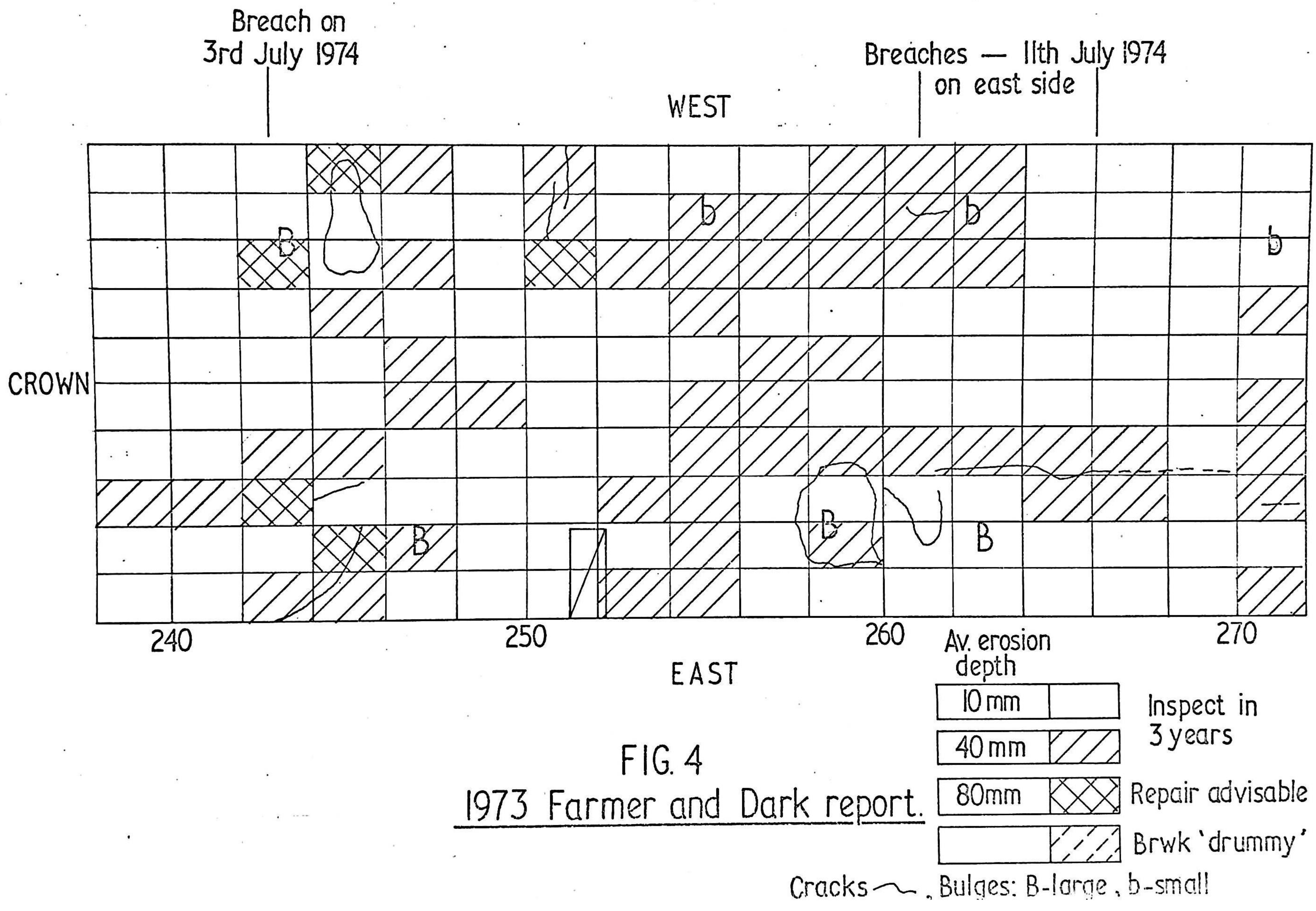


FIG. 4
1973 Farmer and Dark report.

THE SUBSIDENCE

EVENTS AT TUNNEL	DATE	EVENTS AT SURFACE
	JULY '73	
	18-	Cracks noted in partition on 1st floor of Block D (Link). Level check to underside of floor reveals slight difference & SE column 5mm lower than NE column
	AUG. '73	
	SEPT. '73	
	OCT. '73	
F & D report lining particularly bad at about Ch.240-250. Worst place 2 half brick rings had failed.	- 5	
	NOV. '73	
Submission of F & D Report on tunnel	-20 late	
Periodic inspections no perceptible change	DEC. '73	
	JAN. '74	
	FEB. '74	
	19-	Hair crack 3' long in ground floor partition SW wing of Block B.
	MAR. '74	
	APRIL '74	
	Late-	Hair crack in GF partition of SW wing of Block B slightly increased in length.
Inner face of lining deforming Ch. 243 and 260	-26	
	MAY '74	
Local falls of brickwork at Ch. 243 and 260 approx. Haunch and crown unaffected.	-Early	
	Mid-	Slight widening of crack in GF partition of SW wing of Block B and 2 or 3 other fine cracks in partitions noted on first floor of Block B.
Cracks in ash filling to the floor at about Ch.240 which suggested filling was heaving. At Ch. 240 brick spalling over about 6 sq.m mainly ½ brick but deeper in places. At Ch. 260 brick spalling over about 20 sq.m.	-31	
	JUNE '74	
Scaffolding placed to stabilise peeling brickwork. Trenches reveal invert failure at Ch. 248 and 256.	-Early	
Further local peeling of brickwork and fall of inner rings. No change in levels of crown.	-Mid	
Scaffolding deformed slightly	-19	
Brick samples analysed for sulphates.	-20	
F & D estimate that max. heave in floor is at least 150mm and max. inward movement of tunnel sides at least 50mm	-25	
	End-	Cracks slightly enlarged. Frequent monitoring started from pencil tell-tales.
	JULY '74	
Breach of tunnel lining with ingress of clay at Ch. 243 cavity to west side of tunnel.	-3	
	4-	Cracks enlarged - new ones in partitions and cols. Doors binding. F & D consider that settlement was taking place.
	5-	Glass tell-tales fixed. Building heard to creak
Further ingress of clay at Ch. 243	-8-	Some cracks enlarged with new cracks in Block D (Link) levelling of floors.
Further ingress of clay at Ch. 243, hole enlarged, crown intake.	-9-	Some tell-tales cracked. Frequent creaking heard. 37mm settlement measured.
Further ingress of clay at Ch. 243	-10-	Evacuation from SW wing of Block B and also Block D. 50mm settlement measured.
Further breaches between Ch. 260 & 268	-11-	Loud reports from SW wing of Block B & also Block D in after-noon followed by rapid settlement in evening.
First borehole started @ Ch.195 for filling tunnel.	-18	

FIG. 5

CHRONOLOGICAL LIST OF EVENTS (FOR TUNNEL AND CORNWALLIS BUILDING) FROM JULY 1973 - JULY 1974

2.1. Events Prior to the Subsidence

2.1.1 Sequence of events

The history of the tunnel prior to the subsidence is given in section 4, but a detailed chronological account of both the events in the tunnel and the Cornwallis building is given in Figure 5 for the 12 months preceding the subsidence.

2.1.2 Photographic record

A photographic record of the events in the tunnel from May 1974 to July 1974 was made by the University. Figures 6 - 14 inclusive have been selected to illustrate the progress of events. Figure 15 gives a plan of the falls within Chainages 240 to 270 together with a photographic key.

2.1.3 Eye witness accounts

The following eye-witness accounts are included to amplify both the chronological list of events given in Fig. 5 and the photographic record.

Falls of brickwork at Chainages 243 and 260 in early May 1974
(Letter from University to F & D on 9th May 1974)

A recent inspection of the tunnel showed that near the 240m (Report Note: probably 243) mark the inner face of the brickwork on each side of the tunnel had fallen to a maximum depth of 1½ bricks. In the main the depth of spalling in this area was about ½-brick thickness and the whole area affected is about $6m^2$ with the apparent possibility of this spreading in the near future to a similar area in extent. Again, at the 260m mark there has been a similar deterioration and in one area the brickwork has come away for its total depth so that for a small crevice of about 100mm it is possible to see right through to the clay surround. At the 260m mark the collapsed brickwork extends over an area of about 20m and a further area of similar size is highly suspect. At no point does the deterioration extend as high as the crown or haunches of the structure.

(a) May 1974

(b) Early June 1974

FIG: 6. Brick fall on east face at ch 260

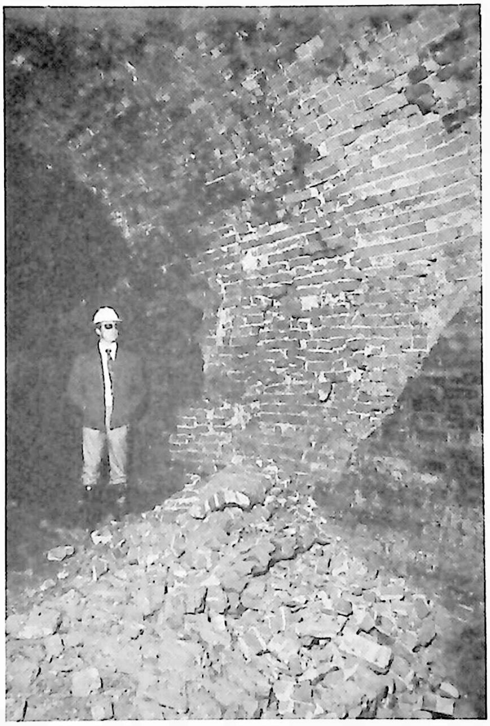


FIG: 7. Brick fall on west face at ch.243 — 10 June 1974

FIG: 8 Undisturbed brick invert at ch. 230 — 10 June 1974

FIG: 9. At ch. 248 the brickwork had heaved west of centre by 6"——10 June 1974

FIG:10. At ch. 256 showing the brickwork had sheared through over 6ft width and tilted by dropping 6" on the east side and rising 12" on the west - 10 June 1974.

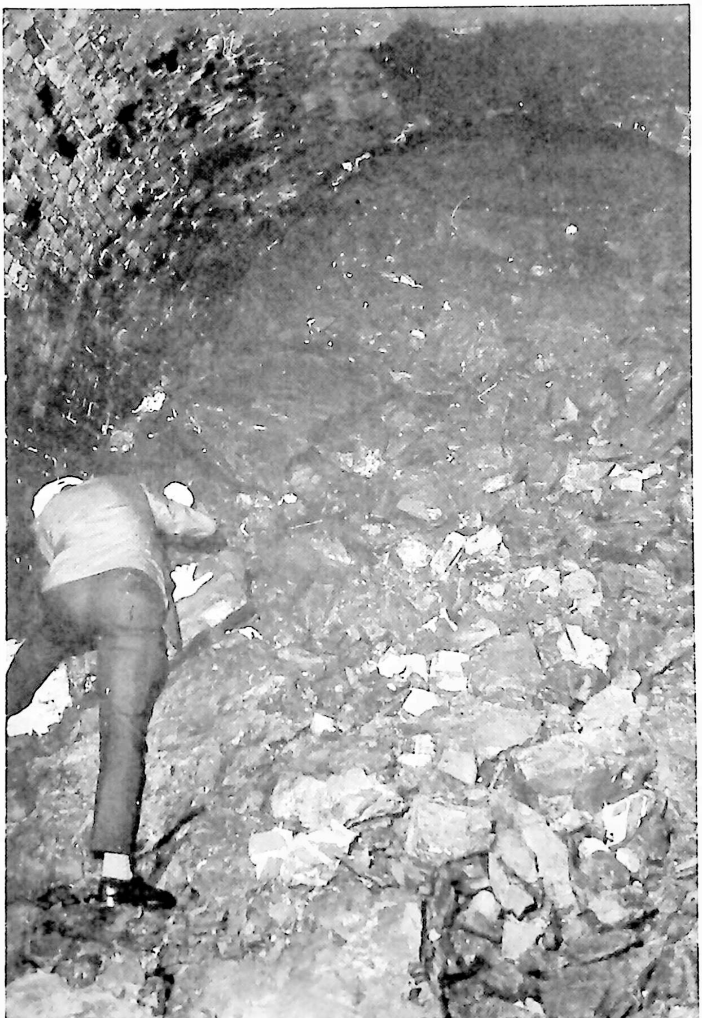


FIG: 11. Ingress of clay at breach. ch 243 looking south.
9 July 1974.

FIG:12 Cavity above clay fall at breach. ch 243 app. — 9 July 1974

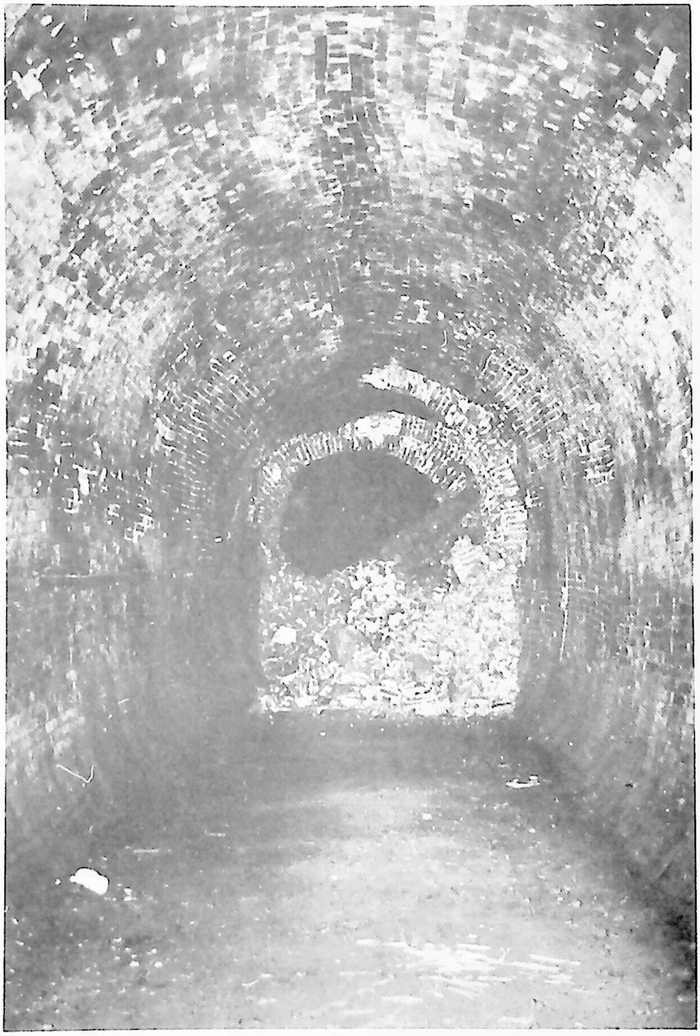


FIG: 13. Showing falls at the second breach from about ch 263 to 266 (note fall at first breach ch 243 in background).—11 July 1974.

FIG:14. Showing top of fall at second breach at about ch. 266 - 7. — 21 July 1974

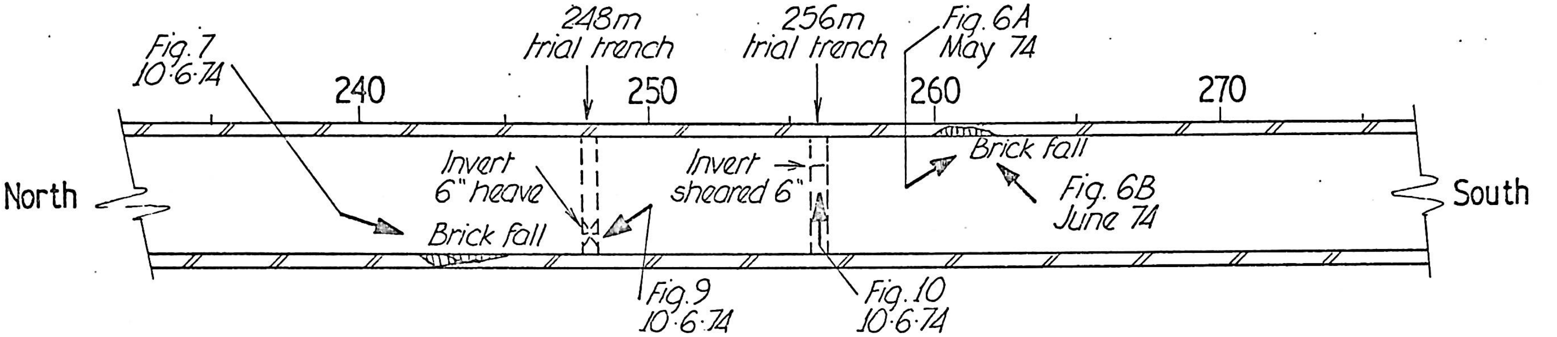


FIG: 15 A. Brick falls during May-June 1974.

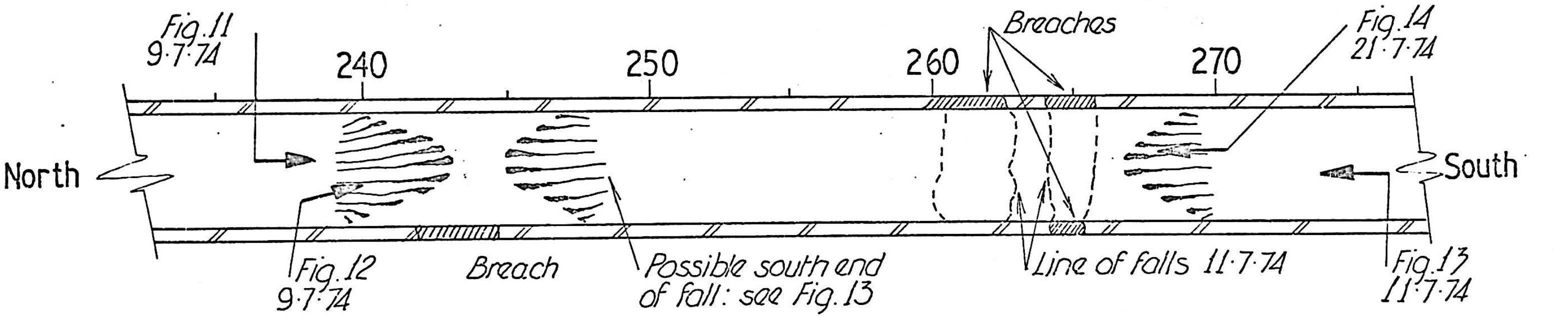


FIG: 15 B. Clay falls 3rd-11th July 1974.

NOTE.

Only photographic information is recorded on this figure.

FIG: 15 Plan of falls and photographic key.

Invert failure at Chainage 248 and 256 in early June 1974

(Letter from University to F & D on 11th June 1974)

I enclose some more photographs from the tunnel. Four sets show the invert at separate positions along the tunnel - namely at 6, 230, 248 and 256 metre points.

You will see that at 230m the bottom brickwork is in perfect condition; at 248 the brickwork shows a hump at invert (not particularly visible now due to the cutting of the trial hole through the brickwork) and at 256m the bottom section of brickwork has sheared and tilted.

In the last few days there appears to have been considerable movement of the brickwork on the west side of the 240, (Report Note: probably 243) position. Another area of brickwork has fallen from the wall, exposing the fourth skin in some places and even the clay beyond. Immediately to the north of this the outer skin of brickwork is moving outwards, and will almost certainly fall in the near future.

Invert failure at Chainage 248 and 256 in late June 1974

(F & D Diary of Events for 25th June 1974)

Visited site and inspected tunnel. Heave in the floor of the tunnel was more pronounced and it was clear from the trial trenches that the brickwork had sheared and the sides of the tunnel over this worst section had moved inwards. I estimated the maximum heave as at least 6" and the maximum inward movement at least 2".

Breach at Chainage 243 on 3rd July 1974

(Letter from University to Dr. Vaughan on 9th October 1974)

We were talking last week about the apparently curious nature of the cavity to the west side of the tunnel at the point where the tunnel wall was breached at approximately Chainage 243m.

I am enclosing a rather poor sketch (Report Note: Figure 16) of a section of the north breach as it was when I first saw it. I stood on the debris in the tunnel, which at that time had not extended completely across the tunnel floor and carefully examined the cavern which could be seen through the breach. I was particularly interested in the quite level and practically smooth bridge over the cavern which was about 8ft in width and length along the length of the tunnel. The underside of the clay looked completely undisturbed and natural.

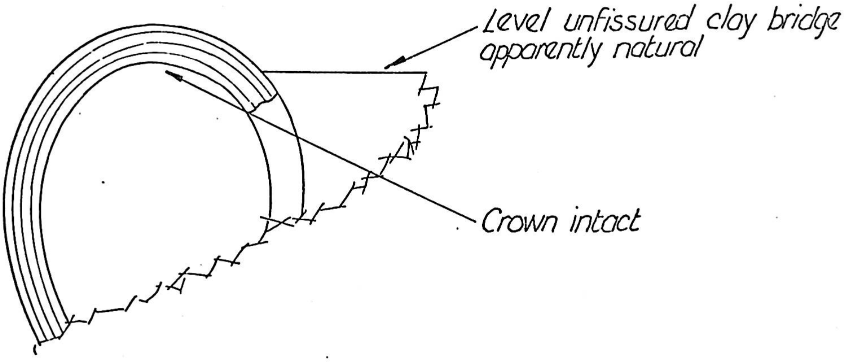


FIG. 16

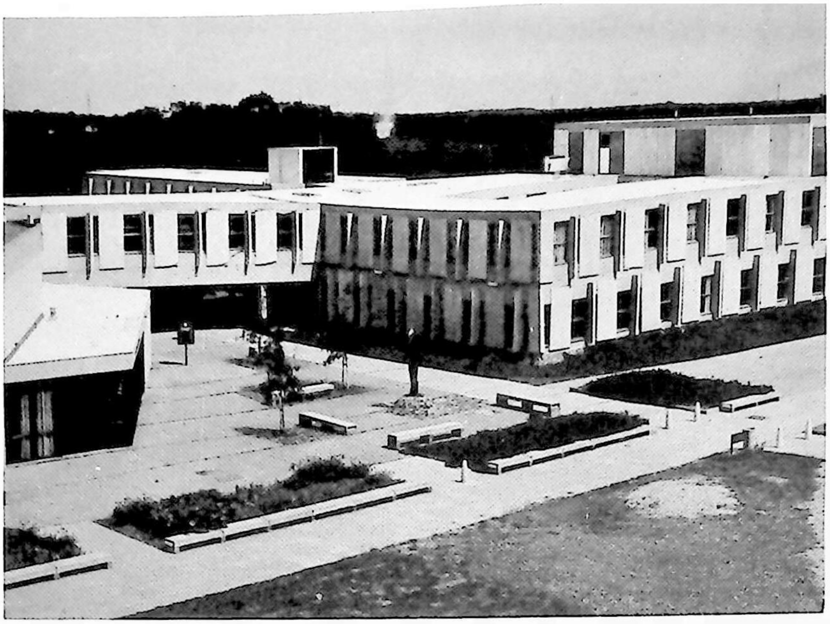
CAVITY TO WEST SIDE OF BREACH AT CH.243 ON 3RD JULY 1974
(From sketch by the University Surveyor and Deputy Registrar)

Figure 12 tends to confirm the presence of some form of cavity on 9th July at this Chainage.

2.2 The Subsidence

From Figure 5 it will be seen that cracks were noted in the bridge link (Block D) as early as July 1973 and in the SW wing of Block B in February 1974. While it is possible that these cracks were precursive to the subsidence, this is not necessarily so. However, in late April 1974 with the discovery of the deformation of the lining at both Chainages 243 and 260, which lay directly under the Cornwallis buildings, the crack in the SW wing of Block B was seen to have lengthened. Thereafter, throughout May and June, a developing pattern of events in the tunnel and in the Cornwallis building occurred culminating in the breach of the lining with the ingress of clay at about Chainage 243 on 3rd July 1974.

Although minor settlement may have been taking place in the Cornwallis before 3rd July, it was noticed distinctly on 4th July. By the 9th July a settlement of 37mm was recorded and on the 10th it had developed to 50mm.



Cornwallis buildings photographed from Library roof.

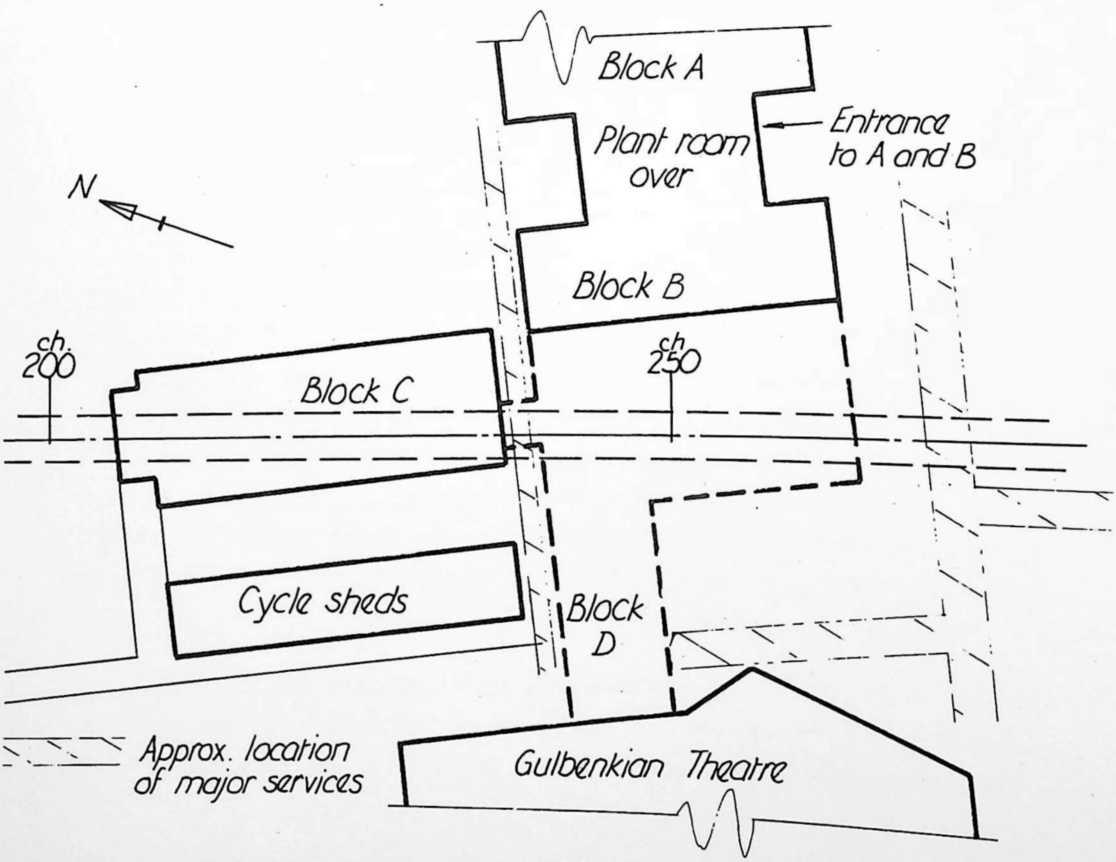


FIG: 17. Plan of Cornwallis Buildings (1: 500)

These accelerating events on the surface were accompanied by rapid changes of the conditions in the tunnel with the breach at Chainage 243 enlarging with more and more clay flowing into the tunnel. Finally, eight days after the first breach at Chainage 243 the series of breaches between Chainages 260 and 270 occurred on 11th July, as described below by the Surveyor and Deputy Registrar.

The final and dramatic stage of the subsidence began during the afternoon of 11th July. Three members of the University Surveyor's staff who were inspecting the now ailing tunnel from its southern end were very nearly caught in the fall at the second breach at about Chainage 266. This fall is shown in Figure 13 which was photographed before the tunnel was evacuated. The eye-witness accounts reveal that loud rumblings were heard from the failed section of the tunnel during the clay fall and on the surface three loud reports from the link bridge were heard at approximately the same time as the fall.

At 9 p.m. in the evening of 11th July the building began to settle. Within an hour or so the movement had stopped. On the morning of 12th July Block B and the link bridge of the Cornwallis Building and the adjoining ground surface were noted to have sunk by up to 700mm. Figure 17 shows the damaged area photographed from the Library roof together with a plan of the Cornwallis Building on which the severely damaged parts are indicated.

For obvious reasons of safety events in the tunnel were not monitored after the subsidence. However, Figure 14 shows the top of the southern fall, as photographed on 21st July. The fall completely blocked the tunnel and reached almost to Chainage 270. Evidence from the site investigation and the remedial grouting indicates that the tunnel between the two observed falls did not remain intact and it is most probable that the arch collapsed over most of the length from Chainage 243 to 269.

The probable mechanisms of both the subsidence and the tunnel failure are described in section 5.

2.3 Effect on Buildings

2.3.1 Block D or Link Bridge

The link bridge suffered major damage from two causes. First the supports settled differentially giving rise to a considerable rotation of the structure, and second, largely as a result of this rotation, the upper floor of Block B remained connected to the end of the bridge even after its foundations had dropped away. This caused additional damage due to the extra load imposed on the cantilever end of the bridge. The damage is shown in Fig. 18, and Figures 19a & b show photographs of the foot of one of the link bridge columns and the west elevation of Block B from under the link bridge. An appreciation of the distortions described above can be gained from these pictures.

2.3.2 Block B

The differential movements in Block B were of the order of 1 in 40 and the structure failed by breaking its back at a point 14.3m from its western end. The line of this failure can be seen in Figure 17 at the junction of the whole line and dashed portion of the plan which indicate the remaining and the demolished section of the building respectively. A curious feature of the damage to Block B was that at its western end, where the subsidence was maximum, the strip foundation which supported the perimeter of the block settled and dropped away from the load bearing panels it was intended to support. This tended to leave the first floor and roof supported by the internal columns and the cantilever end of the link bridge thus adding to the distortion on this latter structure.

This detail of the failure is shown in Figure 19(c) (It should be noted that the precast panels were designed as loadbearing supports to the structure above).

A detailed photographic survey of the damaged blocks was made and from this Figures 20, 21 & 22 have been produced which show the extent of the visible external damage.

Internally the floors and partitions were also severely distorted and cracks up to 100mm wide opened up in the partitions while the floors, particularly the ground floor, largely followed the profile of the subsidence from the point where the building broke its back to the west side.

FIG: 18. Damage to Block D

FIG:19(a)

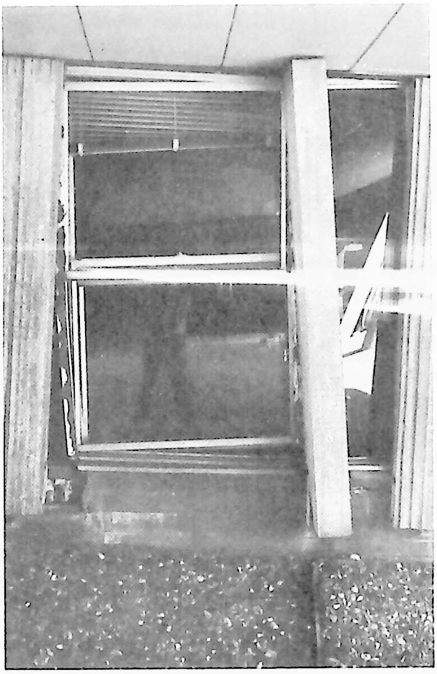


FIG:19(b)

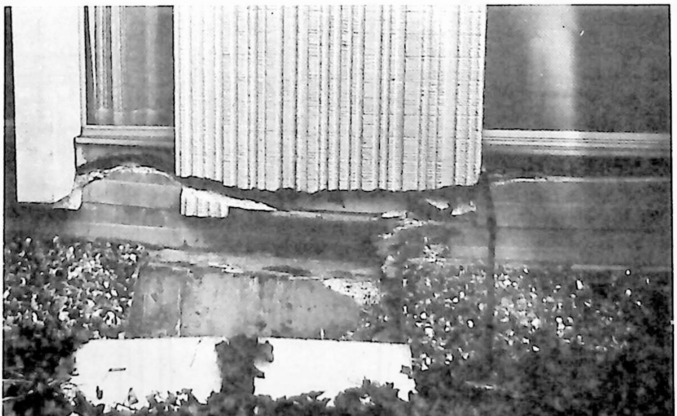
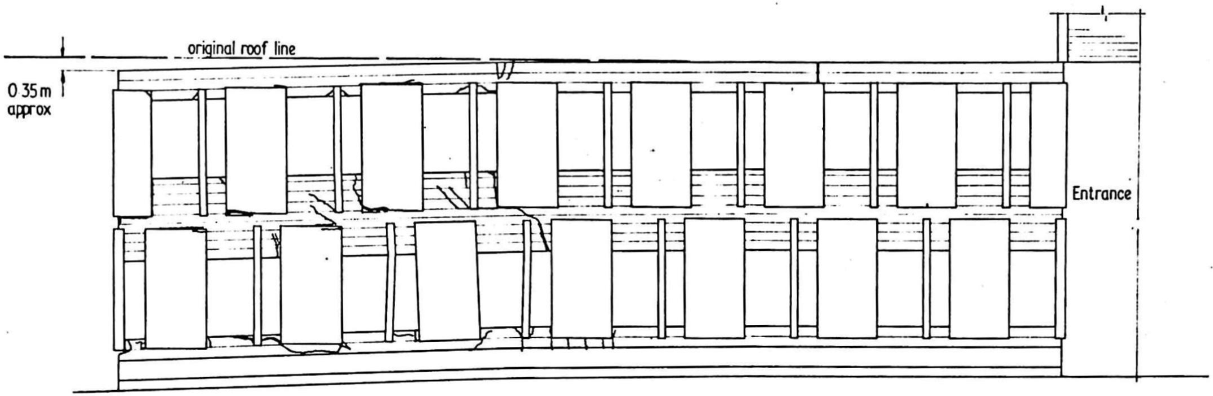
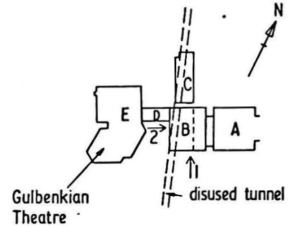


FIG:19(C)

FIG:19. Details of damage to Cornwallis building.



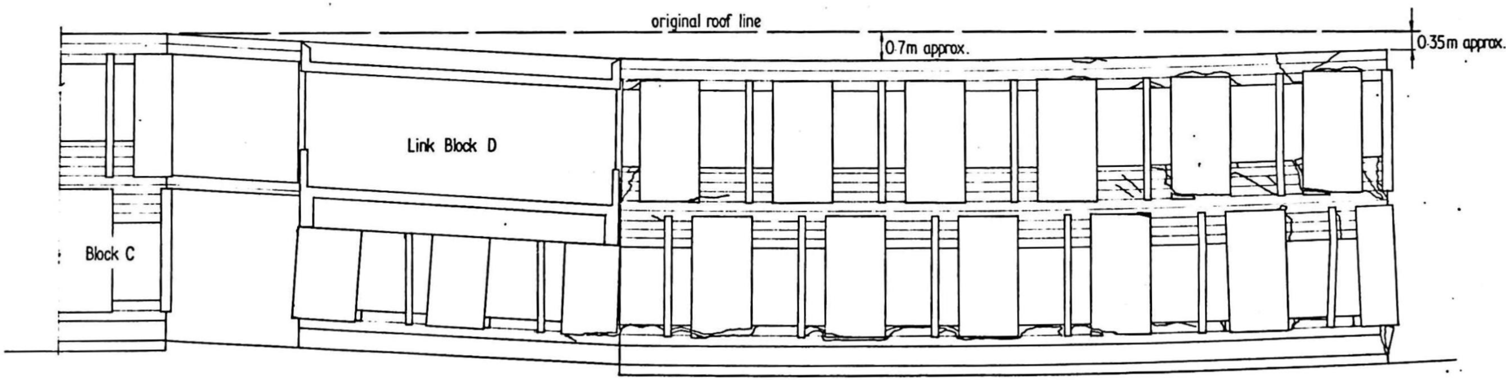
VIEW 1 - SOUTH EAST ELEVATION OF BLOCK B



SITE PLAN

NOTES

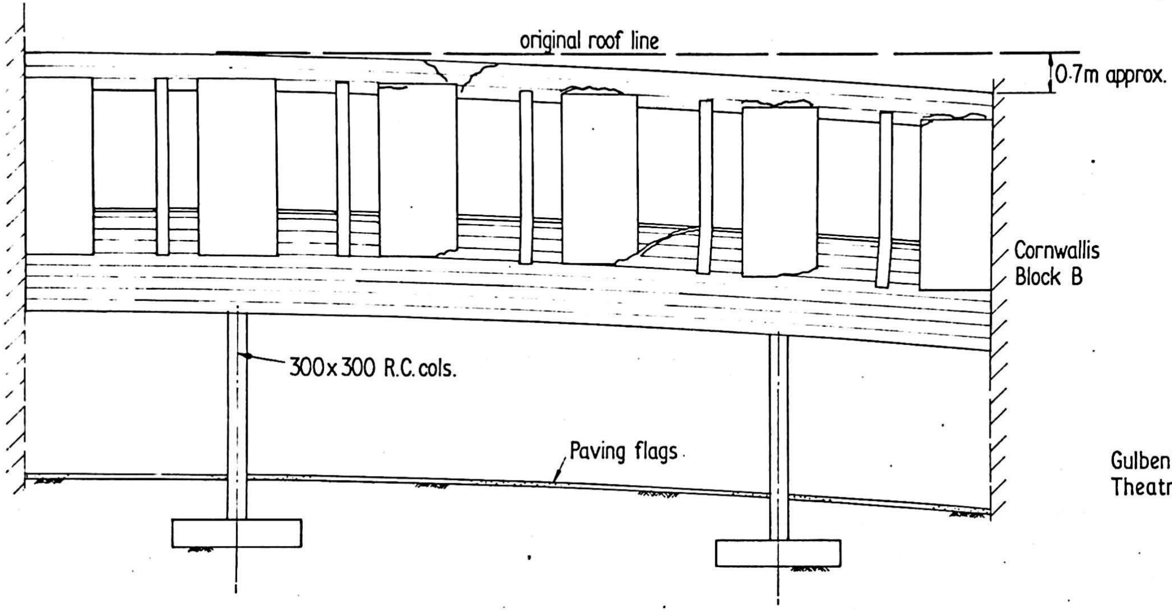
This drg. is based upon photographs taken between 1·8·74 and 3·8·74 and a series of levels of the roof edge taken on 13·8·74



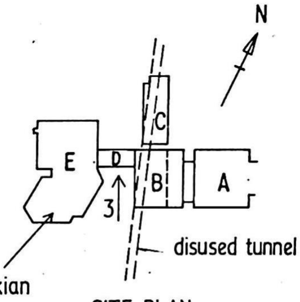
VIEW 2 - SOUTH WEST ELEVATION OF BLOCK B

FIG. 20

VIEWS OF BLOCK B



VIEW 3 SOUTH ELEVATION OF BLOCK D

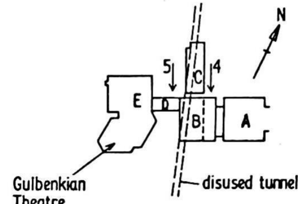


SITE PLAN

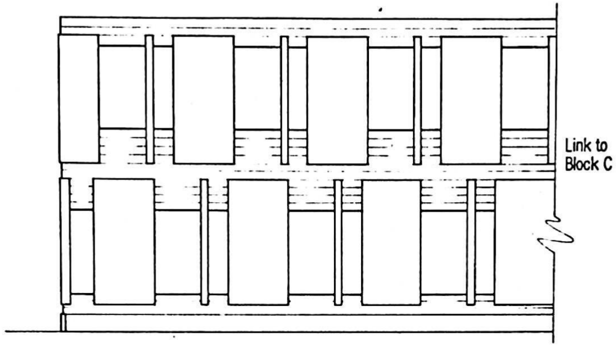
NOTE
This drg. is based upon photographs taken between 1·8·74 and 3·8·74 and a series of levels of the roof edge taken on 13·8·74.

FIG. 21

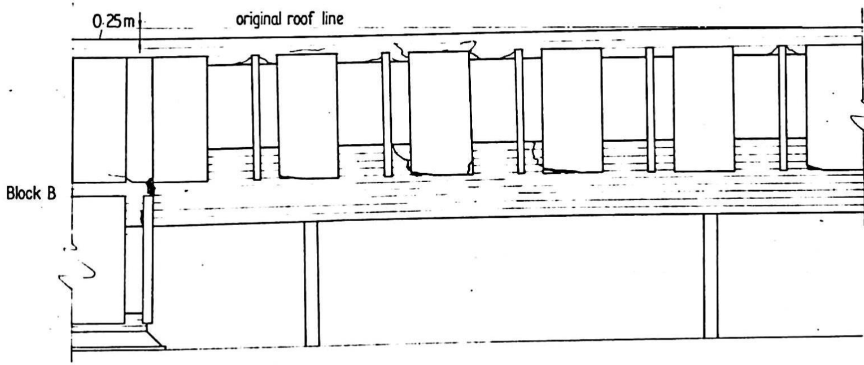
View of Block D



SITE PLAN



VIEW 4 — NORTH WEST ELEVATION OF BLOCK B



NOTE
This drg. is based upon photographs taken between 1.8.74 and 3.8.74 and a series of levels of the roof edge taken on 13.8.74

VIEW 5 – NORTH WEST ELEVATION OF BLOCK D

FIG. 22
VIEWS OF BLOCKS B & D

2.3.3 The Gulbenkian Theatre

The Gulbenkian Theatre although not subject to direct subsidence was subject to damage from the loads imposed on it by the link bridge, the floor of which kicked into the theatre and caused cracking of the internal partitions to a distance of 10 to 12 metres into the structure. This damage was only superficial and is not thought to have affected the structural integrity of the theatre.

2.3.4 Block C

Although the Cornwallis Block C was very close to the subsidence and is located directly over the centreline of the tunnel it did not suffer any appreciable damage. Two or three hair cracks visible in the structural walls have been monitored since the subsidence but do not appear to have moved. The link bridge from Block C to Block B, however, did suffer severe distortion and was demolished with the western end of Block B.

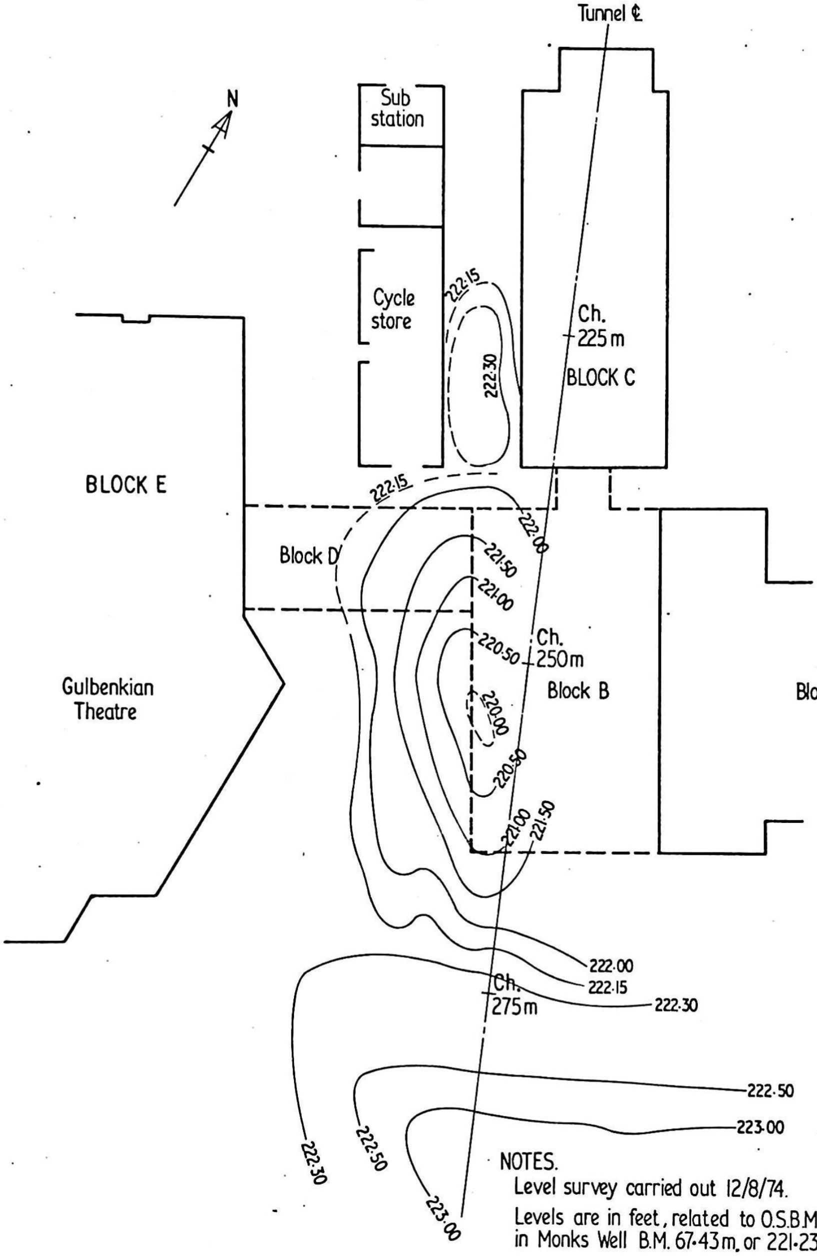
2.3.5 Other Buildings

In addition to the previously mentioned buildings the cycle sheds to the north of the link bridge were also damaged, but fortunately not beyond repair.

2.4 Effect on the ground surface

The subsidence caused the ground surface to deform over an area of approximately $750m^2$ in a bowl shape with a maximum settlement of 700mm. Only an approximate estimate of the extent of the surface depression has been possible since some fifty percent of it lay beneath the Cornwallis Block B. The ground contours shown in Figure 23 were measured on 12th August 1974 and from these the volume of the surface depression has been calculated as approximately $154m^3$. This can be compared with the original volume of the collapsed section of the tunnel which was approximately $348m^3$. This difference is considered in section 5.

Figure 24 shows photographs of the paving slabs inside the depressed area. Figure 24(a) shows the slabs buckling in the compression zone at the centre of the area whereas Figure 24(b) shows a tensile zone at the edge of the depression where the slabs have parted by as much as four inches.



NOTES.

Level survey carried out 12/8/74.

Levels are in feet, related to O.S.B.M. off University Rd. in Monks Well B.M. 67·43 m. or 221·23 ft.

Contours shown on this drg. represent the ground levels at the date of the survey and will reflect falls formed at the time of construction.

The paving slabs between the Gulbenkian Theatre and the Cornwallis Building had been removed before the date of this survey.

FIG. 23

Contoured plan of subsided ground

FIG:24 A. Compression of surface

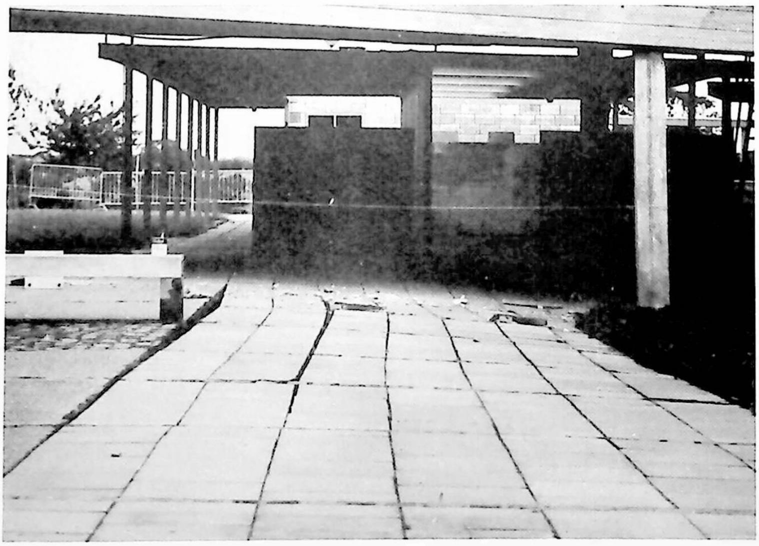


FIG: 24 B. Tension in surface

FIG:24. Paved surface in zone of subsidence

2.5 Effect on Underground Services

The following services which crossed the area of subsidence were seriously affected:

i) high pressure hot water mains laid in a concrete and brick built duct;

ii) LV and HV electric cables;

iii) 4" water main and minor connections;

iv) foul sewers;

v) surface water sewers;

vi) local central heating pipes..

These services had to be supported where practicable. Others had to be re-routed over considerable distances both inside and outside the subsidence area.

The University Surveyor and his staff acted as consultants for the mechanical and electrical engineers services and provided liaison and on-site support through these and all stages of the works.

SITE INVESTIGATION AND MONITORING SURVEY

3.1 Nomenclature

Definitions of the geotechnical terms used in this section are given in appendix F. They are intended to aid the non-specialist reader and while they are sufficiently accurate to give an understanding of the geotechnical investigation described, it should be appreciated that they are by no means "strict" in the scientific sense.

3.2 Site Investigation and Tunnel Research

In addition to the immediate tasks of supervising the remedial works to the damaged buildings and services and the filling of the disused railway tunnel, Harris & Sutherland and Professor Bishop were also asked to report on the cause or causes of the subsidence and to make recommendations for any remedial measures required to stabilize the subsided ground. Furthermore, investigations and any necessary recommendations were sought on the ground conditions in respect of other buildings on the campus positioned over or near to the tunnel.

With these aims, the following investigations were carried out:-

i) a soils investigation and installation of piezometers, and

ii) a ground surface monitoring survey.

In addition useful soils information gained from both the tunnel filling and compaction grouting contract was considered.

3.3 Soils Investigation

Cementation Ground Engineering Ltd. were commissioned to carry out a detailed geotechnical investigation with the following objectives:-

i) to determine the nature and extent of the subsided zone of soil, in order to discover any available information concerning the subsidence and to assess the state of the ground in relation to the undamaged buildings which surround the subsided area;

ii) to investigate the soils beneath <u>existing buildings</u> on the line of the tunnel outside the subsided zone in order to provide information for the assessment of the continued ability of the ground to support these buildings;

iii) to investigate the soils beneath the sites for proposed buildings both over the line of the tunnel and elsewhere on the campus in order to obtain information relevant to the design of the foundations for such buildings;

iv) to obtain information on the soil to a depth of 20m at a location away from the tunnel centre line for comparison with the soil profiles obtained from boreholes in the area of investigation;

v) to install piezometers at several locations so that a study of the porewater pressures in the clay could be made; and

vi) to establish at selected locations the success of the tunnel filling operation by proving the existence of cement/pfa grout.

3.3.1 Method

Twenty-five shell and auger boreholes and eleven rotary holes were put down in the following areas as shown on Figures 25 and 26:

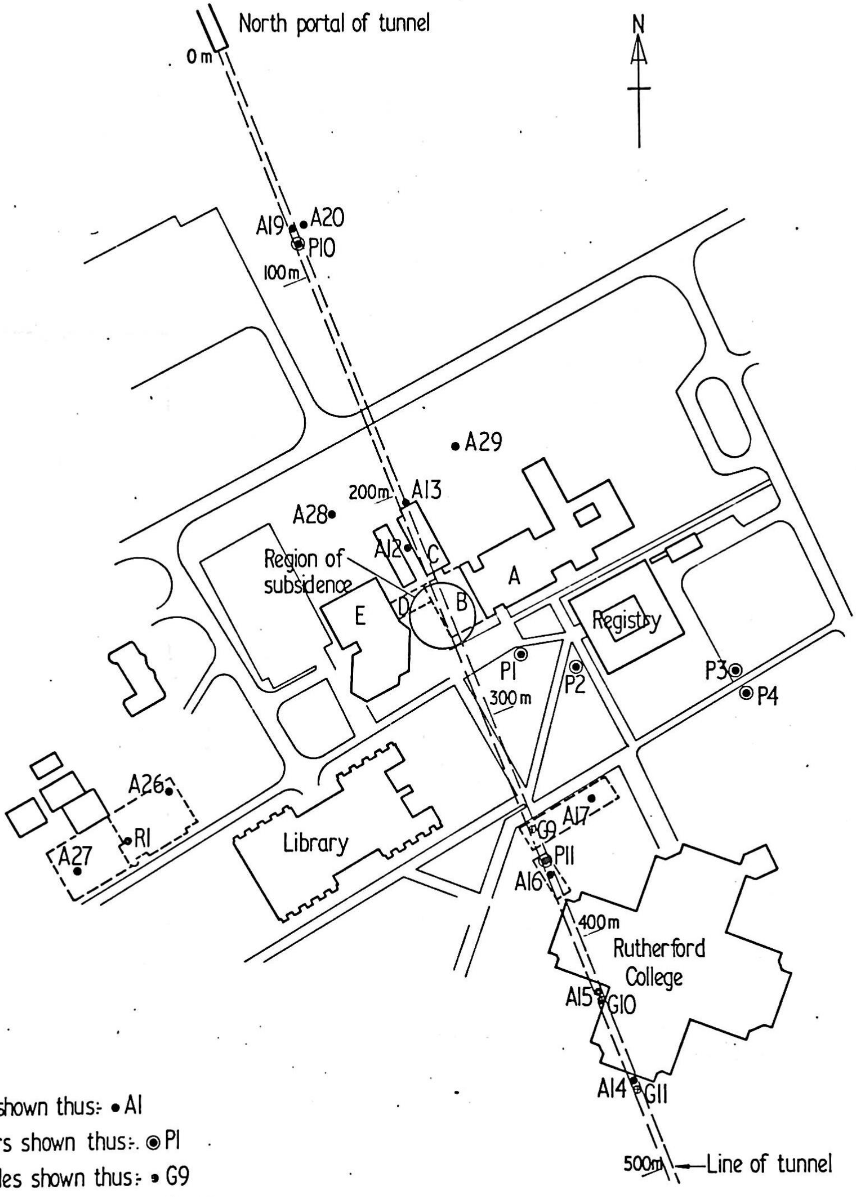
i) in the zone of subsidence;

ii) in the area adjacent to Rutherford College in order to assess the ground conditions under this building and a proposed extension;

iii) at tunnel Chainage 80 where the 1973 Farmer and Dark Report indicated that the condition of the tunnel was similar to that under the subsidence area; and

iv) at a site 150m to the west of Chainage 300 where samples of clay which had not been influenced by the presence of the tunnel could be obtained and used for comparison and control.

In 17 of the shell and auger holes "continuous"U100 samples were taken in the clay. Although the method of sampling was not suitable for the detection of small voids in the clay, it was considered that any sizeable voids or cavities would be evident. With this in mind the



NOTES.

Boreholes shown thus:- • A1

Piezometers shown thus:- ◉ P1

Proving holes shown thus:- ◦ G9

For location of boreholes in subsidence area see fig. 26

FIG. 25

Plan of boreholes

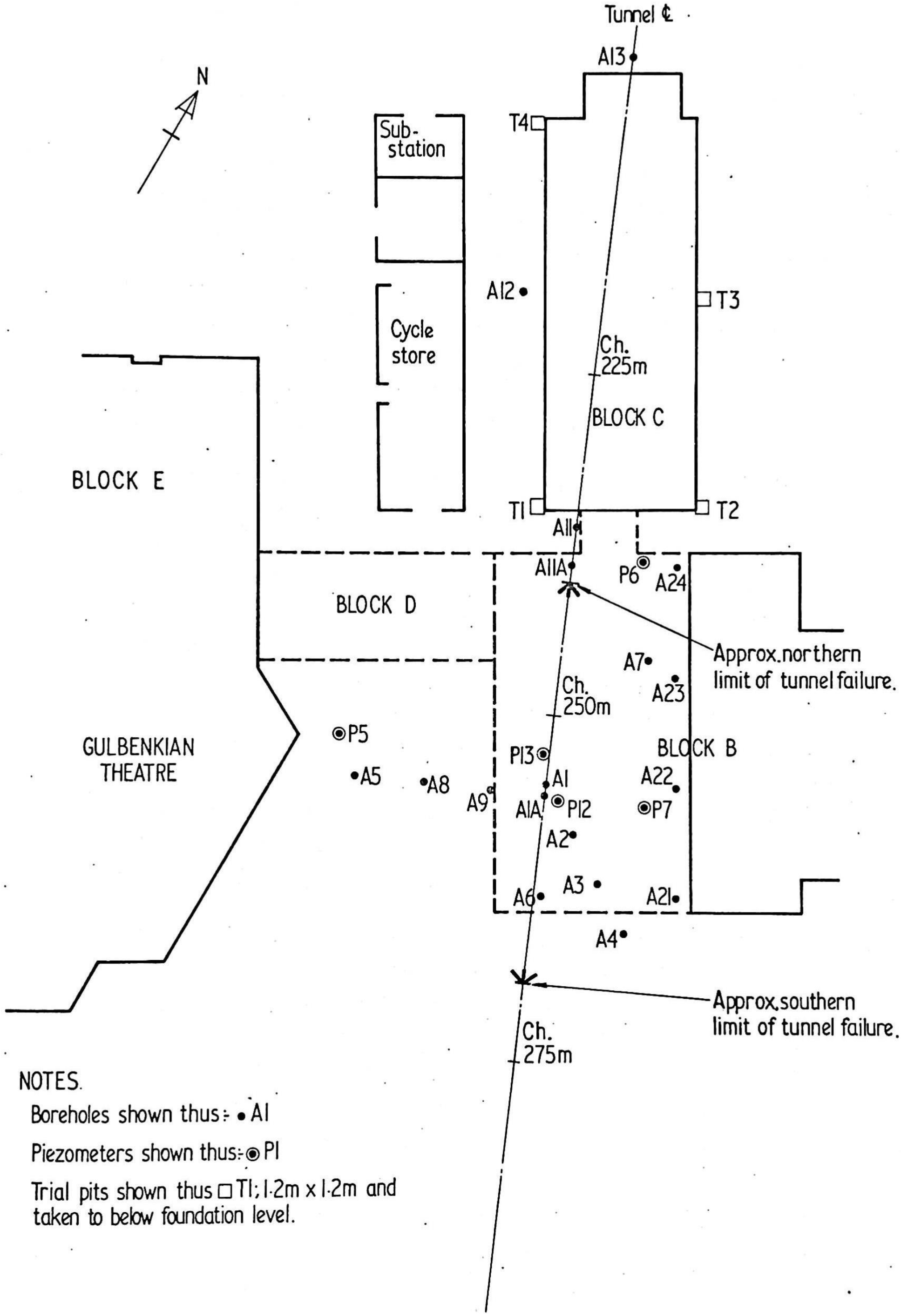


FIG. 26

Boreholes in subsidence area

drillers were instructed to note any unusually soft patches in the boreholes and also to report the locations of any voids encountered. The number of blows required to drive the sample tubes was also requested. In the event, while many areas of easy boring were recorded, <u>no actual voids or cavities were reported</u>.

The success of this method of sampling the soil can be judged from the percentage recovery of 100mm diameter samples which in some cases was as high as 96% of the total length bored. Where the percentage recovery was lower, due to difficulties encountered in boring and cleaning out the bored hole or taking the sample, bulk samples and cutting shoe samples were retained and recorded.

Much of the sample inspection and description was carried out in a mobile laboratory on site. Evidence of fissures, disturbed zones, shear zones and failure surfaces in the samples was recorded and in addition pocket penetrometer and moisture content profiles were measured. Each U4 sample was first split in two. One half was wrapped, sealed and labelled and then taken to store for possible future reference. The second half was first described in detail and then subjected to several pocket penetrometer tests. Next, thin slices were peeled off with a sharp knife and these were examined for evidence of fissures which were more easily visible in the thin slice since this tended to wrinkle on the fissure lines. Finally, moisture content samples were taken. This process of detailed inspection was applied to 433 U100 samples on site.

A further 167 samples were sent to Cementation Ground Engineering's laboratory at Rickmansworth for strength testing. Other routine tests such as consolidation and Atterberg limit measurements were performed on selected samples. The strength tests were performed in triaxial testing machines on 100mm diameter samples according to a procedure laid down by Professor Bishop of Imperial College. This procedure was basically that for the conventional quick undrained triaxial test, but calling for a slower strain rate of 0.02% per minute using a cell pressure of 2.0 times overburden. The pore pressure was measured at the base of the sample after the application of the cell pressure. The results of these tests were corrected for variations of sample suction.

3.4. Piezometer Installations

In order to obtain some measure of the extent of the disturbance of the site both in the zone of subsidence and elsewhere, eleven standpipe and two electric piezometers were installed at the locations shown in Figures 25 and 26. The electric piezometers were unfortunately damaged by the grouting works and thereafter gave no useful information.

The standpipe piezometers were far more successful and appear to give reliable results.

An explanation of porewater pressure measurement is given in Appendix G to this report.

3.5 Results

3.5.1 From Site Investigation

Figure 27 shows graphs of the undrained shear strength with depth at several locations. It can be seen that in spite of the precautions taken there is a large scatter in the results. However, it is also clear that the strength of the clay samples from the zone of subsidence is not significantly different from that of those taken elsewhere.

The complete results of the soils investigation are given in the Cementation Ground Engineering report No. 5088/74/JMJ, dated May 1975. A cross section through the subsidence zone is shown in Figures 28A & B. The information on these figures was taken from boreholes A1A, A2, A3, A4, A5, A6, A7, A8 and A9. It can be seen that the soil in this zone has been subject to considerable disturbance. However, there was no evidence found to suggest that the surface subsidence was caused by a cavity progressively caving in and rising to the surface and the majority of the samples inspected were intact. This leads to the conclusion, as discussed in section 5.1, that the soil probably failed as a single plug. There is evidence of disturbance and clay and brick rubble at the tunnel level and this is consistent with the clay rubble seen at each end of the collapsed section of the tunnel.

Figure 29 shows a plot of the moisture content/depth profiles for four boreholes in the collapsed zone. It will be seen that away from the tunnel the moisture content is roughly constant at about 25 - 27%

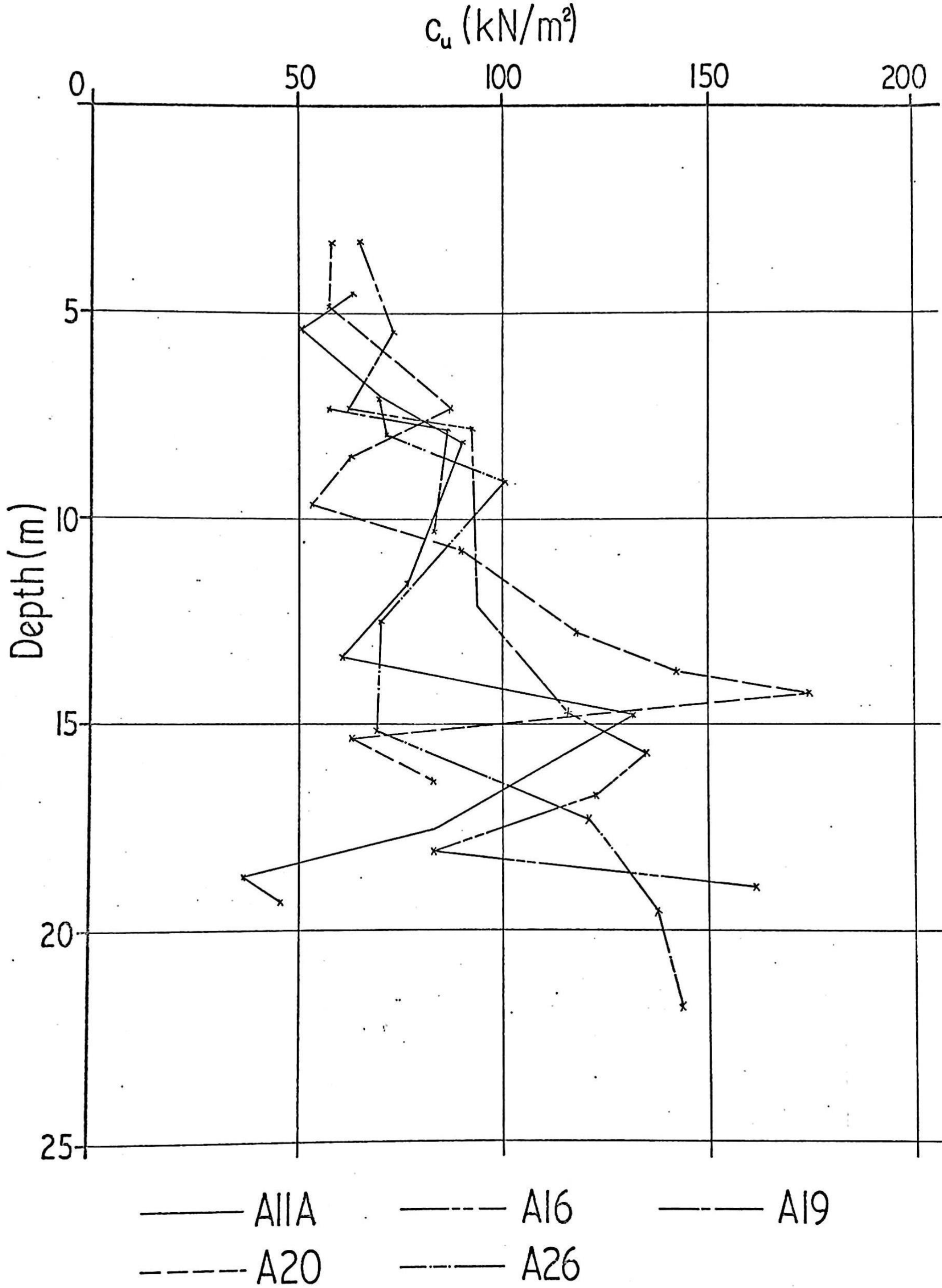


FIG. 27

Graph of undrained shear strength against depth

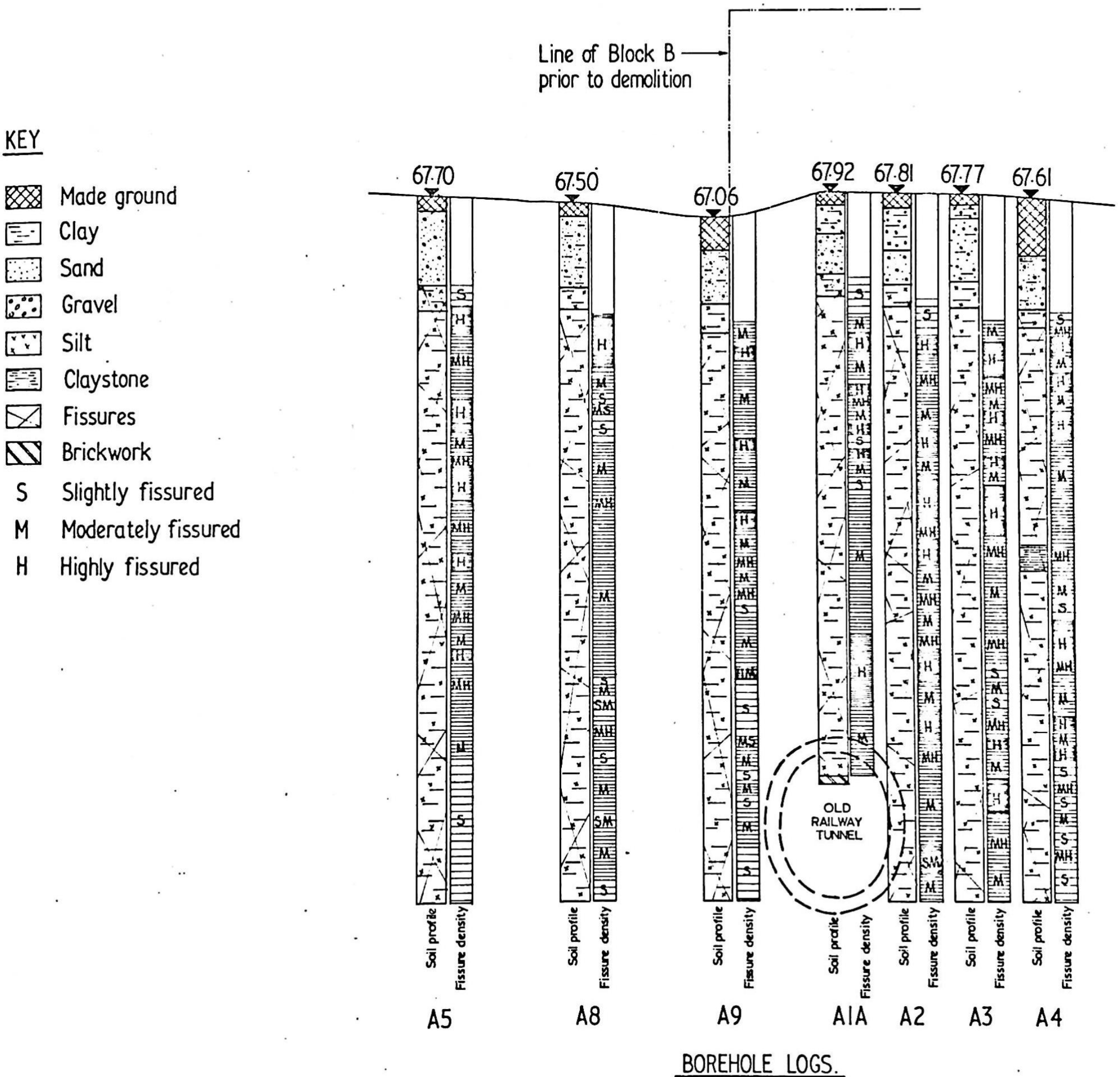


FIG. 28A

SUBSIDENCE ZONE BOREHOLE PROFILES

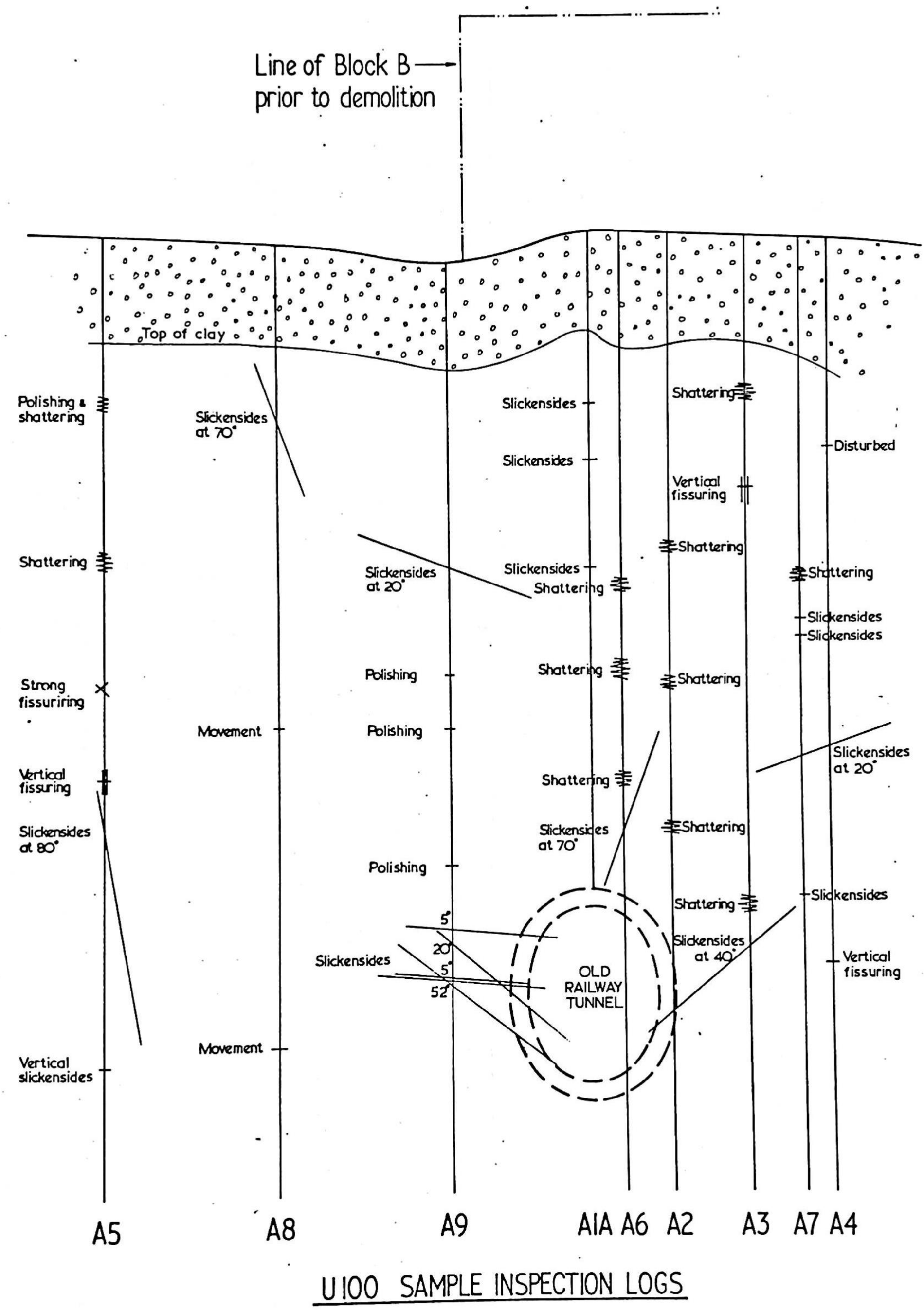


FIG. 28 B

SUBSIDENCE ZONE BOREHOLE PROFILES

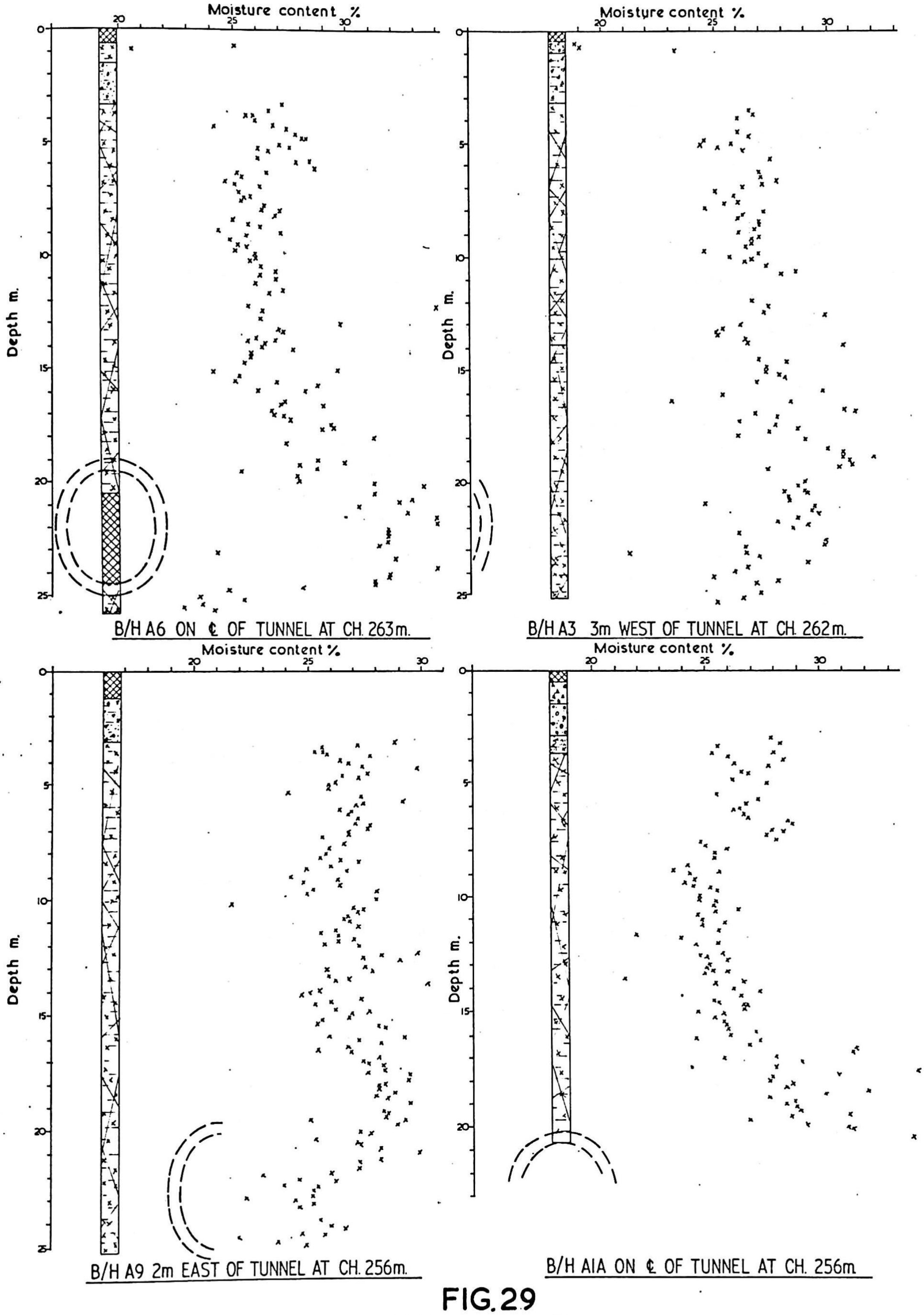


FIG. 29

Moisture content profiles

whereas near the tunnel there is an apparent rise from the approximately constant value of 25% to between 30% and 40%. This phenomenon is most evident in the boreholes put down on the tunnel centre line and its possible significance is discussed in section 5.3.

The measured moisture contents of two samples taken from the fallen clay inside the tunnel on July 21st 1975 were 29% and 29.8% respectively.

The pocket penetrometer profiles in general show an increase in strength with depth. Borehole No. A6, which was near the location of the southern end of the tunnel fall, shows a large variation in strength near to the level of the tunnel. This is associated with a large variation in the moisture content of the clay in this region.

3.5.2 Atterberg Limits

The liquid and plastic limits of the London clay were measured by Cementation Ground Engineering Ltd. at several locations. The results of their tests show that the clay is reasonably uniform across the site and that there is no major geological difference between any of the locations tested.

Tests by Imperial College on samples taken during the site investigation gave average values of:-

Liquid Limit	83%
Plastic Limit	30%

which are consistent with London clay.

Tests on the two samples taken from the fallen clay referred to in section 3.5.1 above gave average values of:

Liquid Limit	93.5%
Plastic Limit	32%

which could indicate a variation in the nature of the clay. However, it is not possible to say from which levels these samples came and therefore no specific conclusion should be drawn.

3.5.3 Piezometer Results

The piezometer results are shown in Figure 30 where it can be seen that

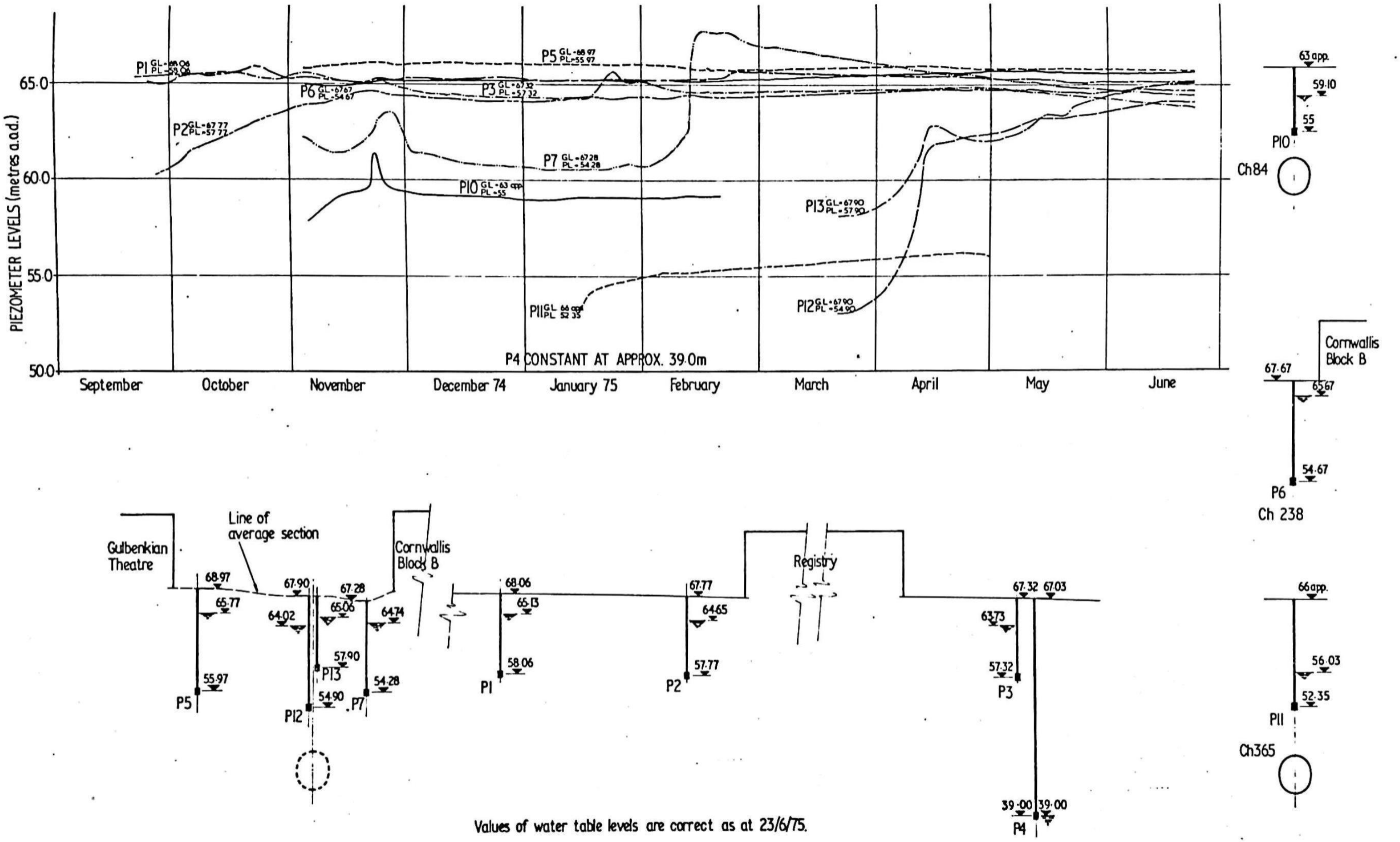


FIG. 30

Piezometric data.

everywhere outside the zone of subsidence the porewater standpipe level is approximately 2 metres from ground level. This is approximately 1m above the clay gravel interface and may well reflect the free water table level in the gravel. Piezometer P7 rose dramatically in February which was when the compaction grouting was in full operation. It is our view that this piezometer was in direct connection with the injected grout and therefore rose to overflow at the surface. It now appears to be registering normally.

Piezometers P12 and P13, near the centre of the subsidence zone, show that the porewater pressure here is approaching that elsewhere. It can therefore be expected that future changes will be of small magnitude only. Monitoring continues.

3.6 Other Geotechnical Information

Further information was also gained from both the tunnel filling boreholes and the remedial grouting works, although in the case of the former this was really only additional information on the strata encountered.

In contrast the remedial grouting works produced much useful information Before this work was started, it was appreciated that although the 16 boreholes of the site investigation in the subsided zone had not revealed any cavities which might have been pre-existing, considerable voiding undoubtedly existed after the collapse between the blocks of clay which filled the tunnel in the collapsed zone.

The grouting works were therefore planned with this in mind and a sequence of holes were put down to one metre below tunnel invert at 2.5m centres and then grouted with cement/pfa under gravity head to refusal. In all, over $100m^3$ of grout was placed in this manner, thus confirming the voided nature of the material in the tunnel. An examination of the boring debris, on the whole confirmed that most of the tunnel between the two initial falls also collapsed.

3.7 Proving Holes

In order to investigate the filled section of tunnel under the Rutherford College three proving holes G9, G10 and G11 (shown on Figure 25) were drilled and grout tested under a gravity head in January 1975. The object of the grout testing was to establish, by measurement of the volume of grout injected and the level of any excessive grout take, the existence or non-existence of cavitites in the region of the tunnel. The first hole, G9, accepted $1.76m^3$ of grout, this being some $1.6m^3$ greater than the nett volume of the drill hole. The majority of this excess grout was believed to have penetrated the region just above the tunnel crown brickwork. However, since the hole had in fact been drilled through the tunnel crown and the cement/pfa filling, the exact level of any cavity could not be ascertained. In view of this, holes G10 and G11 were installed to just touch the crown brickwork. The grout volumes injected in these holes over and above that required to fill the bore hole were:-

G10	$0.3m^3$
G11	$1.67m^3$

In both these holes this excess grout was taken into the ground at a level just above the tunnel crown brickwork. Hole G11 was, in addition, rebored to 1m below the tunnel invert and then successfully grouted with no excess grout required. This shows that at the location of hole G11 there was no grout accepted either at the invert level or at the underside of the crown brickwork. This evidence confirms that the pfa fill is in contact with the underside crown of the brickwork and supports the findings of the site investigation boreholes numbers A11, 13, 14, 15, 16 put down elsewhere on the tunnel centre line away from the zone of subsidence.

The volume of excess grout injected in these holes is consistent with the method of tunnel construction (see section 4). It is evident from contemporary reports in the press and other sources that there were regions of overbreak and that these were filled with local rock material (probably hand-packed ironstone). It is likely therefore that the grout penetrated the void spaces of this porous fill. The volumes injected were not large and clearly each proving hole only affected a small region of the material above the tunnel. Indeed hole G9 was only 2m from a previous tunnel filling hole which on completion of the tunnel filling was grouted in a manner similar to the proving holes described above. No evidence of grout from this hole was encountered when drilling G9 and in addition, it did not appear to restrict the grout take of G9. It can be concluded that the grout, when injected at the points in question, is unlikely to have travelled any more than 1 or 2 metres along the tunnel crown.

3.8 Ground Surface Monitoring Survey

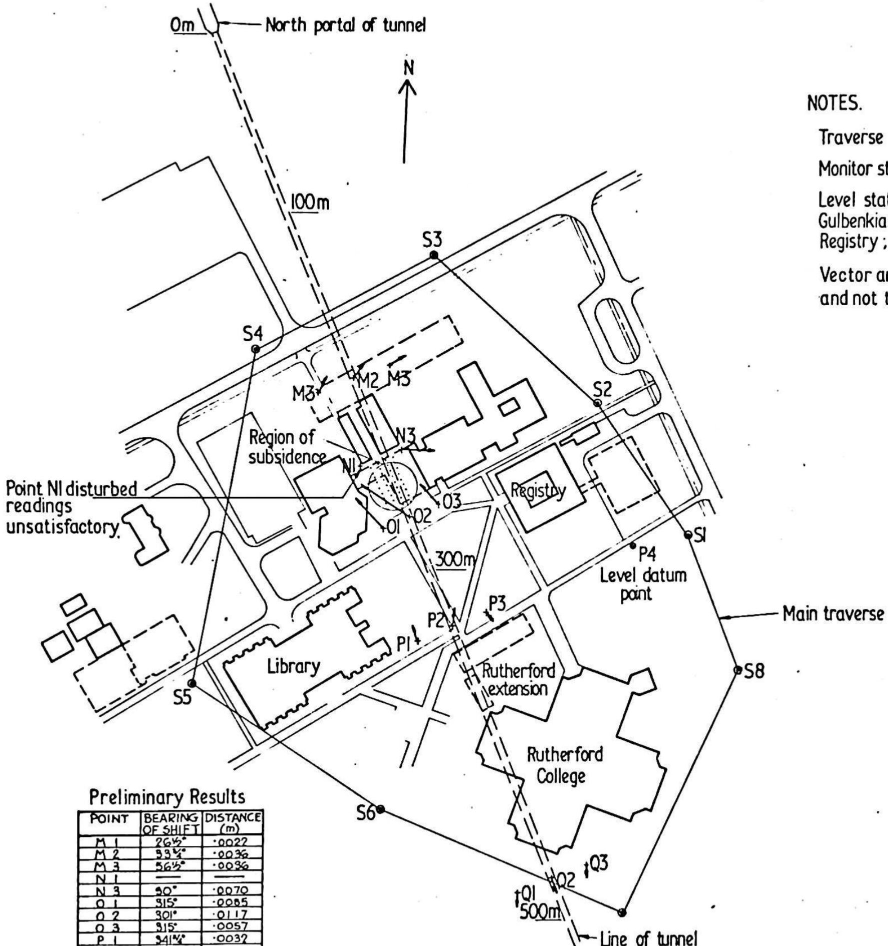
In order to monitor any movements of the surface of the ground both laterally and vertically, a series of survey stations was established (as indicated on Figure 31) by Plowman Craven and Associates in October 1974.

A second set of readings was taken during March 1975 when the remedial grouting had been completed. The reports of both these surveys are included in Appendix H to this report.

Figure 31 shows the measured movements of the monitor points as vector arrows. Point N1 is known to have been disturbed by the construction works in the subsidence area and it is believed that points N3 and 01, 02 and 03 have also suffered disturbance during the remedial works. Future monitor surveys will confirm whether or not this is in fact the case. Likewise confirmation of the measured movements of all other survey points will be obtained in the course of future survey work.

The building station levels measured by Plowman Craven and Associates on their first two visits to site showed an apparent maximum movement of $\pm$ 13mm and even allowing for possible inaccuracies this figure was viewed with some alarm. However, when the work was carefully checked, including an investigation of the stations originally chosen, it was discovered that the locations in question had suffered disturbance not attributable to ground or structural movements. Comparison between levels taken in March, April, May and June show insignificant movement.

The levelling results obtained to date (June 1975) are included in Appendix H.



NOTES.

Traverse stations shown thus:- ●

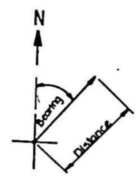
Monitor stations shown thus:- +

Level stations monitored on the following buildings:- Gulbenkian Theatre; Block A, B, C, E; Library; Registry; Rutherford.

Vector arrows show direction of movement thus:— and not to scale.

Preliminary Results

POINT	BEARING OF SHIFT	DISTANCE (m)
M 1	26½°	·0022
M 2	33¼°	·0036
M 3	56½°	·0036
N 1	—	—
N 3	30°	·0070
O 1	315°	·0085
O 2	301°	·0117
O 3	315°	·0057
P 1	341¼°	·0032
P 2	0°	·0030
P 3	135°	·0014
Q 1	180°	·0010
Q 2	161½°	·0032
Q 3	180°	·0020



MONITOR STATION

FIG. 31

Survey traverse & monitor stations

THE TUNNEL

4.1 Sources of Information

Following the subsidence in July 1974 N.L. Durbidge, Assistant Registrar in the University Surveyor's office, conducted an investigation with the object of assembling all available information relating to the construction and upkeep of the tunnel. Unfortunately his search for first hand information (such as constructional drawings and records) was unsuccessful and such information on the construction of the tunnel that was found has been derived from various standard works on railway history, local newspapers, etc. A copy of his report is attached as Appendix B.

With regard to the condition of the tunnel throughout its history, the only record found of inspections between its construction and the fairly regular examinations from 1925 - 1952 was that by Major General C.M. Pasley in 1846. Following the closure of the line in 1952, there was an inspection on foot every 3 years, the last of which was conducted by British Railways in May 1962 and is attached in Appendix B. In July 1963, just before the purchase of the tunnel (Chainages 0 - 737) by the University, G. Maunsell & Partners prepared a report on the condition of the tunnel. In 1964 Ove Arup & Partners carried out a survey of the tunnel crown. During the following years, the tunnel was inspected at intervals by the University Surveyor. In July 1973 Farmer and Dark were asked to survey and report on the condition of the tunnel, and their detailed report was submitted in November 1973.

4.2 History of the Tunnel

The tunnel was required to provide access through Tyler Hill for the Canterbury and Whitstable Railway and was built during the period of the railway's construction between 1825 - 1830. Although George Stephenson was appointed as Engineer, it was one of his assistants, John Dixon, who became the man on the spot. Initially, rolling stock was hauled through the tunnel by stationary engines but in 1836 direct traction by locomotives was introduced. In 1844 the railway was taken over by the South Eastern Railway Company which subsequently became part of the Southern Railway Company who, in turn, were absorbed by British Railways (Southern Region).

The railway was closed in 1952 and subsequently re-opened for a short period in 1953 to provide access to Whitstable for freight when other rail links were cut off by severe flooding. The line was finally closed and the permanent way removed when this emergency use ceased.

In 1955 the land over a 34m section of the tunnel from its southern end Chainage 737-771 was purchased for Archbishop's School. Shortly after this purchase, a brick partition with air bricks and wooden double doors was erected at the south portal together with a similar arrangement at the boundary of the school property at Chainage 737. In May 1974, when a fall of brickwork occurred at Chainage 243, this inner doorway was bricked up by the University leaving an access panel and air bricks.

British Railways retained the ownership of the remainder of the tunnel until it was sold to the University in 1963. The north portal continued to remain open until March 1967 when a timber frame covered with chain link fencing and with a door was fitted. This was broken down repeatedly and was replaced by a brickwork partition with air bricks in March 1969.

In May and June 1974 local falls of the inner surface of the lining occurred within Chainages 240 to 270 and these were followed by breaches of the tunnel sides, leading to the ingress of clay, on 3rd and 11th July 1974. A more detailed account of these events is given in Section 2. During the subsequent grouting work the tunnel was filled from the north portal to Chainage 495 and a 150mm diameter ventilation shaft was installed at chainage 502 to generate a degree of air movement along the unfilled section between Chainage 495 and the south portal.

4.3 Form and Construction

4.3.1 Size

The tunnel appears to have been straight on a uniform gradient of approximtely 1 in 54 rising to the north (Whitstable end).

The length is variously stated as being 764m (Pasley), 757m (Fellows, Hamilton Ellis), 775m (British Railways), 769m (Ove Arup) and 771m (Farmer and Dark). In view of the fact that as part of the Farmer and Dark survey in 1973 the tunnel was marked up at every 10 metres with white painted dimensions, the value of 771m has been adopted for this report and all references to chainages are based on their survey.

From the Maunsell report, the height of the tunnel, between the crown and the top of the invert fill material, varied along its length from that measured at the south portal of 3.86m to that at the north portal of 4.1m. The variation of the crown level over the failure zone is illustrated on Fig. 2B. This is based upon reduced levels taken during the Arup survey and relative levels taken by the University in June 1974.

The width of the tunnel as recorded in the Farmer and Dark report showed considerable variation throughout its length. This is illustrated on Fig. 2A.

4.3.2 Shape

The shape of the tunnel also varies between the two ends. The shape from the north portal to Chainage 465.5 is shown in Farmer and Dark's Report as being basically elliptical in section (except, of course for the deformed section where the width of the tunnel was narrower) with a curved brick invert (see Fig. 1) The remainder of the tunnel was constructed with more or less vertical walls on stepped footings. The Farmer and Dark investigation indicates that the southern end of the tunnel passes through sand while the north end passes through London clay. A report in the Kent Herald, 18th May 1826, refers to the soil on the (Canterbury) side of the hill as being cut through a bed of white sand. Although not established exactly, it would appear that the two different forms of construction of the tunnel relate to the different strata through which the tunnel passes. In his 1846 report Pasley commented, "the northern end of the tunnel, having been excavated in stiff clay, has an inverted arch, and the workmanship of this part is much better than the remainder to the southward, which has no invert, and of which the side walls are irregular."

4.3.3 Method of Construction

Unfortunately, no records of the actual method of construction have been found, although Sir Harold Harding has described the most likely technique in his report (Appendix C). It has also been possible to build up some picture of the general strategy of the tunnel construction and the problems encountered from various works on the history of the railway, and newspaper accounts.

From these various sources, it appears that the tunnel was driven from opposite ends starting from the north. There is no reported evidence of vertical shafts having been used during construction - indeed the Kent Herald when reporting of the meeting of the opposing bores in May 1827 remarked, "the situation of the excavators has been truly distressing for some time, in consequence of the stagnated state of the air".[1] Likewise, there is no indication of a vertical ventilation shaft having existed in the permanent work (see report for 11th July 1925 in Appendix B). No evidence has been found to suggest that the tunnel was driven off course and thus that a dummy length might exist behind the lining, and the same report from the Kent Herald mentions, "it appears, so nice was the calculation of the engineer, that although the line of the rail is more than 2400 feet in length, it has been preserved to within an inch."[2]

Although reports of any major catastrophies in the tunnel construction could not be found, on several occasions in 1826 the Kent Herald refers to both overbreak and falls. The first, on 18th May in reference to the discovery of iron ore during construction of the railway in the valley of Tyler Hill, remarks, "it will not be made available to any other purpose than that of blocking over the arch of the tunnel to prevent the descent of the superincumbent earth."[3] The second, on 28th September, reports "Upwards of 400 yards are already finished without any material accident having occurred, although several falls of earth have hindered and injured the bricklayers and workmen."[4] The third, on 4th October, records, "a considerable portion of the curiously constructed tunnel on the railway on the Tyler Hill side bodily depressed into the earth a few days since in a very extraordinary manner"[5]

In "History of Southern Railway" , C.F. Dendy Marshall makes the following mention of Tyler Hill Tunnel, "No special engineering difficulties were experienced beyond a few falls of earth during tunnelling". Investigations into the possible extent of voids as a result of overbreak and loose blocking over are reviewed in Section 3.7.

1,2,3,4,5 as recorded in "History of the Canterbury and Whitstable Railway", by Rev. R.B. Fellows.

4.3.4 Brickwork Lining

The tunnel was lined with four courses of brickwork to give a total thickness of 457mm (18").

From Figs. 6A, 6B, 7 & 13 it will be seen that the bond varied considerably with English, Flemish and odd variants between the two all existing in close proximity. In addition to these changes in bond, the vertical or perpend joints almost form vertical lines in places. The alignment of the courses along the length of the tunnel is equally erratic. The figures indicate that the joints were generally well mortared up, except that a close-up photograph (not included) indicates that the perpend joints of the outer skin against the clay were not always filled. The figures also show that the bed joint thickness varied and was noticeably thinner on the curved top section. This latter phenomenon might have occurred as a result of the mortar yielding under the forces of the arch with the striking of the drum, (temporary formwork used during bricklaying) or it might have been the practice at the time to build arches with thin joints to avoid just such an effect. This is discussed by Sir Harold Harding in his report (Appendix C).

Although the exterior surface of the brickwork is caked in soot deposits, the interiors of split samples of the bricks are orangey brown (terra rosa) in colour. The interior texture of samples varied between those with a fairly closely knit structure exhibiting minor voids and small intrusions, to those with a distinctly cellular structure and the presence of clinker and other miscellaneous intrusions and even small pebbles. Both types could be scored readily with a blunt knife and pieces could be crushed to dust very easily with a hammer and some even between the fingers. Although this suggests that the bricks have a low crushing strength, the only way of verifying this would be to carry out laboratory compression tests of samples of the brick. Likewise, laboratory tests would be required to determine the porosity of the bricks. The probability is that the bricks were manufactured from brickearth but it would necessitate an analysis of the chemical composition to verify this.

4.4 Maintenance

Almost throughout the entire history of the tunnel, adverse reports on the behaviour of the bricks have been made. In 1846 Pasley reported "the bricks appear to have been bad, as the surface exposed to air has flaked off in some few parts of the arch, which will be replaced with stronger ones."

Unfortunately, records of the tunnel maintenance from 1846 to 1925 could not be found. However, the railway examination reports for the period 1925 - 1962 also refer to scaling of the brickwork as well as to the development of cracks, bulges, and the discovery of drummy (hollow sounding) brickwork. In the final years of rail use, hearsay evidence was obtained that standard 12' 0" sleepers could no longer be installed and had to be cut short to fit inside the tunnel, but it is not known at which section these measures had to be adopted. The last of these reports in May 1962 (included in Appendix B) commented "Side walls and haunches scaled and weathered very badly throughout. Tunnel of irregular shape. Dry and sound but first ring of brickwork perishing rapidly".

The Maunsell report in 1963 confirmed the uneven nature of the tunnel sides and also made reference to the scaling of the bricks which, in some cases, had led to the exposure of the second ring (from the tunnel face) of the arch, leaving mortar joints standing proud as ribs. They also commented on the repairs which had been carried out at various times. over the years with both yellow stock and blue brindle bricks. Finally, in summing up the condition of the tunnel, they recorded, "it is generally dry and sound but the shape is irregular. Many bulges in the side walls and haunches are old, but movement appears to be still taking place, as cracks and bulges are noted in the (British) Railway report of May 1962 which were not recorded in previous reports".

During the period 1963 - 1973 the tunnel was inspected at fairly regular intervals, but with nothing unduly disturbing being noted. However, in November 1973, following the survey and report on the condition of the tunnel in which the state of every 2 square metres was recorded, Farmer and Dark reported, "some sections of the tunnel are in a condition that require very early attention. The section which has deteriorated most seriously lies under Cornwallis......" A comparison of the condition of the tunnel as reported by Farmer and Dark with that recorded in 1962 by British Railways indicates that the total number of significant cracks in the tunnel lining recorded in the 1962 report was 5 and only one of these was in the tunnel length between Chainages 240 and 270, directly beneath the Cornwallis Building. By 1973 the total number of cracks recorded had risen to 50 and seven of these were between Chainages 240 and 270. A comparison between the two reports for the section between Chainages 240 and 270 is given by Figures 3 and 4. Subsequently, the deterioration of certain sections of the tunnel accelerated rapidly, notably within Chainages 240 to 270, leading to falls of the inner surface and finally to the breaches and collapse described in section 2.

4.5 Brickwork Behaviour

In an endeavour to find an explanation for the scaling process, the phenomenon has been discussed with various authorities.

4.5.1 British Railways

British Railways say that it is a frequent occurence in their tunnels and attribute it to attack by sulphuric acid which forms as a result of sulphur trioxide - derived from the products of combustion from steam locomotives - combining with water. It is not unusual, in their experience, for this to occur many years after the steam locomotives have ceased to run through the tunnels. Their policy is to replace the bricks at a limit of 3 inches of scale.

Traces of sulphur trioxide (SO_3) were found in four samples of the brickwork taken from the brick lining in June 1974 for analysis by Messrs. Sandberg Ltd. However, percentages of the order found can be expected in the structure of bricks manufactured from brickearth (ref. Paper 5:Clay Building Bricks of the United Kingdom, published for M.O.W. 1950), and it is not clear from this particular analysis by Sandberg whether the SO_3 was pre-existing or derived from external sources (i.e. the soot deposits).

4.5.2 Brick Development Association

The opinion given by the Brick Development Association is that the spalling could be caused by sulphate attack. Normally this manifests itself on the face of bricks, but if they have a laminar structure then a build up of sulphate crystals on an internal plane would cause spalling. No efflorescence was observed and there appears to be no deposit on the spalled surfaces of the bricks, but microscopic examination and analysis might confirm this.

It would be expected that sulphate or other chemical attack would affect the mortar as well as the brickwork, although the behaviour of the perpend mortar does not confirm this, i.e. it tended to project out after the bricks had disintegrated.

The point was made that prolonged sulphation can cause breakdown of the ceramic bond and consequential reduction in strength.

4.5.3 The British Ceramic Research Association.

The view of the British Ceramic Research Association was that such an acid (as suggested by British Railways) would be more likely to attack the lime (and lime products) of the mortar leading to the formation of sulphates. The results of such a reaction could be the swelling of the mortar with a consequent pressure on the bricks, leading to their disintegration. They considered that the bricks would be relatively inert and would be unlikely to be affected by such acids. However, tests could be carried out to establish the presence of sulphates in the mortar.

4.5.4 Conclusion

The conclusion is that the deterioration and scaling of the brickwork most probably resulted from chemical attack derived from compounds formed from the soot deposits in the presence of water. However, more investigation and analysis would be required to determine a more exact answer to the actual chemical processes.

MECHANISM AND CAUSES OF SUBSIDENCE

5.1 Mechanism of the Subsidence

5.1.1 Evidence from the soils investigation

The following evidence has been drawn from the information gathered during the site investigation, described in Section 3.

i) The region between Chainage 240 and 270 previously occupied by the disused railway tunnel was found to be blocked by a voided mixture of brick, clay, sand and some pfa. Clearly this mixture resulted from the failure of the tunnel lining and consequent ingress of clay and overbreak fill material together with the subsequent introduction of pfa into the zone of the tunnel.

ii) The clay in the subsidence zone above the collapsed length of tunnel was substantially intact. Although it contained many fissures and shear planes, no actual voids were encountered by the boreholes. There was no evidence discovered to suggest that the surface subsidence was caused by a cavity progressively caving in and rising to the surface.

iii) There was no marked variation in the measured undrained shear strengths which ranged from 50 kn/m^2 to 150 kn/m^2 with depth, or appearance of the intact London clay at any of the locations explored, including those outside the subsidence zone, except that in the zone of the subsidence the fissures were more open and the undrained shear strength was slightly lower than elsewhere.

iv) The London clay at all locations exhibited similar liquid and plastic limits: the moisture content also showed no marked difference at any location except immediately around the tunnel where it was approximately 10% higher than elsewhere. This represents an increase of 40% in the weight of water present in the clay.

v) At locations explored on the tunnel centre line where the tunnel was still intact but filled with pfa, no evidence of significant cavities was found, but a zone immediately above the tunnel was found in places to be capable of taking small volumes of grout. This suggests the presence of previous loose filling of overbreak during the tunnel construction which is substantiated by press reports of the tunnel construction.

5.1.2. <u>Mechanism of the subsidence</u>

The mechanism of the subsidence, which seems to fit the above evidence and the events in the tunnel and on the surface, is that as a result of the breaches in the lining and the ingress of clay into the tunnel a cavity developed. When the size of the cavity reached a critical value the clay above failed as a plug in undrained shear.

Clear evidence of the formation of a cavity is given by the Surveyor and Deputy Registrar in his eye witness account of the state of the breach at Chainage 243 on 3rd July 1974 (Illustrated by Figure 16) During the following days this breach increased in size with more and more clay falling into the tunnel until, by the 9th July, the tunnel was completely blocked (see Figure 11) Figure 12 suggests that this void still existed on 9th July since the clay had not bulked up completely to the roof. With the advent of further breaches in the lining at two locations within Chainages 260 to 268 and the consequent ingress of clay during the afternoon of 11th July, further cavities would have occurred. As they developed, probably uniting with the earlier cavity at Chainage 243, a critical condition was reached and the ground subsided as a plug on the evening of the same day. (Possibly as shown in Figure 35).

Calculations have been performed on an idealized model of this situation and it has been shown that, assuming the undrained strength of the clay as measured after the subsidence is approximately 70 KN/m^2, it is possible for the clay to fail as a single plug in undrained shear when the cavity perimeter reaches a critical value equivalent to a length of 12m of collapsed tunnel.

Figure 33 shows the factor of safety against failure in undrained shear for clay plugs of various lengths and values of shear strength. Also shown is the effect of a surcharge of 10 KN/m^2 (200 lbs/ft^2) which has been assumed to represent the approximate weight of the Cornwallis building. It can be seen that the effect of the weight of the building is insignificant and hence the accuracy of its assumed value is immaterial.

5.2 Mechanism of the Tunnel Lining Failure.

From an examination of the photographic evidence taken during May to July 1974 and the eye witness accounts of events over this period, it is concluded that the tunnel lining failed as a result of the breakdown of the arch action leading to a failure of the side walls in bending. Figure 6B clearly illustrates a bending failure at Chainage 260.

The only way such a failure in the sides of the tunnel could occur would be for either the crown or the invert to yield. Cracks in the ash filling over the invert, suggesting heave, were found in May 1974 and on 10th June 1974 failures in the invert at both Chainages 248 and 256 were uncovered (see Figures 9 and 10). In contrast no indication of failures in the crown or its haunch were noted during this period. It is true that the relative levels of the crown taken in June (see Figure 2B) seem to suggest some discrepancy but it could have been either pre-existing or due to a failure in the invert causing a drop in the whole section of the tunnel.

The following mechanism for the failure of the tunnel lining is proposed.

First the invert failed and this resulted in the removal of the effective haunch to the side wall arches of the tunnel. These then deformed under the load of the clay behind and finally failed in bending. This led to the ingress of clay into the tunnel and the formation of cavities which precipitated the subsidence in the manner described in 5.1.2 above. This sequence is illustrated in Figure 32.

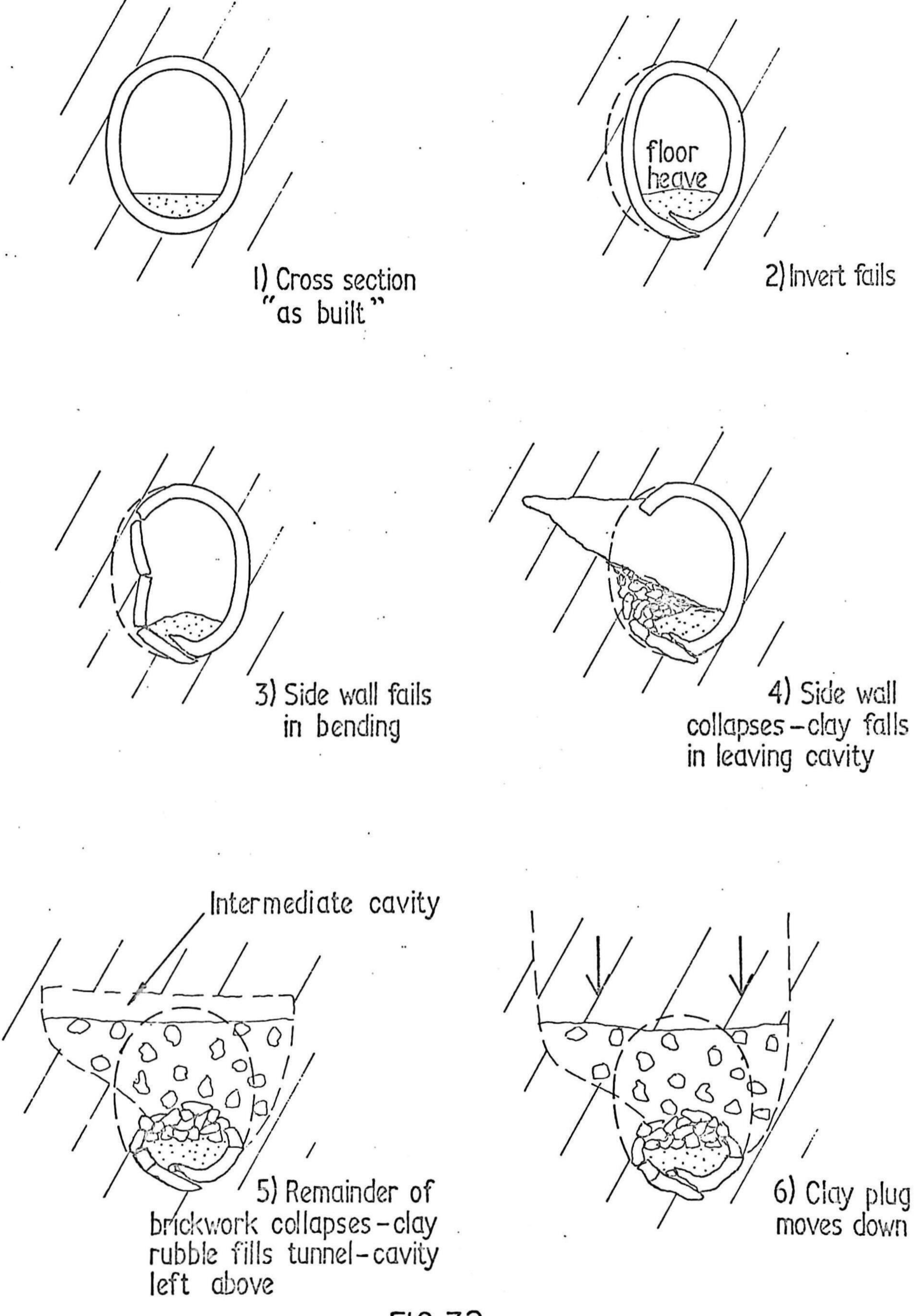


FIG. 32
Possible collapse mechanism

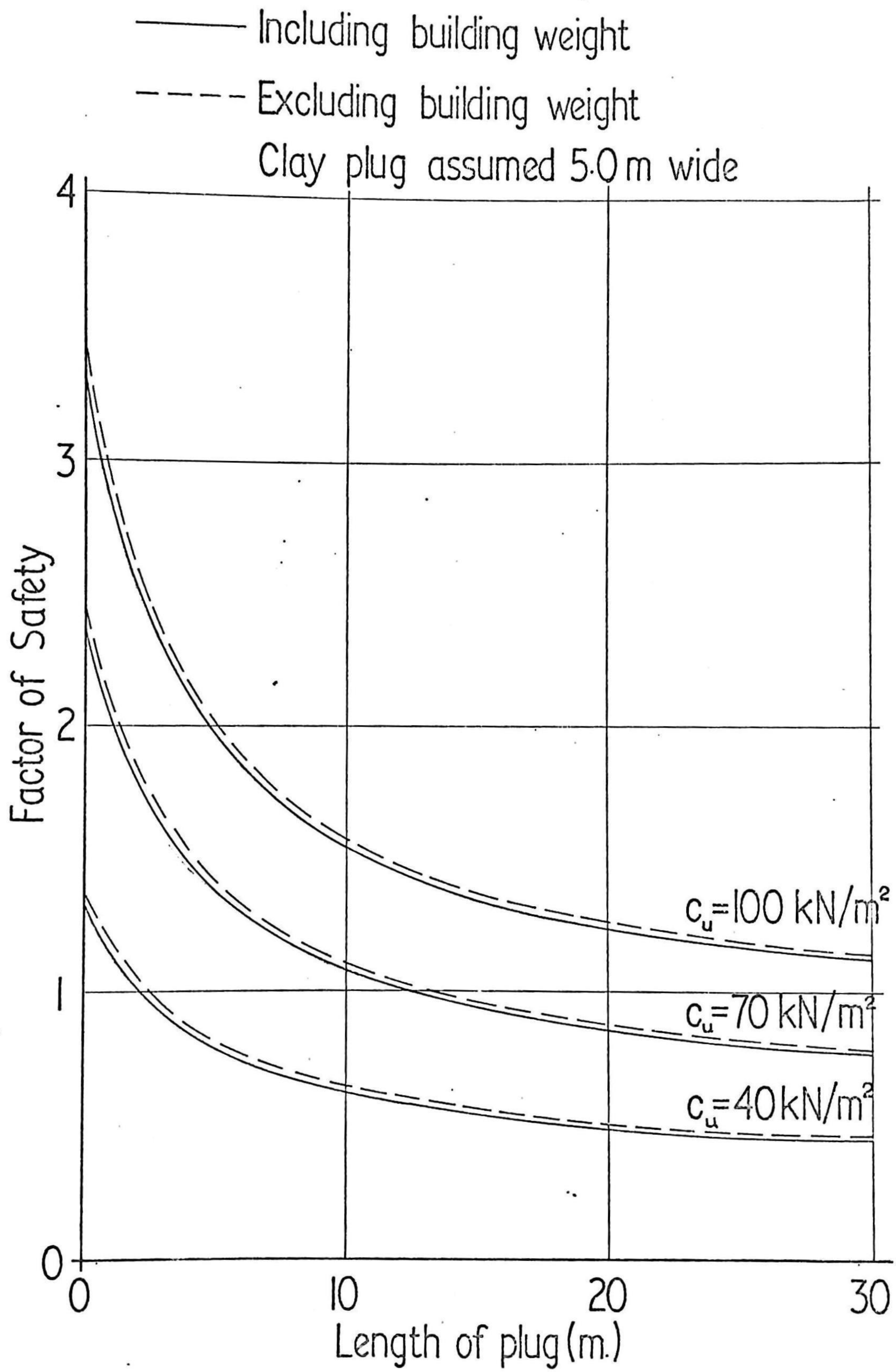


FIG: 33

Graph of Factor of Safety against plug size for varying shear strength

5.3 Possible Factors affecting Lining Failure

5.3.1 Invert failure

The most likely explanation is that the bricks in the invert were imperceptibly weakened because of their location below the invert fill. This led to the failure of the brick invert as the "weak link" in the arch system of the lining. Since the tunnel has been filled over the total length where the invert existed, it is not now possible to establish the relative strengths of the brickwork in the invert and the side walls and hence to verify this proposition.

5.3.2 Scaling of the brickwork

The British Railways 1962 report, the Maunsell report and the Farmer and Dark survey and report all draw attention to the deterioration of the inner face of the brickwork. However, the photographs show that this was not extensive. Obviously any reduction in the area of the brickwork would lead to an increase of stress even under constant load condition. In this case the scaling of the brickwork is not considered to have had any real significance on the breaches in the lining, since it has been concluded that the side brickwork failed in bending not in compression.

5.3.3 Pressures on the tunnel lining from the clay surround

From the reports and measurement of bulges in the tunnel lining where it passed through the clay stratum, it is evident that the clay exerted pressures in addition to static loading on the tunnel lining. Although British Railways state that this is not unusual in ventialted tunnels which pass through clay, the possibility of changes in the ventilation after the tunnel was closed to traffic in 1953 might have hastened a process of clay swelling.

This hypothesis is based on the fact that when water evaporates from the surface of a material with a low permeability, such as clay or brickwork, and is replaced at a rate which is less than the rate of evaporation then the porewater pressure is reduced to a value below hydrostatic. Thus, if the surface of a low permeability material which is saturated under an hydrostatic head is well ventilated, the evaporation of water from this surface will draw water out of the material. In consequence a region of sub-hydrostatic pressure will be set up because the low permeability of the material will prevent the replacement of the evaporated water.

The north end of the tunnel was driven through London clay (which has low permeability) and was reported to be dry, although the surrounding clay was fully saturated. The brick lining was porous and therefore water would have tended to flow into the tunnel at a very slow rate.

Now if the rate of evaporation exceeded the in flow rate, as evidenced by the general dryness of the tunnel, then suctions as described above would have been set up in both the brickwork and the surrounding clay. It is not possible to evaluate the magnitude of these suctions since they would depend on the rate of inflow (and hence on the permeability of the clay, the hydraulic gradient to the tunnel, and the permeability of the brickwork particularly if this was only partially saturated) and the rate of evaporation (and hence on the air humidity and air velocity). However, a guide to the possible magnitude of the suctions can be obtained from those developed in clays exposed in dry weather where values of order one atmosphere have been measured.

These suctions, when developed, would give rise to increased effective stress and hence increased strength in the clay.

If the ventilation and air humidity conditions were then changed, say by the loss of the ventilating action of travelling trains and also the building of cross-walls, then the rate of evaporation would decrease and the pore suctions would be reduced. This would have three effects:

i) the effective stress in the surrounding clay would be reduced;

ii) the surrounding clay would swell and impose additional load onto the tunnel ining;

iii) the presence of sustained dampness could lead to the deterioration of poor quality brickwork, particularly where there were soot deposits with compounds capable of conversion in the presence of water to chemicals harmful to brickwork.

A simple analysis of this situation has been performed and from this it has been shown that if the suction drops from a value of one atmosphere to zero pressure over a region defined by a circle diameter twice that of the tunnel, then the load on the lining will be approximtely doubled.

Two pieces of evidence might be taken to support such a proposition. First, the water content in the clay near the tunnel in the subsided zone was found from the soils investigation (Figure 29) to be up to 10% greater than elsewhere in the clay at this depth. However, the soils investigation followed the tunnel filling operations and as this was a distinctly wet process it is quite possible for the clay to have been affected as a result, especially as the clay was highly fissured in the susbided zone and it was not possible to seal the plastic lining tube into the tunnel lining as elsewhere, since the tunnel had collapsed. This argument is not supported by the moisture contents of two samples taken on 21st July 1974 prior to the filling operations. The average value of these was 29.4%. Second, the bulging and general deterioration of the tunnel lining increased after the reduction in the ventilation from about the mid 1950s. In answer to this, those inspecting the tunnel during this period did not recall a noticeable increase in dampness. Furthermore, the bulging of the tunnel lining could have resulted from natural processes in the clay, as experienced by British Railways in other ventilated tunnels through clay.

In conclusion therefore the ventilation hypothesis should be treated chiefly as an intriguing possibility and one which, without an expensive long term research programme, must remain not proven.

5.4 Other Factors

5.4.1 Presence of Pre-existing "backs" in the clay

The reported falls of clay during the excavation of the tunnel suggest that a possible explanation for the subsidence is the movement of the clay above the tunnel on pre-existing failure planes or "backs" possibly formed or loosened at the time of construction. The weight of the building on the ground surface would be the the "last straw" which caused the delicately balanced failure planes to recommence movement and thereby crush the tunnel. Such "backs" would have been lubricated by free water from the surface over a long period.

However this explanation is unsatisfactory for the following reasons:

i) The failure was 6 years after the construction of the Cornwallis building. It is unlikely that a delicate balance of forces could have remained for that length of time. It could of course be argued that the brickwork was just strong enough at the time of construction but that with deterioration over 6 years the failure condition was reached. However, the clay would have been subject to consolidation and consequential gain in strength over the 6 year period and this phenomenon would counter the argument. Also in view of experience at Waltons Wood it is more likely that the previous failure surfaces or "backs" would have been reactivated very soon after the construction of the building.

ii) If a previous failure had been reactivated by the weight of the Cornwallis building and this had caused the brick lining to fail then it would be expected that this movement would continue to complete the subsidence.

   The initial falls in the tunnel left a cavity above the rubble for about 8 days. Thus, unless the movement was very small and could not be readily detected, the reactivated failure mechanism seems to have remained stationary after moving sufficiently to break the arch. Clearly once the arch had ceased to give support then the failure should have accelerated not remained stationary.

iii) The fact that the failure occurred under Block B alone is not easy to explain but if the building weight is considered as a major contributory factor then surely the tunnel lining under Block C and the Rutherford College should have suffered equally if not to a greater extent. This argument relies on the assumption that the condition of the clay under these two buildings was the same as that at Block B. However, if the clay was weaker under Block B or that was the only location, out of the three, where the "backs" existed then coincidence could be blamed and proof or disproof of the theory is impossible.

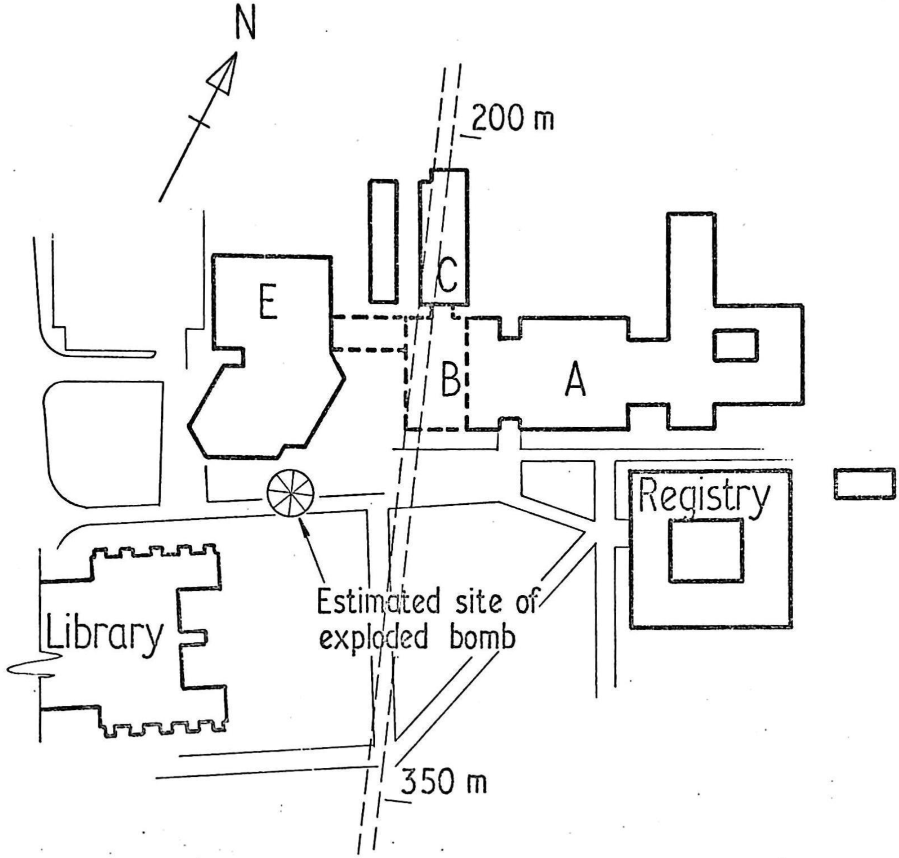
iv) From the eye witness account presented in section 2.1 it is possible that the initial fall at Chainage 243 parted on the line of a horizontal "back". However this neither proves nor disproves the hypothesis that the building weight caused movement of this "back" or any other.

5.4.2 Investigation into reported bomb

Statements were made by three separate witnesses, two of them independently, concerning the existence of a bomb crater following the explosion of a bomb at or near the site of the subsidence during the second world war. These statements were made to the University Surveyor and Deputy Registrar.

A copy of an aerial photograph of the site, taken in 1946, was carefully examined and this showed a discolouration of the ground surface at approximately the position indicated by the eye witnesses. When plotted as accurately as possible on a plan of the University site this mark falls at the position indicated on Figure 34. There is no absolute evidence that this discolouration of the ground surface was in fact the site of the bomb crater but in any event it is considered that at the location plotted it could not have had an effect on the tunnel more than thirty years after the explosion unless it had considerably weakened the ground and the tunnel lining at the time of the explosion. Other than some seepage of water reported at the time (information obtained verbally from a retired railways plate layer) there is no other evidence of this.

No evidence of shrapnel or other fragments was found during examination of borehole samples. Investigations were considered into the possibility of detecting any remaining chemical evidence of the explosion, but it was decided that after thirty-five years this was not really practicable. In June 1975 the University staff excavated a trench as near the probable



NOTES.

Exploded bomb site estimated from aerial photographs to be 21·5 ± 1 m. west of tunnel at 283·5 ± 2·5 m.

Crater approx 10m diameter.

FIG:34

Approximate location of exploded Second world war bomb site of the exploded bomb as possible (existing services and a concrete duct prevented the exact location of the trench). This trench did not reveal anything of real significance to corroborate the existence of a bomb crater.

5.4.3 Leaking drainage pipes

During the remedial works, evidence was found that at several points 9" surface water pipes had been discharging directly into the hoggin and London clay at or about the level of their interface. However, this is not thought to have had a serious effect on the foundation situation since the hoggin cap was saturated for most of the year and the additional discharge was only that which had been collected from the surface and which, prior to development, would have seeped into the hoggin anyway.

5.4.4 Unfilled overbreak

Further to the newspaper report of the construction of the tunnel in Section 4.3.3 ref. 3, the possibility of any unfilled overbreak existing outside the tunnel lining was considered. During the course of the soils exploration some evidence of filled overbreak was found but no large cavities were detected. In any event since it is clear that a large cavity could not stay open in the clay for any period of time (as proved by the speed with which the subsidence followed the lining failure) it is unlikely that a cavity dating from 1828 could have affected the tunnel at the time of the subsidence. It can be argued that a moderate cavity could have remained open and that the additional load imposed by the Cornwallis complex finally produced unstable conditions. However, if this were the case then, as with the "backs" theory, it is unlikely that the effects of this would be felt some 6 years after the imposition of the load. The weight of the building may, however, have acted as a stress raiser and this could explain why the tunnel finally failed at Chainage 240 - 270 and not at Chainage 80 where the bulging of the walls indicates deterioration of at least equal magnitude, but does not explain why it did not fail under Rutherford where the weight on the tunnel was equal if not greater than that under Cornwallis.

The difference between the volume of the ground surface depression and the collapsed length of tunnel (see section 2.4) can be explained by the bulking of the clay debris which fell into the tunnel and the possibility that some portions of the tunnel between Chainage 240 and 270 may not have collapsed completely and thus cavities may have been left between the locations where complete collapse occurred. The quantities of grout injected into the subsidence zone at the tunnel level during the tunnel filling and remedial grouting works confirm this possibility.

5.4.5 Possibility of tectonic movement

Enquiries were made to the Seismological Unit of the Institute of Geological Sciences, Edinburgh, which has no record of any significant earth tremor in the area during this period. They are of the opinion however that no accurate seismological data can be obtained from such a restricted period of time, and any readings should be supplemented by a search of pre-instrumental records. Distance of the Unit from the Canterbury area is also a problem. The nearest monitoring station in England, M.O.D. (PE) AWRE., Aldermaston, Berkshire, has no record of seismic activity for that area either, but points out that even if a small movement had been noted by them, it would be difficult to attach any significance to it just as a single reading from one station.

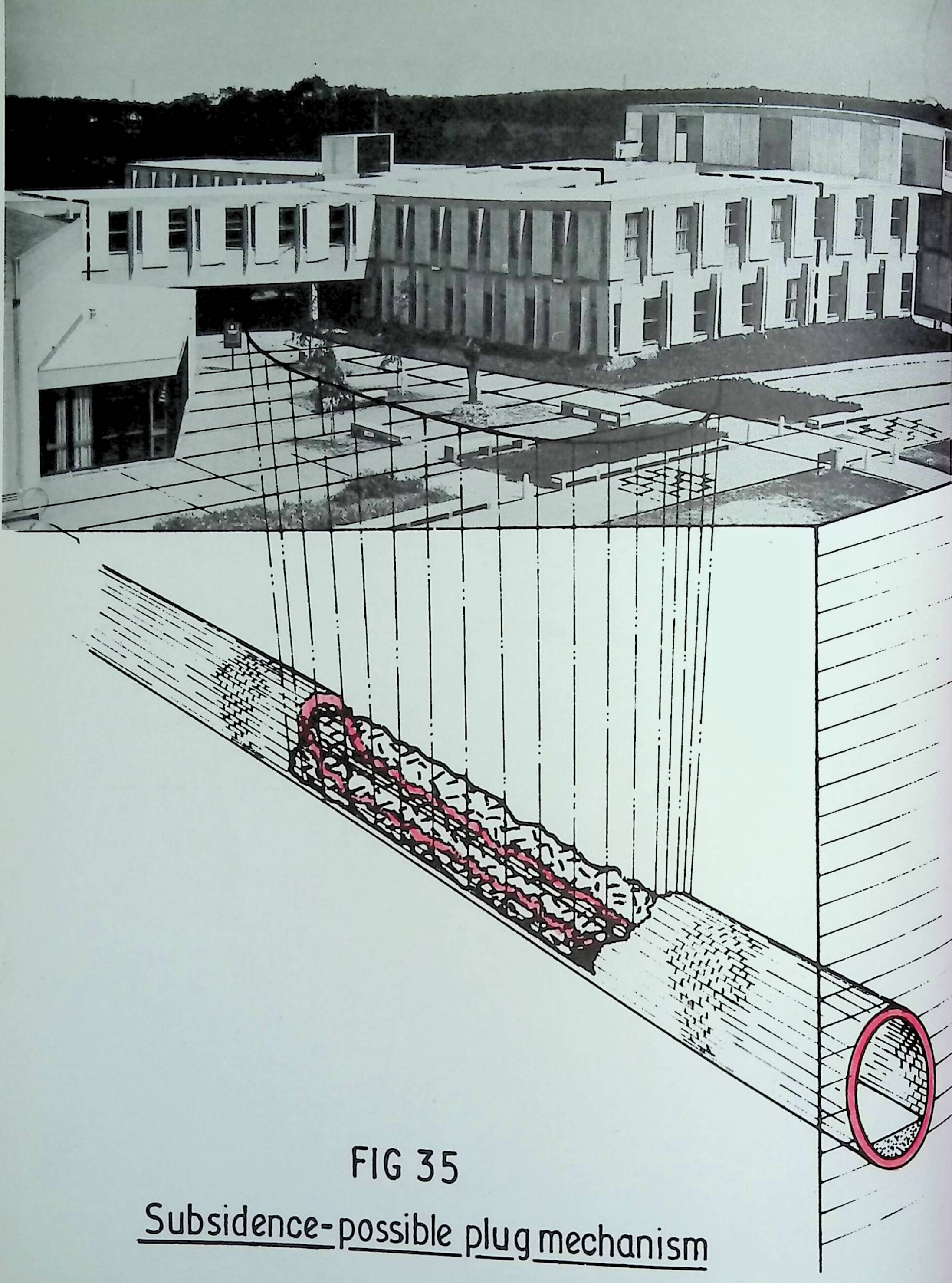


FIG 35

Subsidence-possible plug mechanism

5.5 Conclusions

5.5.1 Mechanism of subsidence

The recorded sequence of events in both the Cornwallis building and the tunnel below indicates that the subsidence was directly linked with the failure of the tunnel lining. Furthermore, it is evident that the subsidence was a result of, and followed, the failure of the section of lining from Chainage 240 to 270.

We are of the opinion that the mechanism of the subsidence which fits all the available evidence is that as a result of breaches in the lining and the ingress of clay into the tunnel a cavity developed. When the size of the cavity reached a critical value the clay above failed as a plug in undrained shear. There was no evidence from the soils investigation to suggest that the subsidence was caused by a cavity progressively caving in and rising to the surface.

5.5.2 Mechanism of the failure of the tunnel lining

In our opinion the failure of the tunnel lining within Chainages 240-270 occurred as a result of the breakdown of the arch action in the walls leading to failure of the brickwork in bending and consequent breaches.

The available evidence indicates that the breakdown of the arch action in the walls was probably caused by failure in the invert. It has not been possible to establish clear reasons for this failure, and it is not certain even with a further extensive research programme that the true cause or causes could be found.

5.5.3 Other factors considered

The scaling of the brickwork, although serious in places, is not thought to have been a direct cause of the lining failure since there is clear evidence that the side walls failed in bending. However, the reduction in the cross sectional area of the brickwork may have caused local weakness.

The theory that pre-existing "backs" in the clay moved under the load of the building above is difficult to prove or disprove. The principal argument against this theory is that if a previous failure surface or "back" had been reactivated by the weight of the Cornwallis building and this had caused the brick lining to fail, then it would be expected that this movement would accelerate as the resisting force from the lining reduced. In fact after the first breach of the tunnel lining, a cavity is known to have existed for eight days before the subsidence in the evening of the 11th July 1974 occurred. On balance therefore it is considered that this theory cannot be substantiated.

The reported second world war bomb and leaking drainage pipes are not thought to have contributed to the failure. Likewise tectonic movements and unfilled cavities in the clay surrounding the tunnel have been discounted.

EVALUATION OF THE SITE

6.1 General

6.2 Land Over the Backfilled Tunnel

6.3 Land Over the Unfilled Sections of Tunnel

6.4 Land Over the Zone of Subsidence

6.5 Conclusions

6.1 General

The history of the present site at Canterbury, since it was acquired by the University, shows that until the failure of the tunnel lining within Chainages 240 to 270 no major engineering difficulties were encountered. There had been no significant movements of the ground or any of the structures placed thereon. The 1974 site investigation and exploration have revealed no evidence of any conditions likely to adversely affect present or future structures other than those above the tunnel (including any future buildings on the site of the subsidence itself). We therefore conclude that outside the areas influenced by the old tunnel and the zone of subsidence the site can be treated as "normal" and only those precautions and investigations undertaken on any normal London clay site need be pursued. Clearly any measures adopted for site exploration and design of foundations away from the tunnel centre line must be considered in the light of the proposed structure, any new information obtained from a site investigation and any other circumstances peculiar to the particular scheme.

There thus remain three further areas to be evaluated:

i) the strip of land over the back filled railway tunnel;
ii) the strip of land over the unfilled railway tunnel; and
iii) the zone of subsidence.

6.2 Land Over the Backfilled Tunnel

The structural integrity and maintenance of existing buildings over the backfilled sections of the old railway tunnel will depend in the main on the behaviour of the London clay and also on the success of the tunnel filling operation. In addition to any settlements and movements which can be predicted from the measured properties of the clay several further possibilities and conditions have been considered.

6.2.1 The possibility of a small void existing just below the crown of the tunnel if the tunnel has not been completely filled.

Several boreholes have been put down to investigate this possibility and in no case was a void found. To augment this drilling data, grout injection at the tunnel crown level was attempted at several locations. From the small quantities injected under hydrostatic head it can be concluded that at the locations investigated no such void existed.

However, if such a void did exist at any location not investigated it is likely that it is of insignificant volume compared with that involved in the original failure. This small void, if in existence, would be supported by a well haunched short span arch of brickwork which is extremely unlikely to collapse. Even in the event of the failure of such an arch the effects on the surface are likely to be insignificant.

6.2.2 <u>The possibility of unfilled overbreak outside the tunnel lining.</u>

Again grout injection tests have shown this to be insignificant at the locations tested. If, however, poorly compacted or unfilled overbreak existed at other points it is unlikely to subside further now that the tunnel has been backfilled.

6.2.3 <u>The possibility that the filling of the tunnel has significantly changed the drainage pattern of the London clay in the vicinity of the old tunnel.</u>

The effects of any change in drainage are at present the subject of a piezometric investigation. The results so far obtained from the piezometers installed during the course of the remedial works indicate that no great porewater pressure differentials exist in the bulk of the clay. We therefore conclude that any consolidation or swelling of the clay due to porewater pressure equalization will be of small magnitude.

6.2.4 <u>The possibility that movements initiated prior to tunnel filling might still affect the surface.</u>

The tunnel walls can no longer yield and therefore new movements due to this cause can be ruled out. Evidence from the Farmer and Dark Report and previous reports suggests that tunnel movements prior to the failure of the tunnel lining were of small magnitude and therefore it can be inferred that any surface movements would likewise be very small. In any event surface movements associated with yield of the tunnel walls are likely to have taken place very shortly after any such yield and hence there is no real danger of significant future movements from this cause.

6.2.5 The possibility that long term movements of the ground surface due to the initial construction operation to form the tunnel might still be taking place

Movements associated with local softening of the clay due to stress relief, caused by the presence of the tunnel and any overbreak during construction, could affect the ground surface but these movements are likely to be complete by now.

More general subsidence or movement of the clay above the tunnel during its construction (see section 4.3 ref. 5) could have created conditions similar to those which existed in the subsidence zone before the compaction grouting. In consequence some regions of clay may have been subject to changes in moisture content and compressibility giving rise to long term surface movements. We have concluded that after 150 years any further movements due to these causes are likely to be small and of an order normally accepted for buildings constructed on London clay. The satisfactory performance of the Cornwallis building during its early life prior to the failure of the tunnel lining supports this view.

6.2.6 Summary

It is our opinion that the sum of the movements considered above (sections 6.2.3, 6.2.4, and 6.2.5) is unlikely to be of great significance. However it would be prudent when considering future buildings spanning the filled sections of the tunnel, to allow for differntial settlements of up to twice those normally assumed for the building load on a similar but undisturbed London clay site. Further detailed investigation of a particular site might render this conservatism unnecessary.

The design and orientation of existing buildings located above the filled sections of the tunnel are such that future maintenance problems are likely to be little greater than for a normal London clay site.

6.3 Land Over the Unfilled Section of Tunnel Chainage 495-737

This land is to the south of the Rutherford College and slopes down towards Canterbury. The tunnel here is located in the Oldhaven sands which underlie the London clay and if the lining were to fail the mechanism could be quite different from that experienced under Cornwallis. It is likely that any subsidence would be more abrupt, with the sand flowing into the tunnel to form a swallow hole. Depending on the location of the failure the clay mantle above might simply sag or else fail in undrained shear with a plug of clay subsiding into any void left by the subsided sand.

The University main storm and foulwater outfalls cross the line of the tunnel at approximately Chainages 665-680 and in the event of a failure these are at risk. Recommendations for filling the tunnel over this length have been made to the University of Kent elsewhere.

It is unlikely that any developments of this sloping ground will ever be considered. However, if this were ever contemplated the tunnel beneath any proposed foundations should be completely backfilled.

6.4 Land Over the Subsidence Zone

This area of ground has been subjected to large movements and the site exploration has revealed considerable fissuring, slicken sided surfaces and shattered zones. The remedial grouting has substantially filled any voids or cavities at the level of the collapsed tunnel section and above this level the compaction grouting will have closed open fissures and voids and will have restored the horizontal stresses in the ground to a value equal to the vertical overburden pressure. From the fact that at the end of the grouting programme the complete soil mass could be forced to move upwards by a $0.1m^3$ injection of grout it can be concluded that the ground in this region has been substantially stabilized.

Movements of the magnitude of the original subsidence can no longer take place, although minor movements due to the redistribution of porewater pressure and ground stresses can still occur and are being monitored by regular observations.

6.4.1 Movements due to redistribution of porewater pressure

As reported in section 3.5.3 the piezometer results obtained so far indicate that the porewater pressures in the subsidence zone have now equalized with the values in the adjacent areas. It is therefore likely that future ground movements due to this cause will be of small magnitude only.

It is advisable that the piezometric measurements and monitoring of the site should be continued so that an assessment of actual movements can continue to be made. The results of such measurements, taken over a sufficient period of time, can then be extrapolated and the effect on any future structure estimated.

6.4.2 Movements due to redistribution of ground stress

The clay in the subsidence zone has suffered severe disturbance. It has only been possible to restore the horizontal stresses in the ground to a value equal to the vertical overburden pressure which is less than the equivalent value in the undisturbed clay. Higher values of pressure used in compaction grouting would only result in surface heave. Consequently small deformations must be expected to continue for some time as the ground adjusts to this modified stress condition, and a slightly higher moisture content and higher compressibility may be expected. As a result any building constructed across this region of ground will be subject to an increased differential movement as compared with a virgin site.

6.4.3 Rebuilding on subsided ground

Despite the fact that porewater pressure equalization is almost complete any new building on this ground should be considered in the light of 6.4.2 above. Special precautions should be considered so that the new structure will not suffer structurally from any future ground movement or require excessive maintenance. Such precautions might be to design a flexible building on normal foundations or else to provide foundations which would be able to resist the effects of any future ground movements.

The total of the likely diffential vertical movements due to the causes described above may be up to twice that which would normally be assumed for a building elsewhere on the campus. In addition small lateral ground displacements may occur. It is therefore suggested that if reinstatement of the demolished building on this site is required then the new building should be articulated in such a manner that it can handle differential movements of at least twice those normally catered for. A steel framed structure could be designed to accommodate the likely movements. Alternatively a building similar to that which had to be demolished could be built on the site if it was founded on deep piles which were sleeved through the disturbed zone.

The buildings adjacent to the subsided zone of soil have now been substantially protected by the effects of the compaction grouting. Some small movements may take place in the future but these should be well within normal maintenance tolerances.

6.5 Conclusions

i) The buildings situated above the filled sections of tunnel are now safe from catastrophic settlement. However, future maintenance may be marginally greater than that normally required for such buildings founded on a uniform stratum of London clay. Any proposed buildings to be constructed across the filled sections of the tunnel may need to be designed to cater for differential settlements of up to twice those normally assumed, but further investigation of a particular site might show this conservatism to be unnecessary.

ii) The subsidence zone can support the loads of a building similar in weight to that demolished if suitable construction and foundation techniques are employed. The design should assume that differential movements will be at least twice those normally allowed for if a shallow foundation is adopted.

iii) The buildings adjacent to the area of subsidence have been protected by the effects of the compaction grouting and any future movements should be within the limits of normal maintenance.

iv) No reason has been found why the remainder of the campus, remote from the tunnel, should be treated as in any way unusual. All normal site exploration procedures and foundation design investigations should of course be undertaken for the particular site in question, when further building development is proposed.

Note should be taken of the precautions necessary on the sloping ground to the south of the campus.

PREVIOUS SURVEYS AND REPORTS

Previous Surveys and Reports

For completeness the previous recent surveys and reports are listed below. In addition, reports on three site investigations carried out on the University campus are also listed.

Report

1962	Proposed site at Canterbury for the University of Kent - Ove Arup and Partners
1963	Report on old railway tunnel under site - G. Maunsell and Partners.
1973	Survey of old railway tunnel under the site of the University of Kent - Farmer and Dark.

Site Investigation Reports

1964	Report on site investigation for proposed new building off Giles Lane - Marples Ridgeway.
1965	Report on trial borings for proposed new chemistry building in new Giles Lane - Marples Ridgeway.
1968	Site investigation report - Soil Surveys Ltd.

HISTORY OF THE OLD RAILWAY TUNNEL

N.L. DURBIDGE

Subsidence

Report on enquiries made with the object of ascertaining all information relating to the construction and upkeep of the tunnel, either by explicit information such as constructional drawings and records, or by more indirect information from other sources.

1. The list of all sources where enquiries have been made is as follows:

   Mr. Charles Lee (Transport Historian)
   Professor T.C. Barker
   Local newspaper offices
   The City Reference Library (The Beaney Institute)
   The University Library
   The Cathedral Library
   The County Archivist
   British Rail Historical Records Department
   British Rail Civil Engineers covering Works Maintenance and Bridges and Tunnels
   Messrs. Kingsford, Arrowsmith and Co. (Kingsford was Solicitor to the original company which undertook the promotion of the Canterbury and Whitstable Railway)
   The former City Librarian (Mr. F. Higenbottam)
   George Stephenson Collection (Chesterfield Public Library)
   Darlington Reference Library
   Newcastle Library (Local History Department)
   Institution of Civil Engineers
   The Permanent Way Institution
   The Railway and Canal Historical Society Journal
   "Railway World"
   Railway Museum, York
   Canterbury Archaeological Society
   City of Liverpool Museum (Transport Section)
   Science Museum (Department of Civil and Mechanical Engineering)
   "Railway Magazine"
   The British Museum Newspaper Library
   Harris Public Library, Preston
   Picton Library, Liverpool
   Loughborough University of Technology Library
   Institute of Geological Sciences
   Institute of Mechanical Engineers

Messrs. Ian Allan (Publishers of Railway Books)

Mr. John Parkinson (Author of articles on Cornwallis Building collapse in "New Civil Engineer"

Mr. I. Soilleux (former Clerk of Bridge-Blean Rural District Council)

Mr. I. Maxted (Author of "The Canterbury and Whitstable Railway")

2. The search for information on constructional drawings and records was singularly unsuccessful, possibly due to the loss during the last war fromthe Royal Museum, Canterbury (The Beaney Institute) of most of the records of the Canterbury and Whitstable Railway Company. The only documents now available are the Company's Letter Book for 1825-30, the enabling Act of Parliament, and the Share Books, none of which are helpful.

3. References to the tunnel in the various books and papers are as follows:

(i) "History of the Canterbury and Whitstable Railway" (Rev. R.B. Fellows)

This would appear to be the definitive history of the railway and the references to Tyler Hill and the tunnel are as follows:

p.18 "If the hill is not levelled then the expedition by the Rail Road is destroyed, and if it be levelled the cost will be enormous and the work attended with the same result of the sides slipping down as is exhibited at Boughton Hill".

P.24 "During October, as stated in the Kentish Chronicle, 'a bore was made to ascertain the nature of the soil in Tyler Hill. In the section this will be found the highest point of the line of road marked out and from the nature of the earth - gravel, clay and sand at a depth of sixty feet below the surface - will be very easy of excavation.' This tunnelling was necessary to the North of Canterbury since the route to Whitstable which was chosen for the railway was the most direct possible. On 31st October, the Chairman, John Brent, in the presence of several Directors and a large concourse of spectators, filled the first barrow with mould to begin the works for the tunnel under Tyler Hill. The height of this hill through which it was necessary to penetrate in order to obtain as near a level as possible, is 213 feet by the section. The whole line of communication between Canterbury and Whitstable was to be finished in 18 months, out of which the tunnel would occupy workmen for a year."

P.26 Quotation from the Kent Herald(14)
"The tunnel of the Canterbury and Whitstable Railway is now going on with considerable rapidity. The work is also going on in the valley of Tyler Hill where considerable progress is already made. The difference of the soil to that on the side of the hill nearest the city is very remarkable, on the latter the whole line has been cut through a bed of white sand; while the former consists entirely of limestone, in which a large portion of iron ore is deposited; it is also frequently found encased in crystal of lime. Owing to the difficulty of smelting, it will not be made available to any other purpose than that of blocking over the arch of the tunnel to prevent the descent of the superincumbent earth. Some inferior crystal has also been discovered about eighteen feet below the surface, and it is expected considerable quantities will be found ere the task is finished."

In the autumn a further account was given of the progress of the work:
"Notwithstanding the many obstacles the projectors of this scheme have encountered, the work is proceeding with the utmost rapidity. The tunnel, which from the commencement has been an object of curiosity, attracts great numbers of persons to view it. Upwards of 400 yards are already finished without any material accident having occurred, although several falls of earth have hindered and injured

(14) Kent Herald, 18th May, 28th September 1826

the bricklayers and workmen. It is computed about 2,400,000 bricks will be required, or about 1,000 per foot. Six feet of brickwork are usually completed in a day, when another set of men excavate sufficient earth during the night for the same quantity of labour to be performed the following day. When the tunnel shall be rendered fit for the reception of waggons, at the same moment the road from Tyler Hill to Whitstable will be laid down and also from St. Stephen's to Canterbury."

The work in the tunnel was checked by a subsidence about this time, as we read in a contemporary record that "a considerable portion of the curiously constructed tunnel on the railway on the Tyler Hill side bodily depressed into the earth a few days since in a very extraordinary manner." (15)

P.27 Meanwhile progress was being made with the tunnel as the following account published on 17th May shows:

"On Sunday morning, last about half-past two o'clock, the workmen employed in the tunnel of the Canterbury and Whitstable Railway effected a communication with the north and south ends by means of cutting an apperture about sixty-five yards in length. The situation of the excavators has been truly distressing for some time, in consequence of the stagnated state of the air, but the great rush of the purer element has entirely cleared the tunnel. It appears, so nice was the calculation of the engineer, that although the line of rail is more than 2,400 feet in length, it has been preserved to within an inch. We understand a thin strata of coal (of about two inches) was discovered within these few days; the perforation of this hill has brought to light many curious geological specimens" (16)

Copies of the relevant issues of the Kent Herald have been seen and add nothing further.

(15) Kent Herald, 4th October 1826

(16) Kent Herald, 17th May, 1827.

P. 63/64 These include several references to complaints concerning the accumulation of fumes in the tunnel but no relevant detail.

P. 73 "THE TUNNEL The Tunnel is on the first inclined plane out of Canterbury; it is 828 yards long, 12 feet wide and 12 feet high; there is no ventilating shaft. It is the earliest railway tunnel through which passenger traffic worked, but not the earliest railway tunnel that was made. The tunnel being low pitched and situated on a steep gradient, became particularly foul when locomotives were introduced on this section in 1836. At the South or Canterbury end there were iron hooks set in blocks of stone and built into the brickwork on which a pair of gates swing across the tunnel's mouth. Owing to the scanty dimensions of the tunnel, special rolling stock had to be used and the locomotives working through it were fitted with specially low funnels."

(ii) "British Railways History 1830 - 1876" (Hamilton Ellis)

"The company increased its capital by £6,000, and appointed Stephenson engineer in place of James, whose feelings were understandably bitter. The line surveyed by him through Tyler Hill was that followed, only one deviation being made, giving a more direct entry to the city of Canterbury. Stephenson and Locke made brief visits, but it was John Dixon, another of Stephenson's assistants, who became the man on the spot and made the line. In spite of some trouble through subsidence, the opposing bores of the Tyler Hill tunnel met in May, 1827. The Kent Herald announcing this, remarked that ' the situation of the excavators has been truly distressing for some time, in consequence of the stagnated state of the air, but the great rush of the purer element has entirely cleared the tunnel. It appears, so nice was the calculation of the engineer, that although the line of rail is more than 2,400 feet in length, it has been preserved to within an inch.' The actual length of the tunnel was 828 yards."

(iii) "History of the Southern Railway" (C.F. Dendy Marshall)

On p. 21 there is the following mention of Tyler Hill Tunnel:

> "No special engineering difficulties were experienced beyond a few falls of earth during tunnelling."

(iv) "The Canterbury and Whitstable Railway" (Ivan Maxted)

P.31 "Tyler Hill Tunnel has always been a source of considerable interest and nuisance to both the general public and the railway companies. In the early days of the railway it was fitted with wooden gates which were locked at night to keep out inquisitive locals. Since The Archbishop's School, Canterbury converted the railway to playing fields and tennis courts the tunnel entrance has been bricked up and partly used as a garden shed, although the hinge posts embedded in stone blocks can be seen a few feet above the ground on the tunnel portal. The interior is now quite derelict and loose bricks and soot fall at regular intervals into the hollows left by the railway sleepers. Pieces of wooden cable channel pinned to the wall, cable brackets and various bolts have been found here. Recently it has been suggested that it might be necessary to fill in the tunnel with soil or concrete piles in order to support the weight of the University of Kent at Canterbury, being constructed on the land directly above."

I have spoken to Mr. Maxted who was unable to recall with certainty the origin of the statement concerning the possible filling-in of the tunnel with soil or concrete piles: he thought that it arose in the course of conversation with the present Headmaster of Archbishop's School. Mr. Maxted was not able to give any further assistance on any other aspect and it was not possible to ask the Headmaster at the time of preparing this report because of his absence through illness.

(v) Extract from an article on the Canterbury and Whitstable Railway by C.R. Heaney published in "Railway Magazine" dated October 1907:

> "Constructed in four sections each of varying shape. The working force evidently started at the Whitstable side of Tyler Hill since as it advanced towards Canterbury each section becomes larger than the preceding. The first three sections are the usual egg shape but the final section at the Canterbury or southern end has perpendicular instead of bow walls and is the largest of the four."

4. The railway was taken over by the South Eastern Railway Company in 1844 and the line was inspected by the Inspector General of Railways in 1846. (No official inspection of the railway would have been carried out at the time it was opened in 1830 because the Regulation of Railways Act establishing the Board of Trade Railway Department was not passed until 1840) A copy of the 1846 Report is available. The section on the tunnel is reproduced below:

" VI <u>Tunnel</u> There is one tunnel commencing at the distance of 52 chains from Canterbury, 836 yards in length, and lined with brickwork throughout. The arch is semicircular having a span of 12 feet 6 inches, with a clear height of 12 feet above the level of the rails. The northern end of the tunnel having been excavated in stiff clay, has an inverted arch below, and the workmanship of this part is much better than the remainder to the southward, which has no invert, and of which the side walls are irregular, as if they had bulged out towards the bottom by the lateral pressure, and there is a transverse crack near the South Entrance of this tunnel, but I have no doubt whatever of its safety. The bricks appear to have been bad, as the surface exposed to air has flaked off in some few parts of the arch, which will be replaced with stronger ones."

5. The South Eastern Railway Company subsequently became part of the Southern Railway Company, which was in turn absorbed by British Rail and the Chief and Divisional Civil Engineer's Department of British

Rail (Southern Region) have been asked for such information as is in their possession. This consists of a file of correspondence and brickwork examiners' reports for the period 1925 - 1962, and a plan showing the profile of the tunnel.

The earliest report following an examination of the tunnel is dated 11th July 1925 and reads as follows:

> "Generally speaking the brickwork in the tunnel is good, though near either end the bricks are somewhat weathered. A few hollow sounding places (all on the Up Side) are not serious at present.
>
> The tunnel has four rings of brickwork. There are ten manholes on the east side and no shafts.
>
> There is a crack at the south end of the tunnel about 2 ft. in from the face which goes right round the crown; it appears to be of old standing and is not dangerous.
>
> The tunnel is dry except for a place near the south end."

(Note: "Up Side" is the right-hand side looking along the tunnel from Whitstable to Canterbury.)

Further reports made annually call for no comment until 4th October 1929, when it was stated:

> "...... bad scaling between the 8th and 9th manholes from Whitstable end. It is about 4½" deep for about 5' and there is a total length of about 25'. This occurs about cornice level on the Up Side. The profile at this spot is irregular and the brickwork is drummy (i.e. hollow-sounding) above the scale."

The next noteworthy comment appears following an examination on 18th April 1934:

> "Brickwork drummy between 5th and 6th manholes generally and at a few other places in the tunnel.
>
> Between the 8th and 9th manholes the inner ring has dropped slightly at haunch and crown but conditions have not altered since the last examination." (5th April 1933)

On 22nd April 1936 it was reported:

> "The side walls on both sides of the tunnel at No. 4 manhole and between Nos. 4 and 5 manholes from Canterbury have bulged and there is a crack in the left-hand return of No. 4 manhole. These have not been noticed before.
>
> There is no apparent change in the condition of the Up Side haunch just north of No. 2 manhole from Canterbury...."

Following this report testing rods were used periodically through the remainder of 1936 and 1937. On 15th January 1938 the brickwork in the Up Side haunch was repaired between manholes 2 and 3.

No movement between manholes 4 and 5 was recorded until 29th April 1942, when it was reported that the brickwork on the left of a crack 8" on the Whitstable side of the 4th manhole from the Canterbury end had come forward 1/16" since 1940; and on 9th April 1943 it was reported that it had come forward 3/32" since 1942, making a total of 11/32". A cement tab was fixed on the crack on 5th May 1944 and on 22nd March 1945 it was noticed that the tab showed a very slight crack in the centre.

No further movement was reported at the regular 9-monthly [illegible] inspections which followed until 14th January 1948 when it was stated:

> "Measurements between pins fixed 3' up from floor of manhole shows the bulge has come in 1/16" since last examination and tab fixed 1' 9" up from floor shows Whitstable side to have come forward very slightly 1/32".
>
> No change in measurement of top pins."

Six-monthly inspections followed until 17th February 1950 when it was reported that:

> "Measurements between the pins at the Canterbury end of the bulge 3' up now show it to have come in 1/32" more....."

No further reference was made to this and the line was closed in 1952 when, in accordance with British Rail practice, the tunnel was examined on foot every 3 years. The following extracts from a note dated 10th May 1965 and a letter dated 16th May 1966 from Mr. G.N. Cope, District Engineer, to Mr. J.H. Scholes, Curator of Historical Relics, British Rail, in connection with a suggestion by Professor W.F. Grimes of University of London (Institute of Archaeology) that the tunnel should be recognised as a National Monument, are relevant:

10th May 1965 "........There are no drawings of the tunnel in my possession ....

.... The site of the tunnel lies between the London Basin and the Wealden Anticline. The Cretaceous deposits, which are the principal beds of the area, generally dip gently towards the north, and are overlain by the London Clay, Woolwich Beds and Thanet Beds. These three strata exhibit large local variations in depth and exposure due to the contours of the country and this is particularly true of the London Clay, which is the uppermost deposit. No records are now available of the construction of the tunnel, but an adjacent embankment which is presumed to consist of material excavated from the tunnel, is largely composed of clay, and there are records that running sand was encountered when repairs were done some years ago.

It therefore seems fairly certain that the tunnel passes through the very bottom of the London Clay, and probably penetrates the Woolwich and Thanet Beds in places. As the thickness of these beds averages 100 - 150 ft. it seems unlikely that the tunnel passes into the Chalk.

The tunnel was last inspected in May 1962........ the sidewalls and haunches were scaled and weathered very badly throughout .... It was dry and sound but the soffit ring of brickwork was perishing rapidly.

16th May 1966 "The tunnel presents no unusual features in comparison with other similar engineering works of that time.........

.... The construction of brick lined tunnels of considerable length through difficult ground had become well established in engineering practice in the later parts of the era of canal construction, and this fact limits its value as an archaeological exhibit."

The file also refers to the granting of permission to The Plessey Company Ltd. of Ilford, Essex, to carry out experiments in connection with investigations into a system for signalling in mines employing the earth as a conducting agent. The permission was granted subject to the structure of the tunnel and the old railway bed not being interfered with and the period during which it was used was from August 1957 until February 1960.

6. In course of conversation with Mr. Burton of the Chief Civil Engineer's Department he stated that because of regular inspections and strengthening, British Rail rarely experienced tunnel collapses and that when they had occurred, in his experience they had been caused by an unsuspected shaft. He said that there was no indication of any shaft in respect of Tyler Hill Tunnel: he thought that for one of its length (approximately 2,484 ft.) the tunnel would have been dug from end to end. (There is no mention of a shaft being constructed in any of the published books and papers relating to the railway).

7. One other possible factor which has been investigated is that a large bomb fell and exploded on or near the site of the South-West Wing of the Cornwallis Building which would in effect have been on or close to the line of the tunnel. Three persons have stated that such an incident occurred: they have been interviewed separately and in the case of one, no contact was made with the others. These accounts are corroborative and establish the facts beyond any reasonable doubt.

A map was published in the Kentish Gazette on 9th December 1944 which purported to show where incidents occurred in the Bridge-Blean Rural District area and the same newspaper also published a similar map in respect of the Canterbury City area. Both maps show that a number of bombs fell in the vicinity of the subsidence site but none actually on it.

The War Diaries and Maps which are held by the County Archivist were accordingly searched and the earliest recorded incident for the area in question was found to tbe 11th October 1940, when high-explosive bombs fell on Hothe Court Farm and Brotherhood Farm, including one down a well at the latter. Unfortunately the map references given in the War Diary which have been checked by the local authroity, Ordnance Survey, the Army and the Map Research and Library Group appear to be meaningless and it is not possible, therefore, to plot where the bombs actually landed. Other incidents in 1940, 1941 and 1942 were given map references which it was ultimately established referred to the old County Map series and these, when plotted, do not register very close to the subsidence site. However, it is possible that not all incidents were recorded, especially those that occurred in rural areas which did little or no damage.

N.L. DURBIDGE
Assistant Registrar

Annexures to this Report:

Four annexures are included in the original. For ease of reading the relevant parts of the first three annexures (A,B and C) have been included within the text of the report. Annexe D, the British Railways 1962 Report, follows:

APPENDIX B

Annexe D

B.R. 12327

BRITISH RAILWAYS......SOUTHERN......REGION..........ASHFORD, KENT .... DISTRICT

TUNNEL EXAMINATION REPORT

SHEET No. ....1....

OF ....3....

NAME......Tyler Hill................ AT .... Ms. .... Chs. TUNNEL No. .. 2007 ....

ON .... LINE BETWEEN Canterbury West ... & ... Whitstable Harbour

TOTAL LENGTH ...... Ms. ..32.. Chs. ..24.. Ft. DIVIDED INTO ..231.. SECTIONS FROM .... END

FIRST SECTION ..11ft..... LONG ...230..... INTER SECTIONS .....11ft.. LONG END SECTION ..... 21ft. LONG

SYMBOLS S = Scaled H Hollow AV Tunnel Shaft J Joints Open W Water Percolation
DD Defective Drains MH Manhole C Crack (State if Tabbed T)

NOTE.—Approximate sizes of hollow patches and scaling to be given in feet. Cracks to be described in detail at end of report. Date of origin of crack to be given or earliest record of same thus (C pre 1937 T)

114'6" in from face sold to Canterbury Corporation. Agreement 2532 1st Nov. 1955.

SECTION No.	DOWN SIDE SIDE WALL	DOWN SIDE HAUNCH	CROWN	UP SIDE HAUNCH	UP SIDE SIDE WALL
	CANTERBURY FACE.				
	Top 4 courses of coping loose & H. 50; missing. Down side corner of head wall C 2'0" in from end. 3'0" above ground level to top. Up side corner fallen.				
1	Foundations under construction for cross wall.				
2	H. 8' x 6'				H. 10' x 2'
4				J.S. 3' x 2'	
5					H. 10' x 2'
6			S. 4' x 3'		
8				Old bulge	H. 20' x 4'
10				Old Bulge	
9 - 11			J.		
11	Cross wall with door provided.				
11-12			J		
21	H. 5' x 4'				
23	H. 3' x 1½' S. 4' x 1½"				
24			J		
26			J	Bulge slight.	
29					H. 24' x 3'
30		Old Bulge			H. 6' x 2'
32					
36-42		S. 2'6" wide		S. H. 12' x 4'	H. 6' x 2' over M.H.
44				H. 20' x 4'	
50				H. 40' x 4½'	
53				Bulge. H. 6' x 2'	
60				S. 10' x 3'	
62					M.H.C & H.
66				S. 10' x 5' (4½" deep)	
68-72	H. 12' x 3'				

COMMENTS:— Side walls & haunches scaled and weathered very badly throughout. Tunnel of irregular shape. Dry & sound but 1st ring of brickwork perishing rapidly.

EXAMINED BY R. Lloyd Owen. (Examiner) ON .. 22.5.62. (Date)

RECOMMENDATIONS:—

SIGNED............ (Inspector, Supervisor or Technical Asst.) ....(Date)

ACTION TO BE TAKEN:—

Nil - Closed Branch Line.

SIGNED............ (District Engineer) ....(Date)

BRITISH RAILWAYS SOUTHERN REGION [illegible] DISTRICT B.R. 123D/[illegible]

SHEET No. 2 OF 3

TUNNEL EXAMINATION REPORT

NAME Tyler Hill AT Ms. Chs. TUNNEL No. 2X7

ON LINE BETWEEN Canterbury West & Whitstable Harbour

TOTAL LENGTH - Ms. 32 Chs. 24 Ft. DIVIDED INTO 231 SECTIONS FROM END

FIRST SECTION 11ft. LONG 230 INTER-SECTIONS 11ft. LONG END SECTION 2ft. LONG

SYMBOLS : S=Scaled H=Hollow AV=Tunnel Shaft —I—Joints Open W=Water Percolation
DD=Defective Drains MH=Manhole C=Crack (State if Tabbed=T)

NOTE.—Approximate sizes of hollow patches and scaling to be given in feet. Cracks are to be described in detail at end of report.
Date of origin of crack to be given or earliest record of same thus (C pre 1937 T)

SECTION No.	DOWN SIDE		CROWN	UP SIDE	
	SIDE WALL	HAUNCH		HAUNCH	SIDE WALLS
85				H. 6' x 3'	
86		H. 10' x 2'			
87					Bulge H. [illegible] C.
90 - 92				Bulge 9' wide.	
93-102		S.		S.	
100			S. 2' x 2' [illegible]		
105-108	S. 3" Deep				S. Deep
108				S. 10' x 6'	S. 6' x 3'
110	H. 4' x 4'				H. 4' x 4'
115-116	H. 12' x 4'				
118-120	30' . 6' S Bulge slight.				
122	S. 15' x 6'				
123-126	H. 30' x 4'				
127		Bulge H. 12' x 2'			
130				H. S. 15' x 6'	
131		H. 6' x 4' S 4" deep			
132	Bulge H. 12' x 3'				[illegible]
133-135				H. 30' x 5' [illegible] 4½" deep	
134		S. H. 4' x 2'			
137	H. 6' x 4'	Old Bulge		H. 4' x 3'	
139			Old Bulge		
140				← Old Bulge →	
144	H. 4' x 4'				
148			S. 5' x 1½"		
150			S. 5' x 2'		
150 - 155	H. 5' wide			[illegible] 3" deep [illegible]	
152		H. 25' x 7'			
152-156		S. H. 1'6" wide.			
155		S. H. 5' x 3'			

COMMENTS:--

EXAMINED BY R. Lloyd Owen (Examiner) ON 22.5.62 (Date)

RECOMMENDATIONS :—

SIGNED [signature] (Inspector, Supervisor or Technical Asst.) (Date)

ACTION TO BE TAKEN :—

SIGNED [signature] (District Engineer) (Date)

BRITISH RAILWAYS SOUTHERN REGION ASHFORD, KENT DISTRICT B.R. 12327/9

TUNNEL EXAMINATION REPORT

SHEET No. 3 OF 3

NAME Tyler Hill AT ... Ms. ... Chs. TUNNEL No. 2007

ON ... LINE BETWEEN Canterbury West & Whitstable Harbour

TOTAL LENGTH — Ms. 32 Chs. 24 Ft. DIVIDED INTO 231 SECTIONS FROM ... END

FIRST SECTION 11ft. LONG 230 INTER-SECTIONS 11ft. LONG END SECTION 2ft. LONG

SYMBOLS S = Scaled H Hollow AV Tunnel Shaft J = Joints Open W = Water Percolation
DD = Defective Drains MH Manhole C Crack (State if Tabbed T)

NOTE.—Approximate sizes of hollow patches and scaling to be given in feet. Cracks to be described in detail at end of report. Date of origin of crack to be given or earliest record of same thus (C pre 1937 T)

SECTION No.	DOWN SIDE SIDE WALL	DOWN SIDE HAUNCH	CROWN	UP SIDE HAUNCH	UP SIDE SIDE WALL
156		S.4'x4' 4½" deep H.14' x 6'			M.H.C.
158	S.4½" deep			S.9' x 5' x 4½" H. 5' long.	S.6'x6'x4½"
159					
162-168		S.4½" deep		S.4½" deep	H. 6'x 2' over M.H.
179					H.6' x 2'
196					H.6' x 4'
198					Bulge
199 - 201					
210				H. 8' x 2'	
213					M.H.C.
210-216	bulge				H.3' x 2'
216					
218		H.4' x 2'			
220-223	S.4½" deep	S		S	S
229	C	C	C	C	C
230	S. 6' x 1'		C	C	C

WHITSTABLE FACE.
Coping of wing walls C.
Two small cracks on parapet to 2nd ring of 4 ring arch.
Up and Down side wing walls H.
End of Down wing wall missing.

CRACKS.
66 Slight crack in Whitstable wall of M.H. 2'0" below roof to roof.
88 Vertical crack 8" Whitstable side of No.4 M.H. open 1/8".
Whitstable side pushed forward 7/16".
Slight crack Whitstable wall of M.H. No change.
132 Slight crack in M.H. Both sides roof to floor open 1/8".
156 Slight crack in each side wall of M.H. No Change.
Crack in back wall from roof 19 courses down.
230 Crack from G.L. on Upside to centre of crown.
Slight crack in down wall 10ft. above ground to 13ft. above ground diagonal.

COMMENTS:—

EXAMINED BY R. LLOYD OWEN (Examiner) ON 22.5.62 (Date)

RECOMMENDATIONS:—

SIGNED .... (Inspector, Supervisor or Technical Asst.) .... (Date)

ACTION TO BE TAKEN:—

SIGNED .... (District Engineer) .... (Date)

REPORTS

ON

UNIVERSITY OF KENT

TUNNEL COLLAPSE.

SIR HAROLD HARDING
BSc FCGI DIC FICE PPICE

Report dated 17th April 1975

As requested I send you my comments upon the matter of the collapse of the disused railway tunnel under the Cornwallis Building of the University of Kent. Following correspondence with the University Surveyor and Deputy Registrar and discussions with Professor Bishop and Mr. Prichard, I have studied the documents which you sent to me. I visited the site on March 17th and had further discussions with Mr. Edwards, the Surveyor, and Mr. Prichard.

The documents included the Historical Survey, The Report of Messrs. Farmer and Dark of November 1973, Site plans of the area and the building, photographs inside the tunnel and of the building and a table of dates and events.

The Tunnel

The tunnel was built in 1825-1830 and was the first railway tunnel of its kind. It is 771 metres long, designed for single line working, lined with brickwork, eighteen inches thick in four courses. It has a semi-circular arch, 12 feet wide at axis. The shape from the North portal to 465 .5 metres is shown in Farmer and Dark's Report as almost eliptical in section with a curved brick invert. From the photographs the side walls seem nearly vertical. This section was tunnelled in London Clay and the failure has been between 240 and 270 metres.

The length from 465.5 metres to the South Portal which was still available for inspection has vertical walls on footings.

Method of Construction

The Kent Herald, 28th September 1826 recorded that the miners completed a six feet length of excavation in a night shift and that the bricklayers built the lining on the following day shift. The tunnelling in six feet lengths would almost certainly have been carried out by the English Method or "length work." This technique had become routine in the Canal Age (1750-1800) and was carried on in the railway age, using brick lining in most cases.

The length chosen for the driving would depend upon the ground and the size of the tunnel. Usually a heading would be driven ahead and then timber bars would be set with the rear end resting on the last length of brickwork and the leading end supported on vertical props. The bars would support the head trees of the heading. Miners would work on either side widening out with short poling boards and erect further bars, propped at the leading end until an arch had been formed. In loose ground this requires skilful work as the entire surface has to be closely timbered.

The lower part could then be dug out and timbered under the protection of the arch. In London Clay a more skeleton form of timbering would be used but with bars in the same way. The whole scheme allows for each setting of poling boards or "piles" to be supported as the brickwork rises and the bars withdrawn. If the ground is in need of support the boards have to be built in, which can lead to trouble.

The bricklayers would then build up the lining and when axis was reached timber drums would be erected on which loose horizontal "laggings" would be placed on which the bricklayers would build their courses. The crown would be keyed in by using short block laggings and the bricklayer would work backwards into the heading to key up the final courses.

A six feet length would yield about 60 cubic yards of clay excavation which would be in the capacity of the miners if the timbering was only partial, but the 17 cubic yards of brickwork would need very rapid work. From personal experience of this method on London County Council sewers 24 hours would have been needed. The brickwork as inspected would have horrified an L.C.C. Inspector with very high standards of perfection. We can deduce that the brickwork in this tunnel was sufficient for its purpose as it stood for 150 years but was more hasty than perfectionist.

One source of weakness sometimes found in bricklined railway tunnels is due to the bricklayers scamping their work by not filling the spaces left by the tunnel bars as they are withdrawn but there does not seem much sign of this in the present case.

The section from 465.5 metres to the south portal was tunnelled in the fine sand which often occurs when the London Clay lies on the Woolwich & Reading Beds. When this sand is dry it can stand almost vertically during excavation as it is compact and it appeared dry in the invert. In a wet condition it can become a very difficult material.

In this part of the tunnel the walls are set a few inches inside the springing of the arch in several lengths. This gives a strong impression that the walls were built as underpinning to the arch which was built first. This method was sometimes used to give access ahead and to reduce the height of the tunnel face which needed support. In a letter dated 1836 from Sir Marc Brunel to his son Isambard, when the latter was starting on his Great Western Railway tunnels, he advises this method in certain cases. It is uncertain whether this method was adopted under the Cornwallis Building and so become a source of weakness, but it does not appear so in the photographs.

Comments on the Collapse

The tunnel had been inspected and maintained by the Railway under chaning titles and after traffic had ceased in 1952. Several reports are in the Historical Survey. A photostat of a report by G. Maunsell & Partners dated August 1963 is included in Farmer and Dark's Report as Appendix III. In it the condition of the tunnel is discussed at the time of the anticipated purchase by the University and an inspection every three years was recommended.

In October 1973 Farmer and Dark were asked to inspect the tunnel which led to their report of November 1973 which included 37 sheets of details of inspection for the whole length. They concluded that some sections of the tunnel required very early attention. "The section which has deteriorated most seriously lies under Cornwallis and this makes imperative that some action is taken, as the tunnel must not be allowed to collapse. it would be highly advisable also to treat the section which will be under the proposed extension of Rutherford College."

Farmer and Dark rightly concluded that rebuilding the brickwork would not strengthen the tunnel and that cross walls would bring their own problems. Their suggestion of lining with 2 or 3 inches thickness of gunite has often been successfully adopted, but the collapse took place while tenders were being obtained.

In February 1974 signs of distress were observed in the Cornwallis Building and in early May falls of brickwork were found at Chainages 240 and 260. The possible mechanism of the collapse can be supported by the photographs. Cracking and brick falls on the west wall in early May led to a fall of the facing bricks by June 10th from "seven o'clock to ten o'clock". The brick facing seems to have peeled away and as no vertical mortar is visible it seems that the faces of the inner bricks have peeled. The brickwork is in English Bond, alternate courses of headers and stretchers, which is unusual as courses of stretchers are sometimes specified.

Considerable cracking can be seen through the whole thickness of the lining though the joints seem better filled with mortar than might have been expected. By July 3rd the brickwork had collapsed and clay had fallen into the tunnel leaving a small cavity beside the wall.

By July 9th the arch had collapsed down to the old rail level. The large lumps of clay which almost filled the tunnel section show typical "backs" or smooth faces. The cavity can be seen at the crown. The same effect at 269 metres is shown in the photograph dated July 21st. so possibly the movement was from 240m to 270m. From the subsequent tests it has been proved that, after the initial cavity had been formed the London Clay descended into the cavity in the form of a plug with almost vertical sides and did not form a crater with sloping sides.

Comparison with other types of failure

When tunnels are being driven in London Clay, if the tunnel face and crown are insufficiently supported, large blocks of clay can fall out of the face or roof leaving cavities overhead. But in such cases the cavity can be quickly filled as the miners are present with all necessary resources. So the behaviour of clay in reacting to such cavities is not often on the scale at Cornwallis and not often observed.

As an example in my experience, a bomb dropped in Trafalgar Square shattered the heavy cast iron lining of the 26 ft. diameter chamber at the foot of the escalator but did not penetrate. The lining dropped on shelterers followed by massive lumps of London Clay leaving an arch shaped cavity above. But miners were available next morning who were able to erect supports and fill the cavity before any further effect followed.

In the present case no organisation was at hand. The only access was through school grounds so if an experienced tunnel contractor had been available to carry out rapid work it would have taken an appreciable amount of time to organise access and to transport plant and material and then carry out support or filling, so once the brickwork had decided to fail there would have been little chance to prevent it.

A number of brick lined railway tunnels, including several built by Thomas Brassey in France showed signs of distress after 100 years. But these were objects of continuous maintenance and brick linings have the virtue of being easily repairable when taken in time. The tunnels which have become best known for their troubles are those where an invert of brick or concrete had been omitted, in clay strata. But there seems to have been a reasonable invert in the collapsed section, so invert heave does not seem to have taken place.

The case of the railway tunnel at Bo-Peep near Hastings is described in the Institution of Civil Engineers Journal, March 1951. The invert started to rise and the side walls settled 7 inches. The movements were over several years and cracks appeared in houses above the tunnel. The final observations showed a "draw" at the surface for 30 and 35 feet on either side from the centre line of the tunnel, which had an internal diameter of 22 feet with walls 2 ft 10 inches thick. Although there was considerable cracking the arch was supported by heavy timbering prior to lining with cast iron.

In the discussion on the paper I described an experience at Brancepeth Colliery where glacial silty clay had flowed into a mine gallery. The result was a cylindrical cavity reaching 120 feet to the surface. A student at Cambridge University described his research by models of the effect of unlined tunnels in soft clay in a competition. Even in such a small scale he observed a settlement of almost vertical sides.

The seven feet diameter Tower Subway was also crushed by a bomb which fell in the River Thames but without penetrating to the tunnel. In that case the clay seemed to have gradually squeezed the fractured lining without forming a cavity.

When a bomb shattered the heavy lining of the Balham Tube Station down to platform level the London Clay above dropped as a solid block. As the top of this was below the crown of the tunnel the water from a burst main and the ground water washed in sands and gravel which buried the shelterers. As miners worked to clear the tunnel the clay was so intact that it looked as if the miners were excavating a normal clay face.

The failure of the invert in the Arley Tunnel is described in the same Journal as Bo-Peep tunnel but is in different soil. But another case of movement occurred during the construction of a railway tunnel by length work at Cuffley in the 1900's. This was in the yellow London Clay and the squeezing pressure of the clay closed up the bar heading and extra supports had to be added in the lengths. At Shepherds Bush in an L.C.C. sewer which had a length left under water for many weeks, the bar props were forced three inches into the timber sill before collapse.

A number of failures have occurred in brick lined sewers which are not so easy to maintain, though this is constantly done in the London area. More troubles have arisen from empty obsolete sewers. But most of the reported ones have been when running ground overlay the tunnels and, after dislodgement of bricks, cavities developed. Some are described by Sir Francis Fox in his Autobiography. A spectacular case occurred with a three feet diameter brick sewer at Farnworth (I.C.E. February 1963).

The failure under the Cornwallis Building has similarities to the above case and some differences. The tunnel was built with an invert which does not appear to have been disturbed. Most of the tunnels quoted showed signs of trouble but on a much longer time scale and without such instant collapse. There does not seem to be any factor which triggered off the collapse other than fatigue after 150 years.

The weak point of the lining seems to be the vertical nature of the side walls under a semicircular arch. In Brunel's letter mentioned above he criticises just such a section and advises an oval shape with a low axis and the part leading into the invert at a curve and not as flat as in this case.

The time scale of the failure is more rapid than the other cases of trouble to existing tunnels of long life. Other tunnels in rock such as the Severn Tunnel where the lining was strengthened by grouting did not reach collapse. The Woodhead tunnel suffered from deterioration of the mortar in a masonry lining after 100 years of life but in that case a fresh tunnel was driven.

The inspection of brick lined tunnels can take several forms. In a visual inspection only the state of the face of the lining, the state of the pointing (which can conceal much) and possible cracks or deformations can be observed. If a rigorous inspection is required, including cutting out lining, then a considerable expedition has to be mounted including the erection of supports.

The remaining length of the tunnel under Rutherford and beyond appears stable but is vulnerable. The ground is sand and this can flow into any break if sufficient water is present, with quite surprising results. Some of the old-time pioneers of London Tunnels used to preach that abandoned holes should always be filled up. It will be expensive to fill the remainder of this tunnel, including the cost of imported filling but it should be carried out.

disintegrated to such an extent that I saw why my "Masters" had always taught the need to board and grout up any exposed face immediately.

There is also the question as to why the collapse should have occurred under the Cornwallis building and not under the open lengths. The depth of ground is considerable and loads are supposed to spread themselves. It is vaguely possible that when the tunnel was being driven with more speed than precision there was a loss of ground (and this is mentioned in the History) with disturbance but that this had been arrested by the building of the brick lining. If slight failure planes along clay "backs" had been activated then the weight of the new building may have transferred itself to these planes and acted as the last straw in a finely balanced condition.

The effect of a near by bomb can be disregarded. In 1943 I served on an Institution Committee to advise the War Damage Commission on War Damage due to Earth Movement. This was so complex that we listed all the forms of earth movement which could arise from other causes as a guide for eliminating false claims. Many of these claims were in cases where houses had settled due to shallow foundations in clay.

I spent most of 1941 in repairing bomb damage to L.C.C. sewers in the East End of London and the Docks as it occurred. These were in brick-lined sewers up to ten feet in diameter. In every case the bombs had penetrated to the sewers so no lessons could be drawn on the parallel of this collapse. But I can not recall any case where a sewer had to be repaired due to damage from adjacent "incidents".

The filling of the remaining tunnel is necessary especially if there are water mains above. It would be bad practice to allow it to collapse and I agree that bold and simple methods are the best on the lines of your proposals in your letter of April 21st.

I enclose a copy of the letter of Sir Marc Brunel to his son Isambard which I mentioned. It has a certain interest in this situation. The original is in the I.C.E. Library by courtesy of Sir Marc Noble.

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Report dated 3rd June 1975

The following comments are in addition to my Report of April 17th. I have studied the expert opinions on the bricks removed from the tunnel which you sent to me with your letter of May 29th. These deal with the possible effects of water, of brickearth as the brick material and scaling.

Water. As Mr. J. Harding of the Brick Development Association points out moisture and ground water are important agents in such problems but I do not agree with his comments on water in the section of the tunnel in sand.

The properties and behaviour of sands depend upon their particle size and not their geological names. The Oldhaven sands under the London Clay in which the southern part of the tunnel was driven are similar in particle size to the Thanet Sands below and the Bracklesham Sands which came later than the London Clay. These sands when perfectly dry can often stand almost vertically, as could be seen in a quarry of Thanet Sand between Greenwich and Woolwich. On another occasion I saw a builder digging a square excavation at the Royal Aircraft Establishment at Farnborough in the Bracklesham sands. The sand stood vertically for the first ten feet and the builder had rashly used no timbering support. He then had to dig two feet deeper below the ground water table where the sand became almost fluid and caused the collapse of the dry portion.

I mention this to support my theory that the Oldhaven Sands were perfectly dry when the tunnel was being driven and seem to remain so to this day, presumably due to the lie of the land and the strata preventing the entry of ground water and the result would have been slight artesian conditions if this had happened. If the sand had been waterbearing the brickwork could not have taken its present shape; an invert and sub-drain would have been essential.

Moisture is a different matter and the presence of water in London Clay calls for comment. When cast iron lined tunnels were driven deep in the London Clay for Tube railways the joints were pointed with cement. Those who have worked in running tunnels at night after traffic hours can testify to the extreme dryness of the conditions. But during and after the War a number of tunnels deep in the London Clay were driven in which delicate electrical equipment was installed. Strict instructions that no drop of water must fall on the equipment were implemented. After the concrete floors had been laid and before the equipment was installed it was a surprise to many to see the number of drops which showed up on the concrete and which had come from the clay, and had penetrated the pointing. Special measures had to be taken to guide the water down to floor level. This is also the practice in escalator tunnels to avoid discolouration of the plastering.

This appears to have some relationship to the theory of Messrs. Harris & Sutherland that the old tunnel dried out during the passage of trains and the through ventilation, and that when the tunnel was disused and sealed off the moisture from the clay became a factor possibly with condensation.

Brickearth. Brickearths are superficial deposits and alluvial drifts which can be formed in many ways. Surface deposits were easy to dig in the days of hand tools and were often burnt on the spot as the material often contained combustible material which helped the process so it is likely that the tunnel bricks were burnt from brickearth.

The Memoir on the Wealden District, published by the (then) Geological Survey has, on page 62, - "in extreme cases-such as the brickearth on the tracts of London Clay and Thanet Beds in North Kent - the drift is hardly distinguishable from the solid". The maps also show considerable areas of drift in the valley of the River Stour.

Brickmaking was widely scattered about the country until the coming of the railways and problems of transport and availability took precedence over suitability. There is a suggestion that some of the bricks came from Essex. This is plausible as the River Stour is said to have been navigable to Canterbury since the days of Queen Elizabeth I.

Brickwork and Scaling. One of the virtues of brickwork is the ease with which defective areas can be cut out and re-bricked leaving little trace of the effort. In the 150 years of the life of the tunnel it is possible that bricks were replaced in many areas. This may account for the use of headers and Flemish bond as most linings were carried out in courses of stretchers.

The opinions of the experts appear to differ. British Rail are naturally alert to the swelling of clays behind brick linings and they report frequent scaling of brickwork in their tunnels. They attribute this to attack by sulphuric acid formed by water combining with sulphur trioxide from products of combustion from steam locomotives. They also state that this occurs many years after steam locomotives have ceased to run in some cases. This is a helpful opinion.

B.C.R.A. considered that such an acid would be more likely to attack the lime of the mortar leading to the formation of sulphates. The swelling of the mortar could lead to pressure on the bricks leading to their disintegration. The opinion also was that the bricks were relatively inert and unlikely to be affected by the acids.

Brick Development Association agreed with B.C.R.A. that brickearth was the basic material of the bricks, but the rest of the opinion is complex. It contradicts B.C.R.A. by suggesting that the bricks could swell with moisture though the end of his paragraph (11) is not clear. He agrees that sulphate attack could have caused the spalling and it would be expected to affect the mortar but that the condition of the perpend mortar does not confirm this. He does not comment upon the mortar in the horizontal courses. As above, I do not agree with his theory on water in the sand.

In certain areas London clay is known to produce appreciable quantities of soluble sulphate salts and in recent years these have attacked concrete linings. The damage is caused by solution of the salts in water. In this tunnel water was in short supply but there has been a long time for the process to work. So it is possible for all the processes suggested by the experts to have acted in varying degrees, leading in due course to the failure.

At the time of the failure the invert was obscured by the track ballast so observation was not possible and the exact mechanism can only be deduced and not proved. In tunnels which have no invert the degradation usually develops when the ground under the track heaves and so disturbs the toe of the wall which leads up to the arch. In this case it seems that the wall parted at the springing in the first case. If this is so you rightly point out that there would then seem no reason for abrupt shearing of the invert. As only photographs are available it is hard to draw conclusions as to the mechanism.

I still cannot divorce my mind from the fact that the tunnel "decided to collapse" under the building not elsewhere in its length. This gave rise to my theory of the 'last straw'. There were many agents at work gradually weakening this aging tunnel as recorded above. I note from your calculations that the dead weight of the building has only a minimal effect on the factor of safety for varying shear strengths of the plug size.

It is recorded that ground was lost during the tunnelling. If clay had then dropped as a plug and then to have come to equilibrium when supported by the brick lining it is possible that additional "backs" were left in the stiff fissured clay which normally has a number. This could lead to access of more moisture. There may have been much disturbance and agitation from mechanical plant during construction so that could combine with the admittedly small effect of the weight of the building and originally disturbed ground to add one more small factor to the others which came to act.

It is customary when driving brick-lined tunnels for the drums on which the arch is turned to have its folding wedges which hold it in position to be slightly loosened very soon after the arch is turned so that the arch goes into operation. This may account for thinner courses in the arch which have been mentioned. If the work was carried out as described with miners coming in on the shift following the bricklayers there would have been very little time before the arch drum had to be struck and removed to allow digging to proceed. This does not improve the strength but is only one more small matter.

Structures which fail during, or shortly after construction generally prove to have been endangered by one or more major causes. Structures which fail from old age after many years are usually subjected to a number of small cumulating ailments which combine to reduce its resistance. It is surprising that this tunnel lasted so long and it is a pity that the time of decease cannot be forecast with precision. In this case hope had been abandoned but the end came before help could arrive. The reasons in my opinion are a combination of the circumstances described.

REPORT BY MESSRS. SANDBERG

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Report. A.824/CHEM.

THE UNIVERSITY OF KENT AT CANTERBURY
DISUSED RAILWAY TUNNEL

Your letter reference 1204/GFH/VCB dated 20th June 1974.

INTRODUCTION.

Four samples of brickwork were received for the determination of sulphate content. It is required to be ascertained whether the brickwork would be likely to affect a coating of Gunite applied on it using a 1:3 Mix of Ordinary Portland cement and sand.

The samples were designated as follows:-

Our ref. A:- Brick from second layer at 240 m.
" " B:- Outer ring at 262 m.
" " C:- Outer ring at 246 m.
" " D:- Outer ring at 180 m.

RESULTS.

The samples consisted of brick with only small portions of mortar attached to them. General samples were prepared, dried at 110°C and analysed for acid soluble sulphate contents with the following results:-

Sample	Sulphate (as SO_3) % by weight
A	0.37
B	0.18
C	0.92
D	0.13

/Cont'd..........

REMARKS.

The bricks have appreciable, though perhaps not excessive, sulphate contents. If the tunnel surface is dry, it would appear probable that the Gunite process could be applied using Ordinary Portland cement without undue danger of sulphate attack. Under wet conditions, there would of course be more danger of sulphate attack. The proprietors of the Gunite process could probably best advise in that case.

It will be appreciated that it is extremely important that the surface of the brickwork be thoroughly cleaned, e.g. by wire brushing, to produce a sound base for the Gunite coating.

For MESSRS. SANDBERG.

Messrs. Farmer & Dark,
131, Upper Richmond Road,
London, SW15 2TR.

RBA/RAN.

15th July, 1974.

WORKS ON SITE

E.1 Contractors

Main Contractor:

Cementation Construction Ltd.

Sub Contractors:

BKI Ltd.	electrical cabling
Cementation Ground Engineering Ltd.	tunnel filling (Ch. 293-490) compaction grouting site investigation
Denne (Builders) Ltd.	general building work
Electron Engineering Co. Ltd.	internal electrical services
E G & A Building Co.	fire-escape
Haden Young Ltd.	heating services
H. Smith (Orpington) Ltd.	demolition
Intrusion Prepakt (UK) Ltd.	tunnel filling (Ch. 0-293)
Kent Tarmacadam (Contracts) Ltd.	external paving
Pynford (Southern) Ltd.	underpinning

E.2 Objective

The immediate objectives following the subsidence were:

1) to divert the services which had been broken as a result of the subsidence;
2) to fill the tunnel under Block C, Rutherford and the central service mains as a safeguard against subsidence;
3) to demolish buildings or parts of buildings where these were a danger to other buildings or were beyond repair; and
4) following demolition to erect temporary ends, fire-escapes etc., so that buildings evacuated at the time of the subsidence could be reoccupied.

The intention that all this work should be effected by the beginning of the Michaelmas term was achieved under a contract let to Cementation Construction Ltd. Following these works, sub contracts for site investigation and ground stabilization of the subsided area were carried out, with the final tidying up operations (removal of underpinning etc.) being completed by March 1975. The total value of the works described above amounted to some £370,000.

E.3 Tunnel Filling

Although it was not formally established at the time that the subsidence was a direct result of the tunnel collapse, there was no doubt that the sections of the tunnel under Block C, Rutherford College and the central services had to be filled.

A contract for filling under Block C was let by Farmer and Dark to Intrusion Prepakt. However, it was decided that additional resources had to be mobilized as a matter of extreme urgency, and a second contract was let to Cementation Ground Engineering for filling from Chainage 293 - 495, south of Rutherford College.

For speed of operation and because of the obvious dangers of working in the tunnel, the grouting operations were carried out from the ground above. A 10:1 pfa/cement grout was injected via PVC tubes placed in 150mm diameter bored boles to the crown of the tunnel. Inspection of the filling operations was carried out in the tunnel. Where subsequent proving holes were bored, the tunnel was found to be filled completely to the underside of the crown. The only difficult zone to fill was the collapsed section under the subsidence itself since, at the time, the ruins of Block B prevented vertical boring. However, this was remedied during the compaction grouting contract (see section E7).

E.4 Underpinning

In order to assist in preventing further damage to the remaining portion of Block B, Pynfords, underpinning specialists, were instructed to place collars, needles and jacks so that if further subsidence took place the structure could be held in position by raising the jacks. If necessary the columns would have been severed from the foundations by thermic lances.

This underpinning was maintained throughout all the works on site and was only removed after the satisfactory completion of the compaction grouting contract.

E.5 Demolition

A detailed inspection of all the buildings in the vicinity of the subsidence was made and a photographic record was taken. The extent of the damage to the Cornwallis Block C and the link bridge Block D is described in detail in section 2. However, of immediate concern was the perilous state of the buildings affected by the subsidence.

The link bridge was in imminent danger of collapse since its column supports had failed and an early decision was made to demolish this structure before it collapsed in an uncontrolled manner, with the consequent risk of causing further damage to surrounding buildings. Block B had broken its back at a point approximately two thirds along its length and the portion of the building on the subsidence side of the fracture had suffered so much severe damage and distortion that there was no prospect of saving it. Furthermore, following the initial subsidence of about 700mm, it was apparent that the SW corner of this building was continuing to settle (level checks on 29th July 1974 and 13th August 1974 revealed a downward movement of about 50mm). The effect of this would have been to increase the lateral pulling forces on the relatively undamaged eastern part of Block B, hence endangering the vital plant room over the entrance to Blocks A and B (subsequent investigation revealed local concrete failures to the floor junction of the plant room). It was therefore decided that there was no alternative but to demolish the severely damaged portion of Block B. Because of the precarious state of this section and the risk to life however, it was not possible to demolish in a piecemeal fashion and preserve the precast panels. The demolition of this section of Block B was preceded by the demolition of the first floor glazed link to Block C.

E.6 Building and Service Repairs

Considerable remedial works were necessary both inside and outside the remaining buildings of the Cornwallis Complex and the Gulbenkian Theatre. Externally, these works included the construction of cladding to the ends of buildings left open by the demolition; the support, diversion and repair of numerous electrical, heating and water mains and drainage services; the construction of a fire escape to facilitate the reopening of the Cornwallis Block C and other considerable repairs. Internally a large amount of work was carried out at repairing cracks and damage to the Gulbenkian Theatre and the remaining third of Block B.

E.7 Remedial Grouting of Subsided Zone

The site investigation showed that the clay in the zone of subsidence was largely intact and also that except for the section of tunnel within Chainages 240 to 270 the filling procedure had been effective. As explained in section 5.1, despite the fact that the boreholes into the collapsed section of the tunnel revealed cement/pfa grout, it was considered that substantial voiding might still be present in the subsided material. The remedial grouting scheme was therefore formulated with this possibility in mind.

It was also clear that as a result of the large scale movements of the clay beneath the Cornwallis Building this volume of soil must have suffered a reduction of the pre-existing horizontal stresses in the ground, which under normal circumstances would have been expected to exist.

If this condition were left untreated then the clay surrounding the "stress reduced" area would be subject to long term horizontal and vertical strains associated with the redistribution of the horizontal state of stress in the ground. Further, these strain movements of the ground would be a threat to the structural integrity of the remaining buildings which surround the subsidence area.

Finally, trial pits and excavations near to Block C showed that there had been some ground movement beneath its foundations although the structure itself had not so far suffered significantly.

A remedial grouting scheme was recommended and accepted. This scheme fell into three basic operations:-

i) grouting to fill any remaining voids within tunnel Chainages 240 to 270;

ii) compaction grouting to close any open fissures and to restore the horizontal state of stress in the clay to equal the overburden pressure; and

iii) grout injection beneath the southern end of Block C to fill any possible voids left by the ground falling away due to the subsidence.

In order to ensure that any remaining voids within tunnel Chainages 240 to 270 were filled, grout was injected under hydrostatic pressure through boreholes at 2.5m centres. The sequence of grouting was to first grout at 5m centres until the ground refused to accept further injection and then to grout through secondary holes in between the primaries again until refusal. More than $100m^3$ of clay/cement and pfa/cement grout was injected into the region of the collapsed tunnel.

The compaction grouting was performed through tubes à manchette placed in the primary holes of the first phase of the remedial grouting programme. This process was adopted with the intention of compacting the clay in the subsided area and thereby closing any open voids or fissures in the clay. It is illustrated by Fig. E1. Once any open voids had been closed the compacting process would have the effect of increasing the horizontal stresses in the clay and consequently restoring the horizontal stress in the subsided plug to the vertical overburden pressure. Clearly this treatment could not completely restore the original state of stress in the ground since this was produced by geological over-consolidation and the horizontal stresses in the ground would have been greater than the vertical. With compaction grouting there is no further benefit to be achieved once the horizontal stress has equalled the vertical, since further injection will then only lift the ground above the injection point.

However, this effect was used to indicate when the maximum induced horizontal stress had been achieved and the compaction grouting was curtailed when a single $0.1m^3$ injection of grout at any point in the soil mass caused surface heave of approximately 1mm. Figure E2 shows the cumulative heave measured at the surface after the compaction grouting had been completed. Precise levelling techniques were used to monitor the whole grouting operation in order to avoid damage to the existing buildings which surround the subsidence area. To achieve this precise monitoring 36 short 2.0m long piles were installed on a grid over the whole site and measurements were taken at frequent intervals onto steel reinforcing bars set into the piles.

The grouting under Block C was achieved by injecting a permeating cement grout under the foundations through a grouting probe or lance. This operation was successfully completed without incident and with minimal grout take, thus establishing that the south end of Block C was secure.

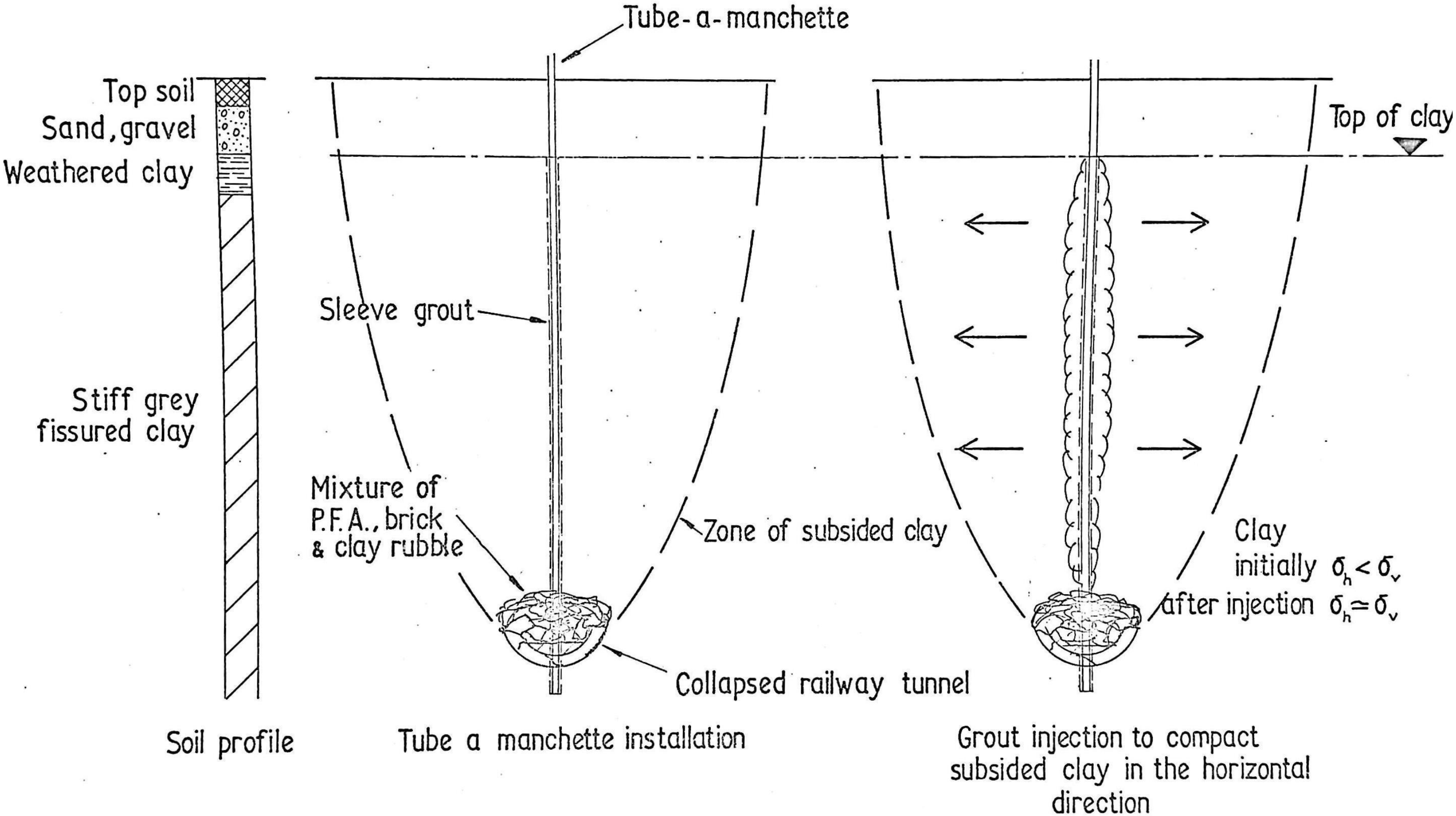

FIG: E1. Compaction grouting.

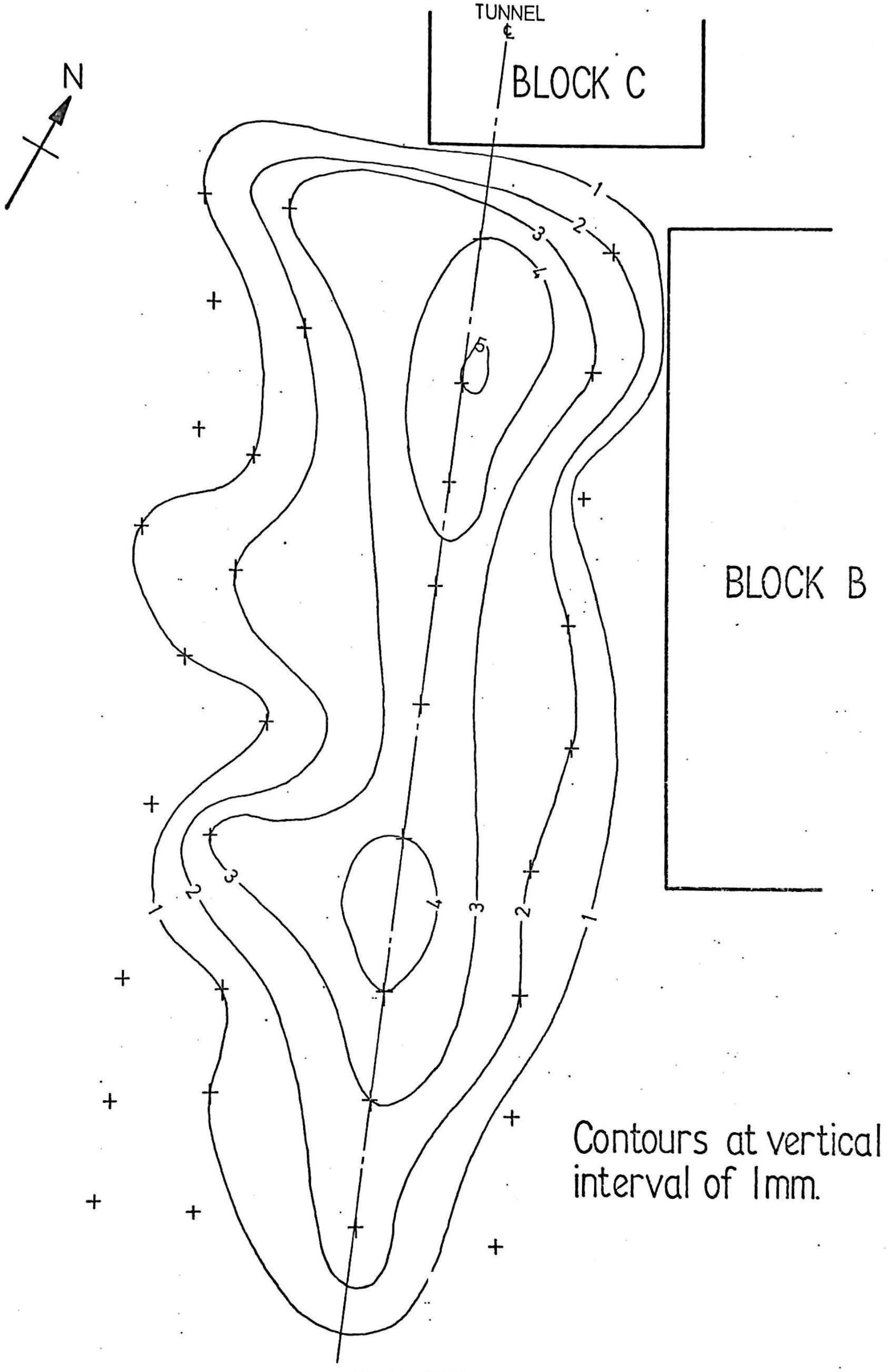

FIG: E2.
Sketch showing ground heave due to grouting.

GEOTECHNICAL NOMENCLATURE

F.1 Shell and Auger Borehole

A borehole, normally 6 inches (150mm) or 8 inches (200mm) in diameter, formed by means of a winch operated gravity-drop rig. In clays the cutting tool takes the form of a steel cylinder with a sharpened cutting rim at the lower end. This cylinder is raised by means of a winch and then dropped to cut a hole in the ground. When the cylinder is next raised the cut material is removed at the surface and can be examined. At any stage a percussive sampling attachment, driven by the winch, can be lowered down the hole and "undisturbed" U100 samples taken.

F.2 Rotary Augered Borehole

A borehole formed by means of a rotary rig. The rotational drive is normally vehicle mounted and connected to the auger by means of drilling rods. The auger cuttings can be transmitted to the surface either by means of a continuous auger bit or by means of an air or water flushing system. Alternatively unbroken cores can be taken by means of a special core cutting barrel.

F.3 U100 Samples

The standard cylindrical 4 inch (100mm) diameter by 18 inch (450mm) long sample taken by driving a sampling tube into the ground at the bottom of a borehole. The bottom rim of the sampling tube has a cutting shoe attached and when the sample has been taken the sampler is brought to the surface and the cutting shoe is removed. The sample is then wax sealed to prevent moisture loss - and end caps are screwed on. The sample can be stored in this state for a considerable period of time.

When the time comes to remove the sample for examination and testing it is forced out of the tube by means of an hydraulic ram.

F.4 "Continuous" Sampling

This is a technique whereby a continous sample of the soil is taken by means of U100 samples. Two sampling tubes are used in combination and thereby 900mm of soil is removed in one sampling operation. The hole is then cleaned out by normal shell and auger methods. A new sample is then taken and thus U100 samples are taken down the full depth of the borehole.

However, it is not possible to obtain a complete set of samples in a borehole for two reasons.

First, the portion of soil in the cutting shoe cannot be kept in a sampling tube as is the case with that portion forced into the tube. The cutting shoe sample can, however, be logged and preserved separately in a sealed bag although it will then not qualify as an "undisturbed" sample for testing purposes.

Second, a short length of the hole will be removed during the process of cleaning out between samples. This inevitably means that some small proportion of the "continuous" sample will be lost, but a recovery of 95% can be achieved.

F.5 Fissures and Cavities

a) Fissures: The fine joint system which may be present in a clay. These joints can either be open, closed or else filled with silt or some other material. The bulk strength of clays is dependant on the size and pattern of the fissures.

b) Cavities:: Any open unfilled hole in a clay mass. A cavity could either be man made or a natural phenomenon, although in London clay the latter is unlikely except in the case where large movements have occurred. In the case in question, cavities due to unfilled over-break during the construction of the tunnel or else caused by the subsidence movements were of prime concern.

F.6 Penetrometer

A device used to measure the shear strength of a clay. The device used was a "pocket penetrometer" in which a plunger is pushed into a clay sample and the force required to achieve a standard penetration is measured and, by means of calibration, converted to an equivalent shear strength.

F.7 Piezometer

A device for measuring the porewater pressure in a soil. The instruments used consist of a porous element placed at the bottom of a borehole with a standpipe attached. The water level in the standpipe is measured at intervals by means of an electronic dipper which detects the free water surface.

F.8 Moisture Content

The ratio by weight of water to the dry soil. It is usually expressed as a percentage.

F.9 Atterberg Limits

The behaviour of a clay will vary according to the its moisture content. Thus a dry clay is brittle while a moderately wet clay will tend to behave like plasticene and is said to be plastic. A very wet clay will flow in a liquid fashion. The moisture contents at which the behaviour of a particular clay is said to change from brittle to plastic and plastic to liquid are known as the Atterberg limits. The former is referred to as the plastic limit and the latter the liquid limit. The methods of measuring these transition points (which are by no means abrupt) are defined in B.S. 1377 'Methods of Testing Soils.' These limits are used for soil classification.

F.10 Triaxial Testing

A method of testing a cylindrical sample of soil to failure under known triaxial pressures or stresses. The apparatus consists of a cell in which a cylindrical sample, which is enclosed in a membrane, is placed between two plattens. The cell is flooded and hydrostatic pressure is applied to the sample. The top platten is then used to load the sample at a known strain rate and the force required is measured. The sample eventually fails under measurable triaxial stresses and from the results of several tests a failure criterion can be established. From this the field behaviour of the material may be predicted.

PORE-WATER PRESSURE MEASUREMENT

G.1 Pore-water

All soils consist of a skeleton of solid particles surrounding open voids.. These voids or pores are normally filled with air or water or a mixture of the two. If no water is present then the soil is said to by "dry". Conversely if the pores are completely filled with water then the soil is "saturated". If, however, there is a mixture of air and water then the soil is termed "partially saturated". The water and air which fill the available void space in a soil are known as pore-water and pore-air respectively.

G.2 Pore-water Pressure

Normally in a gravel or sand the voids are sufficiently large to allow the relatively free passage of water through the soil. Consequently, in the absence of an applied head of water, the pore-water lies in the soil under hydrostatic conditions. The top surface of such water is generally known as the ground water level.

In contrast, clays consist of very small particles and voids which do not allow water to pass through freely. This "low permeability", as it is termed, has a great influence on the physical behaviour of a clay. For instance, if a load is applied to a clay the increase in applied stress will be immediately balanced by a rise in the pore-water pressure in the zone of material influenced by the load. With time the water in this zone will flow to other regions where the pore-water pressure is lower and in consequence the clay wiil consolidate under the load. This pressure is analogous to the water being squeezed out of a sponge. After a considerable period of time a situation of pore-water pressure equilibrium will be reached.

When a large mass of clay has been disturbed in any way the pore-water pressure existing in that clay will give an indication of the extent and magnitude of that disturbance. The continued measurement of pore-water pressure can then be used to predict the return of the clay to equilibrium conditions. However, it must be emphasized that it can only provide a guide, particularly when the measurements have been made over a relatively short period of time.

G.3 Pore-water Pressure Measurement

Ground-water levels and pore-water pressures are determined by means of borings, observation wells, or various types of piezometers and hydrostatic pressure cells. During the advance of a borehole or immediately after installation of a pressure measuring device, the hydrostatic pressure within the hole or device is seldom equal to the original pore-water pressure. A flow of water to or from the boring or pressure measuring device then takes place until pressure differences are eliminated, and the time required for practical equalization of the pressures is the time lag. Such a flow with a corresponding time lag also occurs when the pore-water pressures change after initial equalization. It is not always convenient or possible to continue the observations for the required length of time, and adequate equalization cannot always be attained when the pore-water pressures change continually during the period of observations. In such cases there may be considerable difference between the actual and observed pressures, and the latter should then be corrected for influence of the time lag.

In addition to this,several sources of error in determination of ground-water levels and pressures occur, primarily when irregular and/or rapidly changing ground-water conditions are encountered. Regular conditions, with the piezometric pressure level equal to the free ground-water level at any depth below the latter, are the exception rather than the rule. Irregular conditions or changes in piezometric pressure level with increasing depth may be caused by: (a) perched ground-water tables or bodies of ground-water isolated by impermeable soil strata; (b) downward seepage to more permeable and/or better drained strata; (c) upward seepage from the strata under artesian pressure or by evaporation and transpiration and (d) incomplete processes of consolidation or swelling caused by changes in loads and stresses.

PLOWMAN CRAVEN AND ASSOCIATES REPORTS

UNIVERSITY OF KENT AT CANTERBURY

Report on establishment of markers for monitoring purposes, and first phase results

1. Installation of Monuments

Commenced on 22nd October, 1974.

Feet rests for the tripods were constructed for the 8 control traverse stations.

Final positions of the control stations were approximately as shown on Drawing No. 648/19, with the major exception of S.3 and S.4, which were positioned on the South side of the road.

Upon arrival on site to carry out the first phase of the observations, it was found that control station S.4 had to be moved due to lack of intervisibility, and that monitoring point N.3 also had to be moved in order that it would be visible from 2 control stations.

Monitoring points N.1 and N.3 were difficult to site, and are some way from the proposed locations shown on Drawing No. 648/19.

Monitoring points O.1 and O.3 also had to be located away from their proposed positions to avoid damage to the concrete path. They were sited in the flower-beds close to O.2.

O.2 is sited close to the temporary access route to the subsidence area, and as vehicles frequently passed close to it, may be expected to move in excess of its neighbours.

N.1 and N.3 are in the subsidence area and may be adversely affected by soil-tipping operations.

2. Field Observations

2.1 Methods

All observations were carried out as per the specification proposed on 30th August. 1974.

2.2 Instruments

All instruments, targets and reflectors were centred using a Wild ZNL optical plummet; angles were observed with a Wild T.2 theodolite, sighting onto Wild targets; distances were measured with a Tellurometer MA.100 onto Aga corner-cube prisms. Levelling was carried out using a Wild N3 geodetic level, with a built-in parallel plate micrometer, a Wild invar staff with supporting rods and a metal base-plate.

Continued ....

2.3 Referencing of control points

Referencing for future detection of large shifts of the control stations was by steel tape to two or more points of adjacent detail.

2.4 Base line checks

At stations S.1 and S.6, three distant objects were included in the rounds of horizontal angles observed in order to be able to detect any future swing in the control scheme.

2.5 Results of the angular observations

The angles in the control traverse closed to 1". All angles in the entire scheme were observed on both faces and three zero settings spread around the circle and the micrometer. Only two zeros were used on the monitoring points.

The residuals derived from the least squares adjustment were as follows:-

Main traverse	-	maximum	-	2.3"
		mean	-	0.93".
Monitoring points	-	maximum	-	3.7"
		mean	-	1.19".

All vertical angles were measured once on each face for checking of distance reductions only.

2.6 Results of the distance measurements

All distances in the control traverse were measured twice in each direction in the millimetre mode of the instrument, which in itself includes for a reversal of the phase in that mode, to enhance accuracy.

The distances to the monitoring points were measured twice in one direction only.

The comparison of the reduced forward and reverse distances in the control traverse show a maximum difference of 3.5 mm., and a mean difference of 1.48 mm.

The residuals derived from the least squares adjustment were as follows:-

Main traverse	-	maximum	-	.001 m.
		mean	-	.0003 m.
Monitoring points (being line S.4 - M.1, 77 m. long, or a difference of 1 in 26,000).	-	maximum	-	.003 m.
		mean	-	.0006 m.

Continued ....

2.7 Results of the levelling

Ordnance Survey, Newlyn datum was brought in from the Bench Mark on the North-East angle of the East face of "Monk's Well," just off Giles Lane. Accepted value - 67.43 m.

A single line of levelling was run through the control stations and the site datum bench mark, P.4. The misclosure was 0.003 m. in 20 instrument set-ups.

Three complete circuits of levelling were run through the monitoring points, reading both scales on the invar staff and trying to maintain back and fore sights as nearly equal as possible. A fourth circuit was run to check any slight anomalies.

The maximum difference in level obtained between any two stations was 1.6 mm. between O.1 and N.3. Only two other differences exceeded 1 mm.

The value of the datum bench mark, P.4, was accepted as 67.015 m. to the top of the steel rod.

Mean values of the levels of all other points are shown on the enclosed list.

3. Computations of position

3.1 Distance reductions

These were done both from station levels and from observed vertical angles. The latter provided a check only, and the former was used in the final computation. The formula used was simply $d' = \sqrt{(d^2 - \Delta h^2)}$, where Δh is the difference in level between instrument and reflector and d the mean measured distance.

Distances were then all reduced to mean sea level by the formula $d'' = \frac{d' \cdot R}{(R + hm)}$, a value of earth's radius, R = 6 370 000 m. being adopted. This latter reduction is made to reduce the entire scheme to a single reference plane.

3.2 Provisional co-ordinates

Computations of a simple traverse and bearings and distances to monitoring points were carried out to obtain provisional co-ordinates for the least squares adjustment.

3.3 Final co-ordinates

A least squares adjustment by variation of co-ordinates on the grid method was used, holding point S.5 as fixed, with arbitrary co-ordinates of E, 1000.000; N, 2000.000, and line S.5 to S.4 assigned a bearing of $20^\circ\ 00'\ 00''$.

Control traverse observations were assigned a weight of 1, and monitoring point observations a weight of 0.5.

3.4 Results

The control traverse closed to 1 in 390,000, being .001 m. in Easting and .002 m. in Northing over a total length of 976.993 m.

All the monitoring points were fixed by bearings and distances from two control stations. The maximum difference between the two sets of derived co-ordinates were at monitoring point O.3, where the differences were .002 m. in Easting and .007 m. in Northing.

The mean differences were .0015 m. in Easting and .0016 m. in Northing. Hence, apart from point O.3, it is probably justified in claiming an absolute positional accuracy of all points of better than 3 mm, point O.3 being accurate to only 4 mm.

All final accepted co-ordinates are tabulated in the enclosed list.

4. General Comments

Bad weather and the necessity to check some of the levelling resulted in this initial phase of the observations taking a little longer than anticipated, but it is to be hoped that subsequent operations will be quicker.

Co-ordinates of monitoring point N.3 derived from future observations will have to be examined in the light of its late emplacement.

Co-ordinates of monitoring point O.3 will also have to be compared bearing in mind the above spread of the two sets of co-ordinates.

Levels of monitoring points O.2, N.1 and N.3 will also need to be compared in the light of earlier comments about their locations.

5. Additional Work

12 levels on surrounding buildings, 3 piezometer points, 8 borehole locations and 2 additional temporary bench marks were requested on site, and are shown on a separate enclosed list.

D. Broome.

Plowman, Craven & Associates,
104-108 London Road,
St. Albans, Herts.

DB/DJN/74/1696
4th December, 1974.

CO-ORDINATE LIST

Job No: 74/1696	Location: University of Kent, Canterbury		Page No: 1	
Projection: Plane		Datum: Ordnance Survey, Newlyn		
Units →	Co-ordinates in: Metres		Heights in: Metres	
STATION	EASTINGS	NORTHINGS	Height	Remarks
S.1	1 307.052	2 100.509	66.152	
S.2	1 246.289	2 167.178	67.304	
S.3	1 162.291	2 210.354	66.674	
S.4	1 055.616	2 152.804	67.647	
S.5	1 000.000	2 000.000	68.367	
S.6	1 130.071	1 954.991	67.618	
S.7	1 257.375	1 887.017	59.091	
S.8	1 310.505	1 988.844	60.661	
M.1	1 103.364	2 160.414	67.672	
M.2	1 109.022	2 162.623	67.589	
M.3	1 114.419	2 164.639	67.677	
N.1	1 124.180	2 116.660	67.583	
N.3	1 127.314	2 121.960	67.473	
O.1	1 130.381	2 091.841	67.805	
O.2	1 132.361	2 093.082	67.822	
O.3	1 135.254	2 094.562	67.824	
P.1	1 150.675	2 038.966	67.940	
P.2	1 158.532	2 042.517	67.833	
P.3	1 167.062	2 047.422	67.941	
Q.1	1 208.562	1 900.892	62.460	
Q.2	1 214.302	1 902.386	62.207	
Q.3	1 219.772	1 904.229	62.059	

JOB NO. 74/1696 UNIVERSITY OF KENT

Number	Height	Description (See Diagram).
Building Levels		
B.1	68.688	Library - R.H.S. of window adjacent to exterior stairway.
B.2	67.868	Gulbenkian Theatre - E. side, R.H.S. last frame of window.
B.3	68.196	E Block - N. corner concrete ledge.
B.4	67.774	B Block - Floor, R.H.S. doorway.
B.5	68.596	A Block - E. corner N.W. face, L.H.S. first window ledge.
B.6	68.587	A Block - S. corner S.E. face, L.H.S. first window ledge.
B.6A	68.106	A Block - S. corner S.E. face, concrete ledge below first window.
B.7	68.614	A Block - S.E. face, R.H.S. fourth window ledge (from right)
B.8	65.984	Rutherford - L.H.S. entry under steps, S.E. end walkway.
B.9	67.307	Rutherford - L.H.S. doorway, top of steps.
B.10	64.697	Rutherford - L.H.S. doorway, top of steps.
B.11	63.408	Rutherford - R.H.S. ledge, left hand window.
Ground Levels		
G.1	67.698	Piezometer Points - S.C. cover.
G.2	67.61	Piezometer Points - Ground level.
G.3	67.59	Piezometer Points - Ground level.
G.4	67.86	Borehole 30.
G.5	65.89	Borehole 29.
G.6	65.92	Boreholes 28 and 28A (Midway 29 - 27).
G.7	65.81	Boreholes 27 and 31A.
G.8	65.84	Borehole 26.
G.9	63.31	Boreholes 25 and 32.
G.10	62.20	Borehole 24.
G.11	61.53	Borehole 23 (21 and 22).
	67.015	T.B.M. Top of steel rod - P4. Datum.
	68.583	T.B.M. L.H.S. top step - approaching Rutherford.
	68.573	T.B.M. R.H.S. top step - approaching Rutherford.
	67.43	O.S.B.M. N.E. angle, E. face, "Monk's Well."

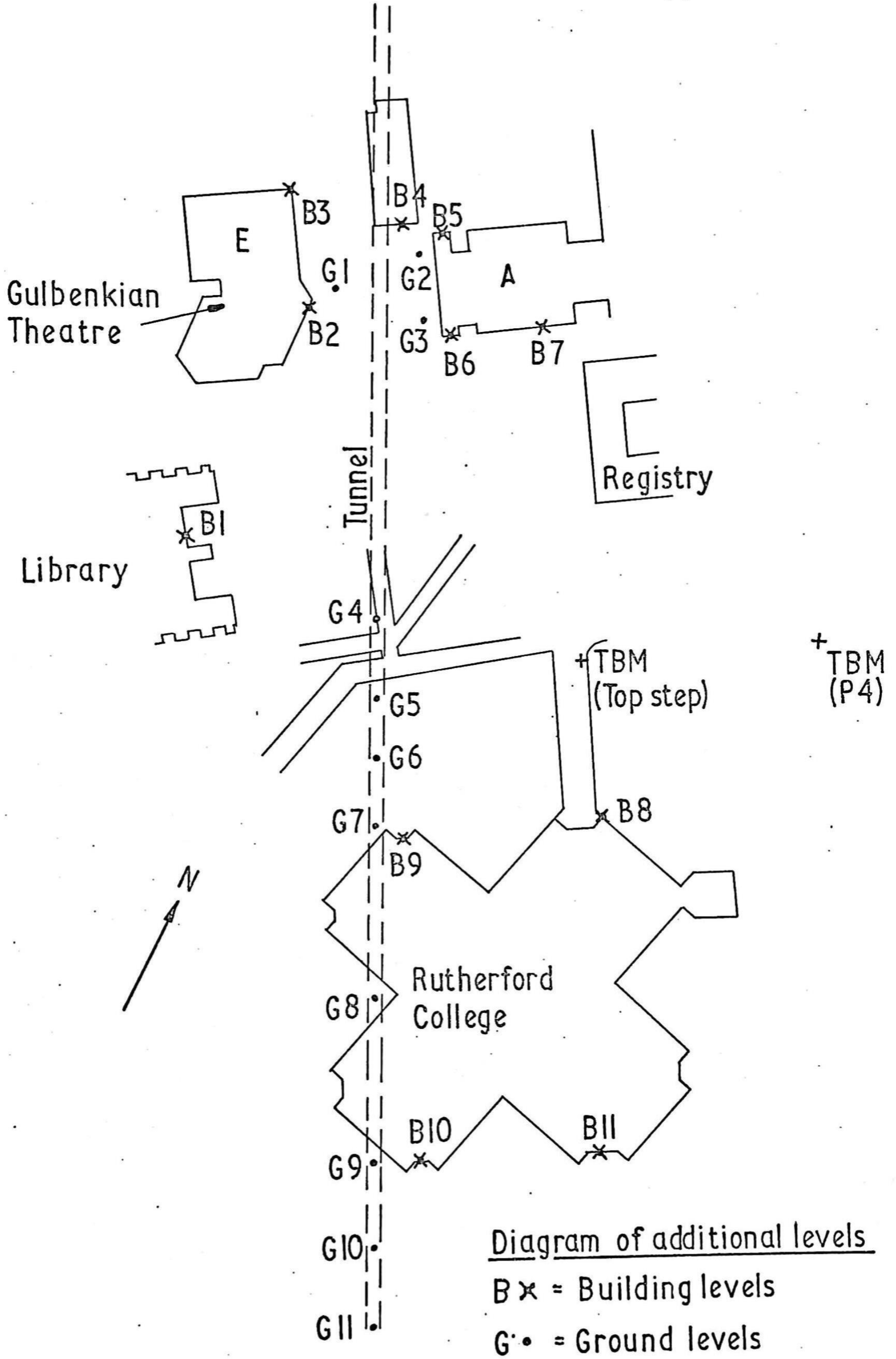

Diagram of additional levels

B ✕ = Building levels

G • = Ground levels

scale 1:1250

UNIVERSITY OF KENT AT CANTERBURY

Report on Second Phase of Monitoring Observations

1. Condition of Monuments

Traverse station S.3 had been disturbed and tripod footings were replaced, and S.4 had been covered by earth, but subsequent results indicate that no movement has been caused.

Monitoring point N has undoubtedly been disturbed by vehicles crossing it, and there is also some doubt about points N3, O1, O2 and O3 as they are in the area of greatest activity.

2. Field Observations

2.1 All methods and instruments were exactly the same as described in the initial Report dated 4th December, 1974.

2.2 Reference measurements were checked, and no differences noted.

2.3 Base-line checks: These were not possible to check due to continuous bad visibility throughout the survey.

3. Methods of Comparison

3.1 Position

The entire scheme was re-observed exactly as previously, and 2 runs were made on the computer. The first run was to hold only Station S.5 as fixed and to allow all the other data to vary. A least square solution was applied and the results confirmed the stability of the main traverse points.

The second run was then processed, holding the co-ordinates of the main traverse stations derived in the 1974 observations as fixed, and only introducing the new observations to the monitoring points as variables. This assumption, having proven the surround traverse, is obviously the best way to detect any movement in the monitoring points.

3.2 Levelling

Straight comparison of the final mean levels of stations is made.

4. Results

4.1 Angular Results

Due to holding the main traverse as fixed, the residuals derived from the least squares adjustment would be expected to be larger than in a free network. In this case, for the monitoring points, the maximum residual was 8.8" and the mean 3.6". These are still within the required limits to detect movements in excess of 5 mm.

4.2 Distances

The residuals here were:	Maximum	4 mm.
	Average	1.7 mm.

4. Results (Continued)

4.3 Co-ordinates and Heights

A table of the new monitoring point co-ordinates and levels is enclosed. However, to analyse the total shift, the following table gives the vector shifts in the form of bearings and distances:

Point	Bearing of Shift	Distance
M1	26½°	0022
M2	33¾°	0036
M3	56¼°	0036
N1	95°	0674 *
N3	90°	0070
O1	315°	0085
O2	301°	0117
O3	315°	0057
P1	341¾°	.0032
P2	0°	.0030
P3	135°	.0014
Q1	180°	.0010
Q2	161½°	.0032
Q3	180°	.0020

Building Levels

Point	Height - 1975	Comparison 1974 - 1975
B1	68.691	+ .003
B2	67.881	+ .013
B3	68.207	+ .011
B4	Access unobtainable due to Student Sit-In	
B5	68.602	+ .006
B6	68.600	+ .013
B7	68.617	+ .003
B8	65.973	- .011
B9	67.307	0.000
B10	64.696	- .001
B11	63.412	+ .004

1 2

It will be seen from the above that the groups of monitoring points all have different co-ordinates in the same general direction except P3, which is in the opposite general direction to P1 and P2.

5. Conclusions

5.1 Due to the methods employed and the specification in seeking shifts in excess of 5 mm, it is reasonable to state that the following points have not moved in position:

M1, M2, M3, P1, P2, P3, Q1, Q2 and Q3.

Point N1 has been seriously disturbed and the comparison is valueless.

Points N3, O1, O2 and O3 have changed between 5 and 12 mm, but being in an area of intense activity, it is still not possible to state categorically that the changes are due to ground movement.

5.2 (a) Levelling - Monitoring Points

As for the 1974 levelling operation, a single line of levels was run through the control stations using the same geodetic level and invar staff, starting and closing on P4. The misclosure was .0002 m. in twenty set-ups.

Two complete levelling circuits were run through the monitoring points, both scales of the invar staff being read at each pointing. Equal foresights and backsights were maintained.

The maximum difference between any two monitoring stations was .0010 m.

The mean of the two circuits was accepted as the final value.

(b) Building Levels

The points were levelled reading both scales of the invar staff at each pointing, and a straight comparison with the results obtained in 1974 is listed.

It was not possible to occupy B4 due to access problems caused by Student Sit-In.

All levelling is related to the T.B.M. value 67.015 m. established in 1974.

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DB/BJN/74/1696
24th March, 1975

CO-ORDINATE LIST

Job No: 74/1696	Location: University of Kent, Canterbury			Page No: 1
Projection: Plane		Datum: Ordnance Survey, Newlyn		
Units →	Co-ordinates in: Metres		Heights in: Metres	
STATION	EASTINGS	NORTHINGS	Height	Remarks
M1	1 103.365	2 160.416	67.673	
M2	1 109.024	2 162.626	67.590	
M3	1 114.422	2 164.641	67.678	
N1 *	1 124.247	2 116.653	67.604	
N3 *	1 127.321	2 121.960	67.497	
O1 *	1 130.375	2 091.847	67.840	
O2 *	1 132.351	2 093.088	67.845	
O3 *	1 135.250	2 094.566	67.830	
P1	1 150.676	2 038.969	67.941	
P2	1 158.532	2 042.520	67.833	
P3	1 167.063	2 047.421	67.943	
Q1	1 208.562	1 900.891	62.463	
Q2	1 214.303	1 902.383	62.211	
Q3	1 219.772	1 904.227	62.063	

* Points disturbed.

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Report on Site Visit made to check Building levels - 7th April, 1975

Personnel:	A.J. Handley and A.J. Elliott.
Instrument:	Wild N3 Geodetic Level.
Equipment:	Wild Invar 2 m. staff, and Wild fixed-leg Tripod.
Weather:	Strong gusty winds and rain showers.

Method

A closed circuit of levelling starting and finishing on Site T.B.M. - value 67.015 m., reading the building levels as intermediates on both scales of the staff. Change points (10) were taken on a staff base plate and another 3 taken on existing monitoring points as a progressive check - these points being P1, O3 and N1.

Results

Total misclosure of the circuit was 0.0028 mm. This amount of error was probably caused through the weather conditions.

On comparison of the final reduced and corrected heights, the largest difference was 0.004 mm. - this being on B2.

An error of 0.011 mm. was found on point B8, but on closer inspection of the March 1975 reductions, this was resolved, making the difference between the November 1974 levels to be 0.002 mm. instead of the issued 0.011 mm. This time a level was also taken on point B4, as previously access to this point was unobtainable. The comparison between the result and the November 1974 result was 0.005 mm. Values obtained this time substantiate the results from the March 1975 visit.

Conclusions

It must be realised that the points levelled on the buildings, in certain cases, are not well defined and error in values is unavoidable.

Efforts should be made to establish more stable points on the buildings in question, such as metal markers, rather than taking levels on the window sills which are constructed in wood and aluminium. This would enable far more reliable comparisons to be made.

A.J. Handley

Plowman, Craven & Associates,
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AJH/BJN/75/1696
8th April, 1975

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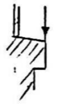
Description and condition of Building Level Points

B1 Flat wooden sill - easily identified. Comparison should be good.

B2 Sloping wooden frame  Level should be taken on front edge. (Maximum possible error 0.10 mm.)

B3 Corner of building on concrete plinth. Comparison should be good.

B4 Floor level inside doorway. Slight error expected on first comparison due to repositioning of step, but further comparisons should be good.

B5 Sloping aluminium window sill. Levels should be taken of front edge (maximum possible error 0.015 mm.)

B6 Sloping aluminium sill (buckled due to movement of building materials through window by contractors). Not stable - difference in comparisons could be high. Level should be taken on front edge.

B7 Sloping aluminium sill. Level should be taken on front edge (maximum possible error 0.005 mm.)

B8 Front edge of concrete step - stable.

B9 Top of concrete steps - stable.

B10 Top of concrete steps - stable.

B11 Sloping concrete window sill - stable. Level should be taken on front edge (maximum possible error 0.008 mm).

N.B. All possible errors are based on the maximum difference in slope.

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Building Levels

	March 1975	April 1975	Difference	November 1974	Difference - April 1975/ November 1974
B1	68.691	68.690	- .001	68.688	+ .002
B2	67.881	67.877	- .004	67.868	+ .009
B3	68.207	68.207	-	68.196	+ .011
B4		67.779		67.774	+ .005
B5	68.602	68.601	- .001	68.596	+ .005
B6	68.600	68.598	- .002	68.587	+ .011
B7	68.617	68.617	-	68.614	+ .003
B8	*65.982	65.984	+ .002	65.984	-
B9	67.307	67.307	-	67.307	-
B10	64.696	64.699	+ .003	64.697	+ .002
B11	63.412	63.413	+ .001	63.408	+ .005

* Error found in reduction.

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AJH/BJN/75/1696
8th April, 1975

JOB NO. 75/1696 - UNIVERSITY OF KENT, CANTERBURY.

Summary of the Reduced Levels of the Building Height Points and Monitoring Points obtained by Precise Levelling.

Height Point	Reduced Level Nov. 1974	Reduced Level March 1975	Reduced Level April 1975	Reduced Level May 1975
B1	68.688	68.691	68.690	68.690
B2	67.868	67.881	67.877	67.878(74)
B3	68.196	68.207	68.207	68.207
B4	67.774	-	67.779	Destroyed
B5	68.596	68.602	68.601	68.601
B6	68.587	68.600	68.598	68.598
B7	68.614	68.617	68.617	68.617
B8	65.984	65.982	65.984	65.983
B9	67.307	67.307	67.307	67.306
B10	64.697	64.696	64.699	64.698(98)
B11	63.408	63.412	63.413	63.413
M1	67.672	67.673	Not levelled	67.673
M2	67.589	67.590	" "	67.589
M3	67.677	67.678	" "	67.677(77)
N1	67.583	67.604	" "	67.602(02)
N3	67.473	67.497	" "	67.494
01	67.805			
02	67.822	67.845	" "	67.843
03	67.824	67.830	" "	67.828

Note. The close agreement between the April and May reduced levels gives a false impression of the reliability of these height points.

Because of the nature of points B2, B5, B6, and B7, little conclusion can be gained about building movement in these areas.

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16th May, 1975.

Job No. 75/1696 - University of Kent at Canterbury

Summary of Precise Levelling carried out in June, 1975

Point No.	Reduced Level	Change since May		Remarks
B1	68.689	- .001		
B2	67.869	- .009		Unreliable point; agrees with original value.
B3	68.208	+ .001		
B4	- Destroyed -			
B5	68.600	- .001	)	
B6	68.596	- .002	)	Unreliable points.
B7	68.616	- .001	)	
B8	65.984	+ .001		
B9	67.308	+ .002		
B10	64.700	+ .002		
B11	63.407	- .006		Sloping concrete sill; agrees closely with original value.
L1	68.6834		)	
L2	67.8837		)	
L3	68.4650		)	
L4	68.4466		)	Quality of marks enables
L5	68.4468		)	levels to be quoted to
L6	68.4433		)	0.1 mm.
L7	68.4677		)	
L8	66.1070		)	
L9	67.3451		)	New points: grouted bolts,
L10	64.7331		)	B.R.E. pattern.
L11	63.4299		)	
M1	67.674	+ .001		
M2	67.591	+ .002		
M3	67.678	+ .001		
N1	67.600	- .002		
N3	67.494	0.000		

Since the major localised disturbances ceased, there have been no movements greater than 2 mm per month, except N1 and N3, 4 mm and 3 mm respectively, which could still be affected by minor works being carried out, but which must be closely monitored.

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19th June, 1975.