

DISCUSSION ON PAPER 8630

Observations on the behaviour of a pilot tunnel in London Clay as the main tunnel was driven

H. S. H. Thomas

Mr R. N. Craig, Sir William Halcrow & Partners

The use of spheroidal graphite iron is now common and nearly all cast iron linings are manufactured with this iron. Grey iron may be used for the smaller diameters or where the moulds are still available and the quantities required are small.

41. The Paper states that the hoop stress increased when the face was within a distance of the depth of the tunnel (§ 36). On the Jubilee Line running tunnels at Green Park and Regents Park movements began to occur when the face was about 1–2 times the depth from the point of observation. The measurements were by inclinometer.

42. The small increase in hoop stress is not surprising, when the tunnel is excavated with a series of pilots. The ground above needs to be supported so that part of the load is transferred on to the pilot iron. This could increase the loadings by 20–25% rather than the 10% measured.

43. When pilot tunnel 3 was under construction, pilot 1 would move towards it. The diagonal of pilot 1 in fact moved towards pilot 3, with little movement in the vertical and horizontal planes. When the main tunnel was excavated around the pilot tunnels, little excavation was needed at the side of pilot 1. I would therefore not be surprised if the vertical diameter increased and the horizontal diameter decreased. Fig. 4 only shows the movement connected with the excavation for the main face.

44. Although during the final stages for the excavation of the main tunnel, the vertical diameter increased and the horizontal diameter decreased, taken as a whole the reverse occurred. The vertical squat was several times the small vertical increase over the final stages. I am not convinced by the argument that the increase in the vertical diameter was due to the additional horizontal stress in the clay caused by its overconsolidated state. The measurements are very small, 0·1 in, compared with 0·4 in for the initial movements.

45. At the Heathrow cargo tunnel, where the clay cover was small, about 2 m, the inward ground movements at the axis were 0·3 in, and in the crown and at the invert 0·45 in and 0·3 in respectively. On the Author's argument the vertical diameter should have increased compared with the horizontal diameter. On the Jubilee Line the downward vertical ground movements prior to the erection of the lining were 12–18 mm, compared with 5–6 mm inward movements at the axis. The argument that the overconsolidated state of the clay caused horizontal squat of the lining at Brixton is therefore not convincing.

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46. I am aware of other cases of horizontal squatting. At Sizewell the outfall tunnels were constructed in compressed air in sand. The movements were small and the construction or the grouting could have caused the horizontal inward movement. A second instance was the Metropolitan Water Board's Southern Main where their wedge block lining was used. The wedge in the crown could have caused the horizontal squat.

Dr R. J. Mair, Scott Wilson Kirkpatrick and Partners

I believe that the measurements made by Mr Thomas provide a useful contribution to understanding what happens to an existing tunnel structure ahead of an advancing heading, and it is rare to see such well documented field data on this topic.

48. Two interesting observations emerge from the Paper. First, the hoop stress in tunnel 1 was observed to increase by about 10% when the main tunnel face was between $2\frac{1}{2}$ diameters and half a diameter from the measurement point (Fig. 3). Second, at the same time, the horizontal diameter reduced and the vertical diameter increased (Fig. 4). These two effects are interpreted in the Paper as being due to locked-in horizontal stresses in the London Clay (§ 36). I would like to question whether this is in fact the reason for the observations, and suggest that they may instead be consequences of elastic behaviour of the ground irrespective of the horizontal stresses in the clay.

49. The changes of stress in front of an advancing tunnel heading are rather complex. It is helpful to consider first the simpler two-dimensional case, as shown in Fig. 6(a). As the radial stress is reduced due to tunnel construction, the mean normal stress remains constant if the ground behaves isotropically and elastically (provided the tunnel is not very shallow), i.e.

$$\frac{1}{2}(\sigma_r + \sigma_\theta) = \text{constant} \quad (1)$$

Therefore

$$\Delta\sigma_r = -\Delta\sigma_\theta \quad (2)$$

In other words, the circumferential stress increases as the radial stress reduces. This is the familiar result known as arching. If the shear stresses exceed the strength of the clay, plastic behaviour occurs and the mean normal stress then reduces with reducing radial stress.

50. Consider now the state of stress of a cube of soil at a distance x in front of an advancing tunnel heading, as shown in Fig. 6(b). If it is supposed that the

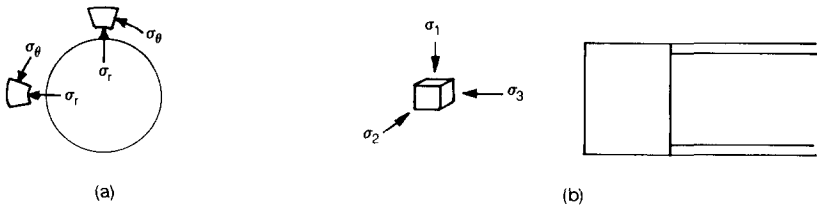


Fig. 6. Stresses adjacent to tunnels: (a) two-dimensional case; (b) three-dimensional case

ground behaves elastically for values of x greater than, say, half a diameter in front of the face, then again (for isotropic soil) the mean normal stress will remain constant for a deep tunnel:

$$\frac{1}{3}(\sigma_1 + \sigma_2 + \sigma_3) = \text{constant} \quad (3)$$

Therefore

$$\Delta\sigma_1 + \Delta\sigma_2 = -\Delta\sigma_3 \quad (4)$$

The horizontal stress σ_3 would be expected to begin to reduce as the main tunnel face approaches. Equation (4) shows that the sum of the change of σ_1 and σ_2 increases as σ_3 reduces due to the approaching tunnel, and the hoop stress in the tunnel lining would therefore be expected to increase. Also, because of the influence of the ground surface, the vertical stress will remain approximately constant and $\Delta\sigma_1$ may therefore be smaller than $\Delta\sigma_2$. If this is the case, the transverse horizontal stress σ_2 will increase, albeit by a relatively small amount, due to the reduction in horizontal stress σ_3 . This conclusion does not depend on the magnitude of the existing stresses in the ground; it is merely a result of elastic behaviour of the clay.

51. An increase in horizontal stress transverse to the tunnel axis (σ_2) was observed in a series of model tests conducted on the Cambridge geotechnical centrifuge as part of a research contract awarded by the Transport and Road Research Laboratory. By making use of symmetry, as shown in Fig. 7, a semi-circular tunnel heading was constructed in a block of clay. The clay was soft, with an undrained shear strength of 30 kN/m². Silver markers were placed in the clay and then a Perspex window was fitted, which was lubricated with a film of grease. The model was accelerated at the end of the centrifuge in a gravity field of 125 g , with stability of the tunnel maintained by compressed air supplied to a flexible rubber bag within the tunnel. The compressed air pressure, σ_1 , was then steadily reduced until failure of the tunnel heading occurred.

52. Some photographs taken during such a test are shown in Fig. 8. These were taken when the model was travelling at a speed of about 200 mile/h. In Fig. 8(a) the air pressure was equal to the total overburden pressure at the tunnel axis (393 kN/m²) and the tunnel was stable. As the air pressure was lowered, so the tunnel stability reduced, and Fig. 8(b) shows significant deformation of the soil close to the tunnel heading when the air pressure was 124 kN/m². Fig. 8(c) shows the tunnel heading after collapse. Of particular significance in this sequence of photographs is the horizontal line just below the ground surface separating darker and lighter zones. The line marks the boundary above which the clay remains separated from the Perspex window. Below this boundary line the clay is in contact with the window. As the support pressure in the tunnel was reduced, the boundary line above and in front of the tunnel heading was observed to rise in all of the tests, and this can be seen in Fig. 8. The rise of this boundary line is associated with an increase in transverse horizontal stress (σ_2) in front of and above the heading, which accompanies the reduction in horizontal stress along the axis.

53. The observations of the model tests support the argument that a small increase of horizontal stress some distance in front of a tunnel heading, transverse to its axis, would be expected to occur irrespective of the horizontal stresses existing in the ground prior to tunnelling. This behaviour, which arises from consideration of the elastic response of the ground, appears to be consistent with the field observations of Mr Thomas.

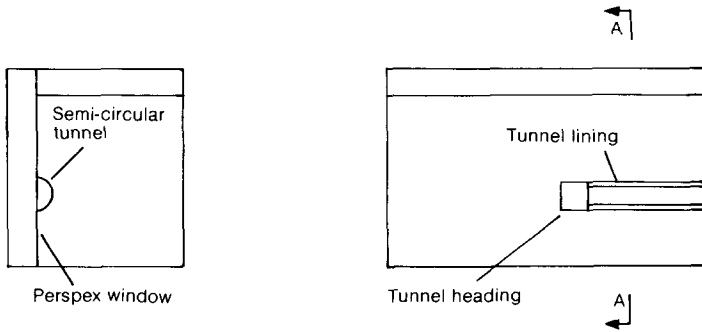


Fig. 7. Test arrangement for model tests on tunnel headings

Dr W. H. Ward, Consulting engineer

The observations have been made using techniques which were established by myself and my colleagues at the Building Research Establishment over the past 30 years; they are reliable and there is no doubt about the details of the tunnel distortions that have been presented.

55. However, I would question the interpretation of some of the observations, in particular the suggestion in § 25 that the small increase in hoop load and the 'peaking' type of diametrical distortion of tunnel 1, as it became encircled and finally unloaded by tunnel 4, is to be associated with a large ratio of horizontal to vertical in situ stress in the London Clay.

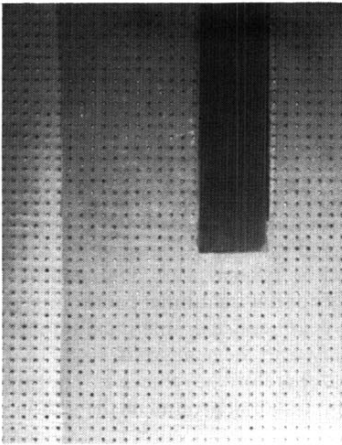
56. Some similar measurements of diametrical distortion of existing London Underground tunnels have been made.¹⁵ Two running tunnels at each of three sites, known as K, G and R, were encircled by larger tunnels and the original linings were removed. The relevant information on these tunnels and on tunnel 1 at Brixton is summarized in Table 1.

57. At each site, K, G and R, observations were made on three consecutive rings, lettered alphabetically in the table, in each of the two old tunnels. The rings consisted of six cast iron segments, and a crown key of 12 ft 6 in o.d.

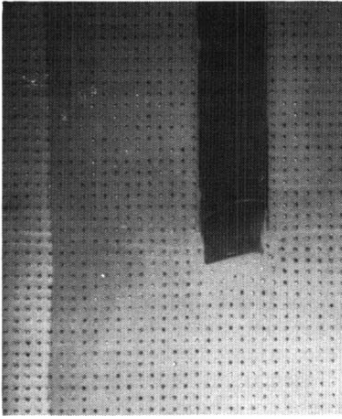
58. At site K the tunnels are in the lowest beds of the London Clay with saturated sand layers below them. There is about another 100 ft of London Clay beneath the tunnels at site G, while at site R the crowns of the tunnels lie in the base of the mottled beds of clay of Reading type with their lower parts in a very tough, sandy clay of Woolwich type.

59. In Table 1 the distance between adjacent existing tunnels at each site is given as a multiple of the tunnel diameter. At sites K and G the tunnels are sufficiently far apart for interference effects of construction to be negligibly small. However, at site R, as at Brixton, there was a strong interference which was recognized for the first time by the differing distortions as each tunnel became encircled and unloaded by a larger one. We could easily recognize that the tunnel containing rings G, H and J had been driven first and that the tunnel containing K, L and M had been driven later. Historical research by London Transport later confirmed the fact.

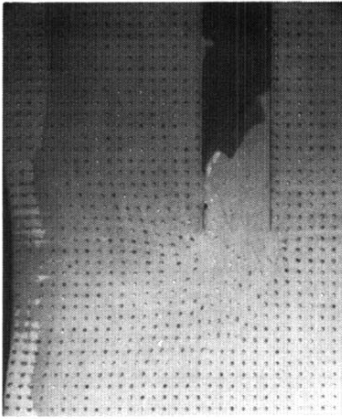
60. Consecutive rings in a tunnel distorted in a remarkably consistent way. The values of ΔH and ΔV in Table 1 are the average changes in horizontal and vertical diameter of the rings in each tunnel from its existing distorted state in the



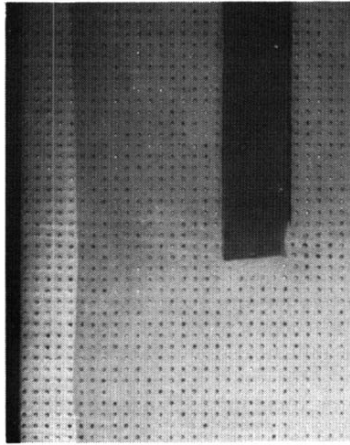
(a)



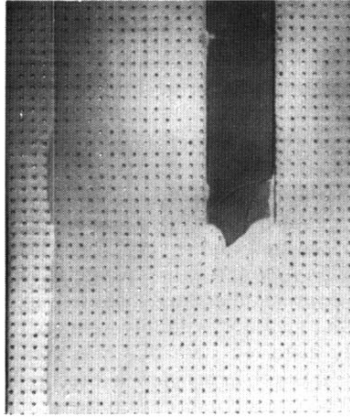
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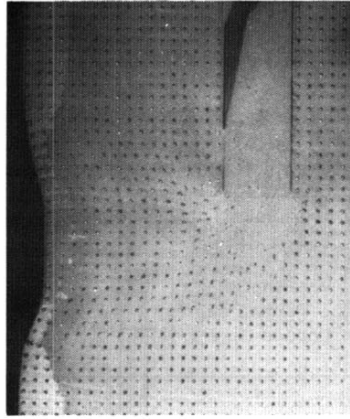
(e)



(b)



(d)



(f)

Fig. 8. Sequence of photographs taken during centrifuge model test as tunnel support pressure, σ_1 , was reduced

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Table 1. Information on the Brixton tunnels

Site	Ground	Tunnel depth, ft	Tunnel age, years	Rings	ΔH , in	ΔV , in	Tunnel spacing, diameters
K	London Clay (basement)	50	25	ABC	+0	+0.03	2.5
		60		DEF	+0.03	+0.06	3.5
G	London Clay	86	50	PQR	0	-0.06	2.2
				UWX	+0.03	-0.06	
R	Woolwich and Reading Beds	71	50	GHJ*	-0.13	+0.13	0.33
				KLM†	-0.06	+0.16	
Brixton	London Clay	72	0.6	—*	-0.11	+0.09	0.17
				—†	—	—	

* Tunnel driven first.

† Tunnel driven later.

ground to the condition when it was completely encircled by a larger tunnel and the ring no longer carried any ground loading. An increase in diameter is denoted by a positive number. At sites K, G and R the diameter changes were measured to the nearest $\frac{1}{32}$ in using a steel tape registering against centre-punch marks on the iron rings.

61. It can be seen from Table 1 that at sites K and G, where adjacent tunnels are over two diameters apart, the recovery distortion is small, with a slightly positive ΔH and a small increase or decrease in ΔV ; whereas in the first-built ring at site R and at Brixton, where there had been strong interference distortion from driving the second tunnel very close, the recovery distortions are much larger, of similar magnitude and of the 'peaking' type.

62. I consider therefore that the peaking type of tunnel lining distortion at Brixton and at site R is simply a minor recovery associated with unloading from the highly distorted state that was created by excavating another tunnel close alongside. It is not, in my view, a direct reflection of the natural state of stress in the ground. The stress conditions in the ground surrounding a tunnel will have been changed significantly as the first tunnel excavation was made and they will have been changed again each time another tunnel was excavated. The stresses in the thin 'neck' of ground between two tunnels closely spaced horizontally are certainly high.

63. The maximum diametrical distortion of tunnel 1 at Brixton amounted to about $\Delta H = +0.60$ in and $\Delta V = -0.48$ in¹ and hence the reverse recovery distortion was only about one fifth as large.

Mr P. Tedd, Building Research Establishment

Mr Thomas stated that the observations in the pilot tunnel suggest that the face of tunnel 4 liberated, in the surrounding ground ahead of itself, stresses which were greater in the horizontal direction than in the vertical. It seems very likely that the relatively large in situ horizontal stresses in the ground would have been relieved by the construction of the pilot tunnels. There are still considerable uncertainties about the undisturbed in-situ stress in London Clay, and the changes in stress that occur due to the construction of a tunnel. It would therefore have been interesting to have monitored these changes in stress in the surrounding ground as the various tunnels were constructed.

65. A suitable instrument for making such measurements of total earth pressure is the push-in spade cell. Two makes of the instrument are shown in Fig. 9. The cell basically consists of an oil-filled chamber which is connected via a short length of steel tube to a pneumatic transducer. Both cells have a piezometer associated with the cell in order that effective stress can be calculated.

66. Recent experience with the cells in stiff clays^{7,8} suggest that they over-read the actual in situ stress, as may be expected from the method of installation. They are usually pushed at least 0.5 m into the ground from the end of a borehole. In order to estimate the magnitude of this over-read, some cells were pushed into London Clay from horizontal boreholes drilled from the side of a 3 m dia. tunnel at a depth of 9 m. Cells were installed about two diameters away to measure vertical stress. The values obtained were then compared with the known overburden pressure (Fig. 10). An equilibrium pressure was reached about one month after installation. It can be seen that the cells over-read by a small but significant

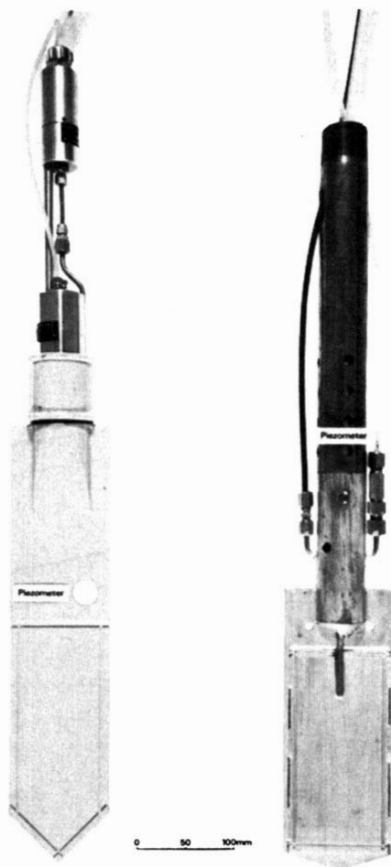


Fig. 9. Push-in pressure cells: (a) soil instruments; (b) Glötzl

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amount. At the same site pressure cells were also installed to measure horizontal stress parallel to the line of the tunnel. Fig. 10 shows that the measured horizontal stresses were significantly larger than the measured vertical stresses.

67. Push-in pressure cells have also been installed to measure horizontal stress adjacent to a propped cantilever wall in London Clay. Pressures have been monitored from before construction until after final excavation on both the active and passive sides of the wall. Very satisfactory results have been obtained. The opportunity to monitor changes in stress in the ground surrounding a tunnel due to its construction would, I believe, provide some very interesting results.

Dr J. Temporal, Transport and Road Research Laboratory

About nine years ago we installed a number of load and pressure cells into the lining of a tunnel in the London Clay at Regents Park.¹³ Fig. 11 shows that the horizontal force on the lining is lower than the vertical force, but both are steadily increasing with time. In general the load cells give higher short-term readings than the pressure cells because of the time taken for the clay to form an intimate contact with the slightly recessed pressure cells, but the load cells give lower long-term readings because of their lower stiffness. A logarithmic extrapolation of the data indicates that at the end of 100 years the loads are unlikely to exceed about 90% of the overburden.

69. We also installed some pressure cells of the type that Mr Tedd has described, in the Oxford Clay fairly close to a 2.85 m dia. tunnel at 18 m depth.¹⁶ The main reason for installing the cells was to look for high horizontal stresses because there had been high lateral movements of the Oxford Clay into the tunnel

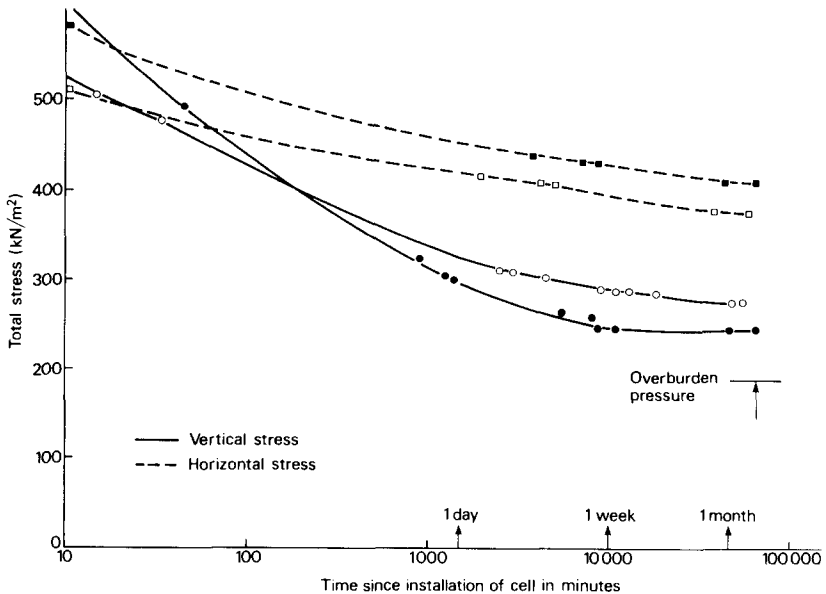


Fig. 10. Dissipation of earth pressures measured by push-in pressure cells

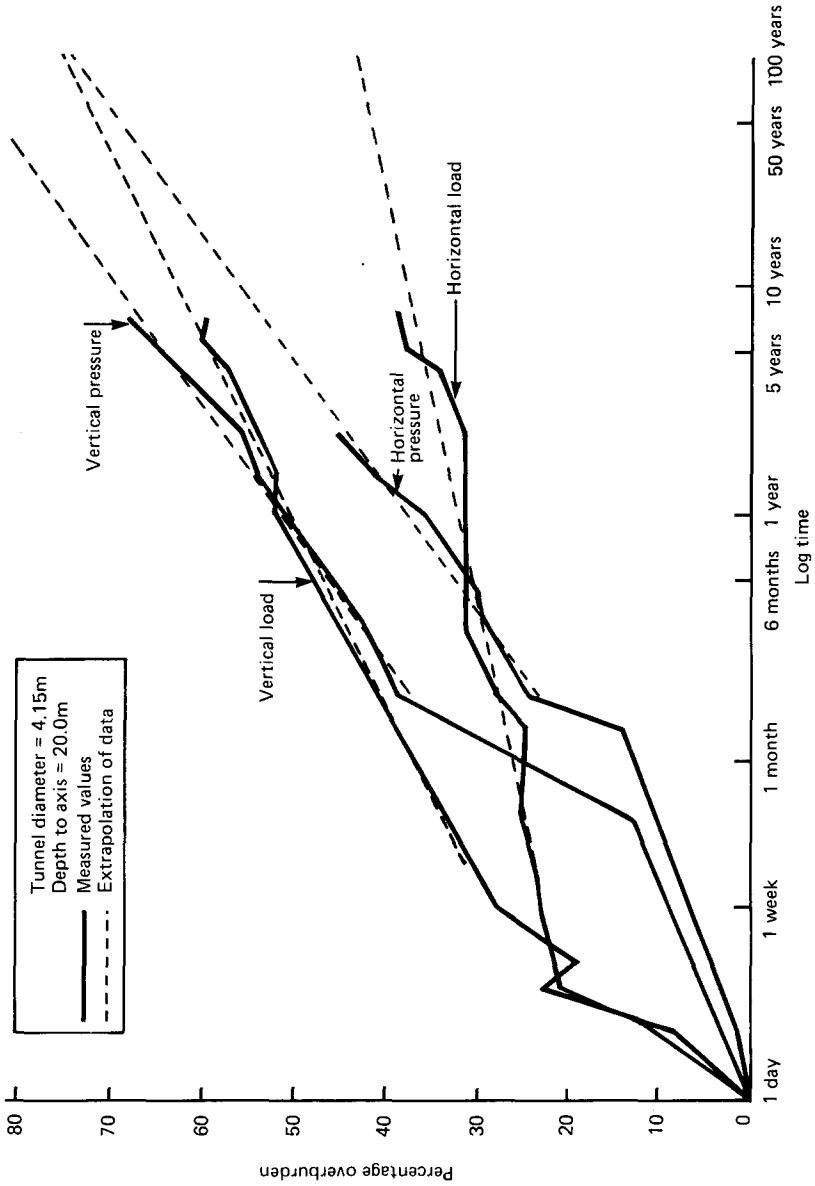


Fig. 11. Lining loads in London Clay at Regents Park

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during excavation. We discovered that at three diameters away from the tunnel the horizontal stresses were of the same order as the overburden stress. This seems to suggest that stress relief occurs to some extent past three diameters from the sidewall of the tunnel. We are putting more cells into the Oxford Clay at a greater distance from the sidewall to shed more light on how far the stress relief extends.

Professor J. B. Burland, Imperial College of Science and Technology

Perhaps the most obvious case where an increase of stress is associated with unloading is the simple case of a uniform circular load applied to the surface of an elastic homogeneous half space. Fig. 12 shows the distributions of vertical and horizontal stress changes beneath the centre of such a load. It can be seen that the radial stress changes $\Delta\sigma_r$ depend on Poisson's ratio. For positive applied pressure q and a low Poisson ratio, $\Delta\sigma_r$ becomes negative below a depth approximately equal to the radius of the loaded area. Conversely if the applied pressure is negative $\Delta\sigma_r$ becomes positive below this depth. If we turn the picture on its side and imagine the face of the tunnel as the unloaded area, at a distance of about half the diameter of the tunnel beyond the face the radial stress changes along the tunnel

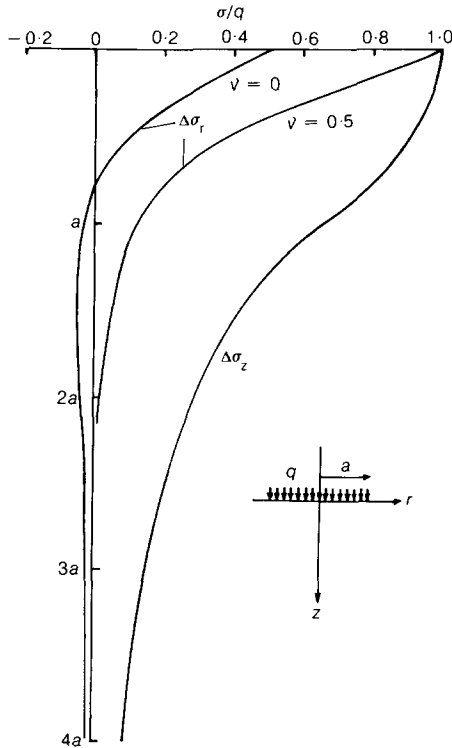


Fig. 12. Distributions of vertical and horizontal stress changes beneath the centre of a uniform circular load

axis will become positive. Beyond a distance of about two to three tunnel diameters the radial stress changes will be negligible. This simple analytical model appears to go some way in accounting for the observed changes in hoop stress given in Fig. 3 of the Paper. This is particularly the case for London Clay which is anisotropic and has a very low Poisson ratio in the horizontal plane.

Mr A. J. Howard, Mott Hay & Anderson

I have some practical knowledge of the Brixton area. I was the Resident Engineer at Brixton and Stockwell Station and beforehand had been involved in the preparation of contract documents and the site investigation. Brixton is an interesting illustration of the anisotropy of the London Clay.

72. We went out to tender with a specification which would certainly have permitted and perhaps even encouraged a tunnelling machine for the fairly long drives from somewhere south of Brixton Station right the way through to Stockwell. During or just before the period of tendering some large-diameter boreholes were sunk into the clay. On cutting windows into the casings we found that around the tunnel level in certain parts the clay was 'backy', that is, it had greasy slickensides in it. We added something to the specification about maintaining the stability of the face, which virtually cut out the possibility of tenderers being able to foresee the use of a tunnelling machine. In the event, an expanded concrete lining was being used, but fingers had to be put in the back of the shield because of trouble with the ground falling in.

73. For the ticket hall, which was immediately between the two station tunnels, we were underpinning in pits down to a level below the station tunnels and we had no problem in the London Clay. We did lower our machine chamber, which is of 26 ft dia., before the station tunnel enlargement, and we made certain, because of the effects of ground movement described by Mr Thomas, that we put up some big timber. We did not have any particular trouble in taking the timber out. We were not aware of any tendency to change shape in that big cast iron tunnel, but that is perhaps rather weak circumstantial evidence.

74. In Caracas where we were constructing twin running tunnels with a full-face tunnelling machine, coming within a metre or so of each other, I expected that there would be trouble if the second were dug and the other not supported. The contractor eventually put up something which was really like using a matchstick to keep a liner off the quay, and nothing happened in fact, although I believe this was more by luck than judgement.

75. We used spheroidal graphite iron at the Tyne and Wear Metro with great success. We found that at 7 m dia. it was cheaper to use nodular than grey iron. Everything at and above 7 m was nodular iron, and we were able to use its important tensile properties by turning over the insides of the flanges. Concerning the changes in stress in tunnel 1 as tunnel 2 passed, Peck¹⁷ has done empirical work and produced a graph which relates the increase in stresses in tunnel A as tunnel B passes to the separation. Were the Brixton results compared with this work?

Mr J. B. Boden, Building Research Establishment

I would like to thank Mr Thomas for seizing the opportunity to add to his earlier measurements at this site, which he reported in 1976.¹ Now we have a complete record extending from the construction of the first pilot tunnel to the completion of the final cross-over enlargement.

Table 2. Tunnelling lining loads deduced from hoop strains measured in the linings and expressed in terms of equivalent hydrostatic pressure as a percentage of the total overburden pressure $[\sigma_L/(\gamma \times z) \times 100\%]$; after Thomas,¹ Ward and Thomas,³ and personal discussions with Thomas

C/D	5	5.5	5.6	6.7
$\left(\frac{\sigma_L}{\gamma \times z} \times 100\%\right)$	45	89	68	75

77. Reliable field evidence of this type is the vital ingredient in advancing our fundamental knowledge of soil/structure interaction, so that we can more readily transfer experience gained in one circumstance to another.

78. Recent model studies carried out at Cambridge University and reported by Atkinson and Mair¹⁸ suggest that there is a systematic relationship between the cover depth/tunnel diameter ratio (C/D) and the long-term load imposed on the tunnel lining. An examination of the field data reported by Thomas¹ and Ward and Thomas,³ summarized in Table 2, shows no such systematic trend. Obviously the loads would be expected to increase with time, but all of these observations relate to a point in time when the loads were only increasing very slightly. Time effects are therefore only able to explain a relatively small proportion of this scatter. A much more important factor is likely to be the variation in the ground-water regime at the different sites. The results of the Cambridge work¹⁸ tend to confirm this view. Measurements of total earth pressures and pore-water pressures at the sites studied by Thomas and Ward would therefore be of considerable value. The spade-shaped cells which are now available, as described by Tedd and Charles,⁷ would be ideally suited for this purpose. Other factors possibly contributing to this scatter could be the type of lining, and the method and timing of installation of the lining in the tunnelling cycle, e.g. bolted and grouted or expanded. This could involve characteristic line type of effects as described by Ladanyi¹⁹ for yielding rocks.

79. The lining deformations reported by Mr Thomas appear to fit with what we would expect for the case of a shallow tunnel constructed in an overconsolidated clay. The shear strength of an overconsolidated clay would be expected to change with time as a consequence of the tunnel excavation. Immediately upon excavation, pore-water suctions would be induced in the clay, dissipating thereafter as the ground started to return to a long-term equilibrium. Thus, in the clay around the tunnel, the maximum shear strength would be expected immediately upon excavation reducing with time thereafter. Arching theory²⁰ can be used to give a qualitative indication of the influence of these changes on the tunnel lining deformations as follows.

Immediately following excavation

80. In his earlier paper¹ Mr Thomas quoted the undrained shear strength of the clay as being about 350 kPa. Simple arching theory suggests that the vertical pressure acting on the lining would be zero at undrained shear strengths considerably less than this value (about 125 kPa). Segmental linings, when built within the

tail-skin of a shield, tend to sag under their self-weight. As the lining emerges from the tail of the shield, the thrust from the rams will encourage the lining to change shape. Since the vertical load at this stage is likely to be small, as indicated theoretically above, there is an opportunity for the vertical diameter to increase. This mode of deformation was observed by Mr Thomas during the first week following erection of the lining.

Long term

81. A total stress analysis using the undrained shear strength of the soil is only appropriate for use in the circumstances immediately following tunnel excavation. As the pore-water suctions dissipate and the ground-water regime starts to return to a long-term equilibrium, an effective stress analysis must be used.

82. Unfortunately there are no data available relating to the ground-water regime at this site. If an effective angle of shearing resistance for the clay of 24° is assumed, and the pore-water pressure is taken to be equal to zero (in the absence of any data), then simple arching theory suggests that the vertical pressure acting on the lining would be of the order of 60% of the overburden pressure, and that the horizontal pressure would be about 35%. In reality, these values would be modified by whatever ground-water pressures were acting. However, at this stage (a few months after installation) the vertical pressures would still exceed the horizontal pressures. In these circumstances, the vertical diameter of the lining would be expected to decrease and the horizontal diameter to increase. This mode of deformation was observed by Mr Thomas until it was reversed by the influence of the approaching enlargement. The interpretation of the model experiments given by Dr Mair offers a possible explanation for this final reversed mode of deformation. However, it is also possible that the high lateral field stresses, present in the larger volume of clay being influenced by the enlargement, may have contributed to this behaviour, as suggested by Mr Thomas.

83. The experimental observations presented by Dr Temporal, relating to a tunnel of comparable depth and diameter in London Clay, yield identical values of vertical and horizontal pressures to those which I have calculated theoretically. One of the fascinations and frustrations of attempting to apply theoretical soil mechanics to experimental data is that, as on this occasion, the only time that you ever obtain a numerically correct answer is when you are absolutely certain that your method of analysis is invalid! This, of course, is one of the reasons why reliable field evidence of the type being discussed is so vitally important in our attempts to advance the state-of-the-art.

Mr T. Bateman, Stanton and Staveley Ltd

The need for observations is vital in this business. When trying to design a tunnel one usually finds that one does not even have one basic piece of information about the kind of loads that are going to come on it, and anything that will lead to information on that, or on the kind of deformations that will happen, is useful.

85. A possible mechanical engineering analogy for driving a tunnel through clay is that of drilling a hole in a piece of material that has residual stresses in it: one can have a mathematical formula to determine the relief stresses around the hole, but only as far as the drill has advanced, not ahead of it. However, one does have to be extremely careful about drilling the hole. One has to drill it as slowly as possible with as sharp a drill as possible, because the vibrations and the effects of removing the material can drastically affect the relieving of the residual pressures around it. So when drilling through stratified material underground the machine

being used may well be influencing conditions ahead and affecting the stresses.

86. I am pleased that, throughout the Author's earlier paper,¹ the graphs show that the ductile iron linings are more flexible or not as robust as the grey iron ones in terms of weight, and yet the Author concludes that they were just as good as the grey iron ones, and Fig. 10 of that paper even shows that the shape of the twelve segments is as good, if not better than, the shape of the six grey iron ones.

87. I am pleased to know that breakages occurred frequently on the grey iron linings but not on the ductile ones. In terms of overload or the unexpected happening, ductile iron is better than both grey iron and steel because of its stress-strain curve. It clearly has twice the design strength of grey iron, it has the ductility that grey iron does not have and, although its modulus of elasticity is less than that of steel, steel will reach a certain stress and then deform very much in the case of overload, whereas ductile iron can carry on taking the load until well above that stress level. Thus ductile iron linings may be permanently deformed but not greatly so, and will still be holding up the tunnel overbearing load.

Mr G. K. Haselden, Fellow

I was the Engineer for A. Waddington & Son who built this length of Victoria Line at Brixton and I was responsible for dealing with the ductile iron. The 12 ft 6 in dia. running tunnel iron (into which the ductile iron was built as pilot for comparison) is under-designed: it is perfectly safe I am sure when it is in place, but it is fragile under construction. To me the 22 rings of ductile iron caused a lot of trouble. The computer always forgot that there was a length of 2 ft wide iron as compared with the 1 ft 8 in standard running tunnel lining. Moreover, we had to account for every single plate of iron that was put into the tunnel and that included the ductile. In my view the contractor should be saved from such experiments. However, I hope that what we did in this was of use.

Mr Thomas

I appreciate the point made by *Dr Mair* that relief of stress at the tunnel face can cause an increase in the transverse horizontal stress behind the face whether or not that stress had been previously greater than γh . I am grateful to *Professor Burland* for providing an example of a similar model of unloading showing an increase in compression extending behind the unloaded area for a distance comparable with that observed at Brixton.

90. *Dr Mair* reminds us that gravity and time play dominant roles in soft ground tunnelling and ensure that unless effects initiated at the face remain very small they will be propagated to the surface. It is interesting to compare present with early (c. 1900) experiences of tunnelling in London Clay recollected by Sir Basil Mott²¹ addressing the Institution of Civil Engineers in 1924: '... a very slight hair crack in the surface of the street was observed about 50 feet in advance of the tunnel face, and it extended hour by hour as the work progressed 60 ft below. ... (Now) the street is, as a matter of fact, about half an inch narrower than it was before the tunnels were constructed'. Transverse bending giving rise to a zone of tension overlying the compressive one is not often recognized.

91. It is likely that a full description of the cause of what was measured at Brixton would involve the overlying ground. Tunnel 4 is large and was built, without a shield, ten times more slowly than tunnel 1. The instrumented tunnel lining acted as quite an accurate earth pressure cell and gave an early indication of

changing stress. When, a little later, deflexions became noticeable, these were very small in terms of strain (measurable to 10^{-5} strain).

92. Should any future worker have the opportunity to instrument the ground through which a tunnel is to be driven, in the light of the above he would be well advised to accept *Mr Tedd's* recommendation of using an array of earth pressure cells. Not only is earth pressure change the most appropriate measurement when investigating the transfer of stress within a soil mass, but it can also be measured more accurately there than can strain or deflexion. Glötzl cell readings are repeatable to ~ 0.5 bar (equivalent to measuring strains to about 5×10^{-5}) which would be difficult to do with horizontal strains using instruments in vertical boreholes.

93. *Mr Craig* states that inclinometer measurements on the Jubilee Line running tunnels showed that lateral movements were recorded when the face was quite a long way away. This conflicts with the measurements on the Victoria Line running tunnels by *Mr Smyth-Osborne* and ourselves (1970).^{22,23} These show that lateral convergence of the ground did not start until the face was one diameter away. These measurements were made with a micrometer (to ± 0.001 in) in a nearby tunnel at the same level. This bore on a pushrod anchored in the clay very near to where the tunnel was going to pass. Using the same method from a chamber on the line of the approaching tunnel, movement which did begin earlier was measured on the axis of the tunnel prolonged behind the face. This movement, first observable at 25 ft behind the face, was away from the face until it was 18 ft away, after which it was towards it. So perhaps it is an axial strain in the ground giving this sort of deflexion which should be associated with the increasing hoop stress, unaccompanied by diametrical deformation, which was the first indication that tunnel 1 was being affected by the face of tunnel 4.

94. The apparent discrepancy between the transverse measurements made on the Jubilee Line and those on the Victoria Line is resolved by *Barratt and Tyler*.¹³ They state that at the time of the Green Park and Regents Park work the pendulum-type inclinometer running in a fluted tube was barely accurate enough to show the *total* change due to a tunnel passing; still less would the start of a trend have been recognizable. Accuracy five times better could be expected now.²⁴ If inclination is to be measured, fixed tilt meters are much better. The Building Research Station has had good results with electrolytic levels.^{25,26}

95. *Dr Ward* has drawn attention to similarity between *Brixton* and his sites K, G and R reported in *Ward and Chaplin*.¹⁵ It is agreed that there was recovery from the state of stress induced in the lining of tunnel 1 when tunnel 2 was driven past it. (Fig. 10(c) of reference 1). The recovery is illustrated by the relief of bending stresses shown in Fig. 5(c) of the present paper. It did not begin to happen until the face of tunnel 4 was 2 ft away.

96. The earlier histories of the running tunnels at sites K, G and R were probably different from those at *Brixton* because the encircling tunnels were only of 19 ft 6 in and 20 ft 6 in i.d. respectively. There was one enlargement for each running tunnel and at site R they were staggered.

97. The measurements of local total earth pressure on small parts of the lining, which *Mr Boden* and *Dr Temporal* mentioned, were tried by the Building Research Station in 1957. A piston was set in the centre of each of six segments of a ring, and each piston bore on a vibrating wire load cell. The scatter in the measurements made was probably due to the 'pocketing' mentioned by *Dr Temporal*. Also the flexibility of the ring of lining tended largely to equalize the earth pressures. Since later experiments were on even more flexible rings we thereafter used the average

hoop strain measured by strain gauges to calculate the earth pressure. When piezometers were inserted in segments they read zero; the linings were uncaulked. Perhaps we missed reading some negative values due to inadequate technique.

98. Although recognizing that Dr Tyler's pressure cell²⁷ used by TRRL is superior to the arrangement described above, particularly when used with an expanded lining, the Author would prefer an instrument of much larger area and with a curved sensitive surface if he were to instrument a stiff preformed lining.

99. The Author thanks *Mr Howard* for indicating the backy nature of the clay overlying the tunnels. Backs in the ground above the face would make it less able to withstand lateral pressures (whether they were original or caused by the excavation) so that these pressures would be more likely to be shed on to the ground ahead and deeper.

100. Mr Howard also asks how the interaction of tunnels 1 and 2 compares with the rules in Peck *et al.*¹⁷ This says that, for equally sized tunnels spaced half a radius apart edge to edge, 'the distortion of the first tunnel may be assumed to be doubled by the second'. Whether this means the distortion as the second tunnel passes or all the distortion that would occur if no twin tunnel had been built is not clear. Their diagram shows the diametrical distortion of the first tunnel as twice that of the second, which could be different again. Tunnel 2 was not instrumented. In terms of outside radii the tunnels at Brixton were one third of a radius apart.

101. Usually when the diameter-time curve of a first driven tunnel is studied, the parts before and after the rapid change due to the other tunnel passing have the appearance of a smooth curve interrupted by a step due to the passing (see e.g. Ward and Thomas⁹). It seems possible that the uninterrupted curve represents what would have happened to a single tunnel. At Brixton, however, the continuations of the curves after the passing were not typical. The curves for the horizontal diameters in that period (August to December) have hardly any slope (Fig. 13).

102. Table 3 compares the diametrical changes for the closely spaced tunnels at Brixton with two rings in tunnels at the usual London Transport spacing of 21 ft centres. Using lines (d) and (c) to calculate the ratio in the bottom line is closer to the ideas of Peck *et al.* than using lines (d) and (a) since the resulting figures straddle the value of 1 which they expect for a spacing of $D/4$ or $r/2$.

103. The local curvatures associated with the passing face at Brixton were high¹ but neither kind of lining was in any danger of breaking. Running tunnel iron has been known to break when small cable bypass tunnels (8 ft i.d.) have been driven past about 4 ft away, whereas Peck *et al.* say that this situation would be only twice as severe as the equal size case.

104. I sympathize with *Mr Haselden's* having had a length of tunnel with non-standard iron included in his job. I cannot recall whether Waddingtons were represented at the meetings between London Transport and Stanton and Staveley, when a trial length of spheroidal graphite (SG) iron lining was first proposed, but I certainly think they should have been.

105. As to the use of the trial, Mr Craig, Mr Howard and *Mr Bateman* have said that SG iron has now replaced flake graphite iron for a large proportion of cast iron linings. This implies a saving of a raw material which can only become more scarce and perhaps Waddington's miners appreciate the lighter segments.

106. BRS is very grateful for the respect shown for its instruments which enabled them to be read right up to the end of the life of the tunnel, hence this Paper, and for Waddington's ready co-operation which enabled us to follow closely the progress of the work.

Table 3. Comparison of changes in diameters of four first driven tunnels before, during and after passing of twin tunnels: from Ward and Thomas⁹ and Thomas¹

Location	Manor House				Brixton Crossover			
	D/2		D/6		D/6		D/6	
Separation								
Lining	12 ft 6 in concrete		12 ft 8 in iron		12 ft 7 in iron		12 ft 8 in SG iron	
No. of segments	14		6		6		12	
Diameter change	Horizontal, in	Vertical, in	Horizontal, in	Vertical, in	Horizontal, in	Vertical, in	Horizontal, in	Vertical, in
(a) Before passing	0.23	0.25	0.20	0.25	0.08	0.06	0.09	0.05
(b) After passing	0.07	0.11	0.16	0.22	0.01	0.07	0.02	0.06
(c) Sum (a) + (b)	0.30	0.36	0.36	0.47	0.09	0.13	0.11	0.11
(d) During passing	0.22	0.18	0.16	0.13	0.34	0.26	0.47	0.30
Ratio $\frac{(d)}{(c)}$	0.7	0.5	0.4	0.3	3.8	2.0	4.2	2.7

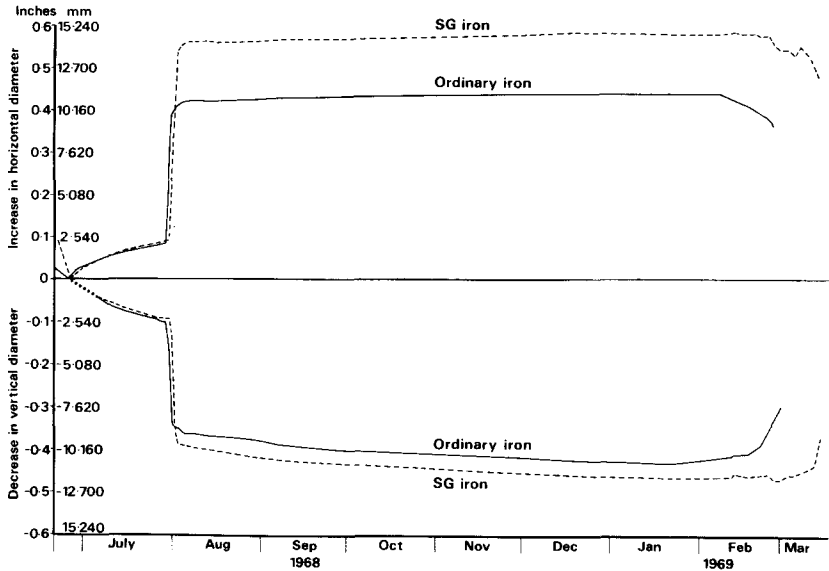


Fig. 13. Changes in diameter of the two instrumental rings

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