TUNNELLING IN LONDON CLAY

A study with field investigations including the long-term structural behaviour of two linings

A thesis submitted to the University of Surrey

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Building Research Station Department of the Environment
Marc Isambard Brunel, 1769 - 1849, inventor of the tunnel shield

from painting of 1813 by J Northcote, in National Portrait Gallery
SUMMARY

Excavation, temporary support and lining of tunnels in London Clay have undergone repeated development. Innovations depended however on the economic and social climate rather than on technical feasibility.

An innovation, which could soften the impact of forthcoming shortages of materials and fuel on tunnelling, is spheroidal-graphite cast-iron.

Experimental work in the laboratory and in a temporary tunnel, on bolted segmental linings in both the old and the new kinds of iron is described. It shows that rings built from iron segments are less stiff than intended, but that in service this stiffness is not required.

Observations of instrumented rings in the tunnel on over fifty occasions, some whilst other excavations were made nearby, also throw light on the interaction between neighbouring tunnels and between linings and the ground.

Measurements were made of displacements in the clay near the advancing tunnel face. A mechanism to explain the observed behaviour is suggested and the conclusion drawn that surface subsidence is best reduced by using an 'expanded' lining (a kind of lining which is built directly against the clay behind the shield) with a versatile tunnelling machine capable of sustaining good progress.

Nevertheless, an appreciation of the possible hazards of tunnelling in the London Clay formation made clear by an explanation of its geological
history and by the study of the experiences of earlier engineers, indicates that contingencies occur which require recourse to some form of bolted lining.

Hence a lining has been designed which embodies the conclusions from the experimental work and which may be built in either the bolted and grouted form or the expanded form. Partly constructed in spheroidal graphite iron, this material is economically used by restricting it to those functions requiring its special properties.
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1.1 THE LONDON CLAY

The London Clay is a formation which is part of the Eocene system. That is to say that it is rock (in the geological sense) formed from material continuously deposited over a time interval beginning about 50 million years ago. Before and after this interval there were times when no deposition occurred.

The original material was mud brought down by a large river and deposited slowly and steadily in the sea near the shore. Palaeontological evidence also suggests that the river was draining low-lying land covered by tropical rain forest. A present day analogue might be the River Ganges flowing into the Black Sea. Mineralogical analysis suggests that the river flowed from the west over Dartmoor (Groves, 1931).

The minerals which constituted the original mud were mainly finely grained quartz and the clay mineral illite in roughly equal proportions, with small quantities of micas and felspars *(Davies, 1919)*. There are few analyses of the clay minerals but one splits the 50 per cent clay content into 35 per cent illite, 10 per cent kaolinite and 5 per cent montmorillonite (Grim, 1949).

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*There was also a suite of less common minerals: garnet, staurolite, tourmaline, topaz, zircon. Although accounting for only 0.005 per cent of the total by weight, these may be used now to identify the parent rocks from which the sediment was derived (Groves 1931).*
The deposition, continuous on a time scale of million of years but intermittent on a scale of days, depended on the varying bed-load of the river. Hence this has been laid down as thin strata. The finely divided clay minerals would have flocculated in the sea water and been deposited in random orientations but the larger mica particles settled differentially in horizontal arrangement giving rise both to partings delineating the strata and to laminations within them. Because the benthic life was restricted to micro-organisms (see below) the fine stratification and lamination have been preserved (Cullen, 1973).

Occasionally the clay was absent and a thin stratum of silt sized (0.002 mm - 0.06 mm) particles of quartz and felspar would be laid down. More often a sharp increase in the flow of the river would enable it to carry a large load of sandy silt. On reaching the sea the silt laden water, having a higher density than sea water, would flow rapidly down to the coastal platform as a turbidity current. This would have had enough energy to cut into the previously laid strata before depositing its own suspended material. In this way lenses of silt of limited lateral extent formed.

Terrestrial floral and faunal remains which had been carried out to sea and the bodies of pelagic organisms sank to the bottom. Sedimentation would not have occurred quickly enough to bury them deeply. The ordinary processes of bacterial decay in these conditions soon depleted the oxygen in the water and made life impossible for ordinary benthitic sacrophagic animals. Anaerobic conditions prevailed. Anaerobic bacteria degraded the organic matter to very simple molecules; \( \text{CH}_4, \text{H}_2\text{S} \) etc. Due to their mobility and low density most of these degradation products floated to the surface, but the highly soluble ones such as \( \text{H}_2\text{S} \) and \( \text{CO}_2 \) were retained in solution.
The pore water in the sediment at this stage was sea water containing an abundance of the chlorides of sodium and magnesium and smaller amounts of the sulphates of magnesium and calcium (Clarke, 1926). The chlorides remained dissolved while the sediment was immersed, but the sulphates, which are much less soluble and have a higher energy state, became involved in the metabolism of the anaerobic bacteria. These reduced the calcium sulphate to sulphide and the sulphide reacted with carbon dioxide to form calcium carbonate. This was precipitated and the reaction continued as more sulphate ions diffused in; thus quite large calcareous concretions were build up (Hatch and Hastall, 1965, pp 331 and 337). Hydrogen sulphide liberated by this and similar processes reacted with any ferruginous components of the sediment. The ensuing reaction formed the commonest allogetic mineral in the London Clay, pyrite, either as disseminated crystals or pyritised wood or other fossils (Love, 1964). At a later stage of diagenesis, after microbial activity had ceased, energy was sometimes available for some of this sulphide to be oxidised and calcium sulphate formed, as the mineral selenite, in well-shaped crystals. The author has not observed this to co-exist with calcareous concretions.

A large proportion of the mud particles were between the sizes $10^{-9}$ m and $10^{-8}$ m and so formed colloidal suspensions in water. Although van der Waals' forces are sufficient to produce colloidal effects most of these particles were of clay minerals which are only electrically balanced when ions are loosely attached to their outer layers. These ions may be lost and regained or exchanged. Thus minor chemical changes may produce Coulombic electro-static forces of attraction or repulsion.

Some of these electrical forces tended to cause the soil skeleton to assume
more compact forms. In doing so water was expelled from the soil skeleton. Centres of contraction would occur at spacings which were small compared with the whole bed of sediment. Between these centres areas of tension occurred. On a plane at right angles to a line joining to two adjacent centres the tension would be maximum, the structure most open, and the water content greatest. If the sediment had not been subject to an all-round compressive stress due to accumulating overburden, the coincidence of tension and conditions tending to produce a weak material would have tended to induce a network of cracks. Such 'syneresis cracks' may be seen in many colloids, eg bentonite gel, if they are left standing. They form a pattern which is the three-dimensional analogue of the hexagonal shrinkage pattern seen when shrinkage occurs in a tabular body. However, in the case of a sediment subject to overburden pressure, the tensions became locked-in discontinuities of stress. The relative tensions which surrounded the centres at a distance were balanced by relative compressions close to the centres.

The stresses locked into the buried sediment coincided with planes of weakness. Much later when the sediment had turned to a rock, the London Clay, and had been exposed by erosion there would be a tendency for these planes to rupture.

The accumulating overburden was building up vertical stress in the young sediment. This produced vertical and volumetric strains which occurred by rotation of the platy particles and collapse of the voids between them. This brought the flat, charge-bearing surfaces of the clay particles into juxtaposition despite the Coulombic repulsion between like charges. These surfaces also have an important effect on the pore-water close to the
Because the associated ions are not free to migrate into the pore water an osmotic pressure is set up in it. Due to the large specific surface of the clay minerals, much energy can be stored in this way although the mineral particles need not be in mechanical contact. A theoretical estimate of the maximum energy is $10 \text{ MN/m}^2$ (100 tons/ft$^2$) acting over a distance corresponding to a change in void ratio from 1 to a ratio of 20 (Bolt, 1956). In the mixture of roughly equi-dimensional quartz particles and flat mica and clay particles that was to form the London Clay, many clay particles would be held too far from their neighbours for this energy storage potential to be fully realised. However, sufficient overburden would accumulate for a great deal of energy to be stored in this manner. Its recovery would not depend on mere reduction of the overburden, but would require restoration of the pore water that had been expelled against the osmotic pressure.

Throughout the several million years of the deposition of the material which has formed the London Clay, the seabed was sinking. Thus the depths and gradients influencing the diagenetic processes were maintained and a formation which is uniform, considered on a large scale, has resulted. This movement also marks the beginning of the formation of gentle folding in the underlying chalk, which was to become more marked in post-Eocene times and form the London and Hampshire Basins. By late Eocene times deposition on top of the London Clay in the two Basins was sufficiently different that formations in the two Basins have been given different names. In the London Basin they are the Claygate Beds and the Bagshot Sands with present day thicknesses of 15 m (50 ft) and 35 m (115 ft) respectively. The tectonic movements produced the usual vertical joint planes and sloping fault planes in the London Clay.
Rocks of Oligocene, Miocene or Pliocene ages are absent from, or at best doubtfully authenticated in, the London Basin. That is to say that for 38 of the 40 million years since the end of the Eocene period the geological record is very obscure. Deposits may have been laid down and later completely eroded.

A full but complicated record of Pleistocene times has been left in the London Basin showing that the Thames had been established and was taking various routes across the region at different times and that four glaciations had occurred during which periglacial conditions prevailed here (Wooldridge and Lynton, 1955). From these times on fluvial or fluvio-glacial channels would have been cut in the top surface of the London Clay and filled with a variety of alluvial deposits. Also fresh water would always be available so that a hydraulic gradient across the Claygate beds, the London Clay and the Woolwich and Reading beds and into the pervious Chalk would leach out the soluble salts from the connate waters of the clays and replace sodium by calcium as the exchangeable cation in the clay minerals.

The diagenesis and later geological history of the London Clay have produced a rock which for a full description requires not only the familiar lithological description but a recognition that it is in a state of stress (and ideally measurements of this). Because exposing the rock, as in sampling or boring a tunnel, necessarily relieves the stress, the rock cannot be observed in its virgin state. Also, because the relief of stress is time dependent, the manner of making the exposure changes the material that is examined or in which the miner works.

Causes of the state of stress include:

6
(i) overburden load

(ii) the local out of balance forces which cause local residual stresses and

(iii) tectonic forces.

In (i) the stress is due to the greatest cover of overburden during history of the formation. When this stress is removed water is able to enter the clay and the energy stored by the mechanism described on page 5 is able to do work on ground supports or tunnel linings. Because the clay is not very permeable this takes some days, but in the times when the timbering for a length of brick tunnel was in place for a fortnight or more either the 'swelling pressure' of the ground had to be resisted or the deformations associated with it had to be allowed for.

The residual stresses (ii) due to the process of syneresis mentioned on page 4 cause the clay to have a propensity to crack along the in-built planes of weakness. This process starts immediately the stress is relieved and within an hour the hitherto continuous mass of clay will have become a collection of polyhedral blocks bounded by undulating and often interlocking planes. The further behaviour of the clay mass will be much influenced by these cracks particularly if they provide a pathway to a water bearing layer. The pressure of the pore water in the clay bordering the cracks can rise, so that the clay here becomes very weak and the blocks are in effect lubricated so that the bulk strength of the clay is also dramatically reduced.

Not being due to gravitational or other outside forces, the cracks are not oriented but are distributed in nearly random attitudes (Skempton, Scholster and Petley, 1969). Near the surface where the stress has already been
largely released the cracks are already open. This is often indicated at depths as great as 6 m (20 ft) by traces of tree roots that had sought an easy path through the clay. Because all possible planes of weakness have become cracks the size of the blocks near the surface can become quite small (around 5 cm, (2 ins)) whereas at depth in a tunnel, when the cracks have just occurred, blocks of 0.5 m (18 ins) are found.

In the terminology of soil mechanics these cracks are known as 'fissures' (and in the engineering part of this work this term will be used). In geology, as in ordinary English, the word 'fissure' means an open cleft. Presumably the early investigations were looking at exposures which were both shallow and old, and therefore open, but nevertheless the choice of this word for such small scale features seems strange.

One cannot, with any certainty, state that stresses due to tectonic forces remain in the ground since the evidence that tectonic forces were ever acting is based on the dislocations they cause and these are evidence of the dissipation of the forces. Direct field measurement is very difficult as also is the verification in the field of the hypothesis supported by laboratory data (Skempton, 1961) that there are high horizontal stresses corresponding to an earlier higher overburden load, whereas the stress in the vertical direction is only equivalent to the present overburden load unless swelling (in the soil mechanics sense) is allowed to occur.

The usual sets of orthogonal vertical joints that occur in most sedimentary rocks are found in the London Clay (eg the Wraysbury site described by Skempton, 1969). Usually they accord with geological definition of being fractures of the rock mass along which there has been little or no displacement,
but occasionally they do show polishing or slickensiding suggesting movement. The more usually found polished surface is the 'greasy back' of miners' parlance, a shallowly dipping plane often of several square metres area, probably a small local thrust fault. There is evidence (page 206) that not very much movement is required in order to produce the polish. Movement of joint bounded blocks seems to occur in very deep tunnels and it may be that if the ground stays uncracked (the 'fissures' do not appear) the stress in it can only be relieved by movement on joint planes and bedding planes.

1.2 GEOMORPHOLOGY OF THE LONDON BASIN

The post Eocene folding has formed a shallow syncline plunging in an easterly direction. Its axis runs in the direction N 60° E passing through Kingston on Thames and Blackwall. A geological section is shown in Fig 1. The dip of the northern limb is in fact only about 1 in 250. The southern limb dips more steeply and also has minor folds such as the Crystal Palace syncline superimposed. Faults traverse the southern limb running roughly parallel to the strike of the main syncline and throwing to the North West. The most notable are the Greenwich fault and a longer one running from Raynes Park to Deptford.

The full thickness of the London Clay (as evidenced by the presence of the Claygate beds) varies in different parts of the Basin: 137 m (450 ft) at Esher and Wimbledon, 114 m (375 ft) at North Hampstead, 110 m (360 ft) at Crystal Palace, 64 m (210 ft) at Harrow Hill, 61 m (200 ft) at Stanmore.

*Geological Survey (Institute of Geological Sciences) 1:63360 scale geological maps, sheet 256 (North London) and sheet 270 (South London), also Bromhead and Dines (1925) and Dewey and Bromhead (1921).
Figure 1  Geological section of the London basin.
There are hills where the London Clay has been protected by a cap of gravel or sand. The Bagshot sand overlies the Claygate Bed and London Clay at Esher Common, Harrow Hill, Highgate and Hampstead. Glacial gravels cap Hangar Hill, Richmond Park and Wimbledon Common. The 'plateau gravels' (of glacial origin but apparently derived from the south) cover St George's Hill and Crystal Palace hill. The pebble gravels, which are thought to be pre-glacial, extend in a broken belt along the northern limb of the syncline: at Bushey Heath, at Elstree and at Barnet for example. The Boyne Hill Terrace, the oldest of the three Thames terraces, was laid down early enough to protect Clapham and Wandsworth Commons and account for the tongue of high ground running south from Islington to Myddleton Square. The first tunnels in London Clay were driven to provide routes of shallow gradient through these hills.

The present-day drainage is based on the Thames, the east-west valley of the river crossing the syncline at a small angle. The longer northern tributaries such as the small rivers Colne, Lee and Roding once drained a glacial area. They then cut deep valleys which have become filled with gravel and they may have changed course. The upper surface of the London Clay is therefore dissected with gravel filled 'buried channels'. In central London, both north and south of the Thames, old rivers such as the Tyburn, Westbourne and Effra have disappeared (Barton, 1962), but they too have buried channels of sufficient depth to be a tunnelling hazard.
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   Explanation of Sheet 270. Memoir of the Geological Survey of England and Wales, HMSO.


Soft ground tunnelling had its origins in mining and military engineering. Before power was available for large pumps, minerals were usually sought by driving an adit into the side of a hill; this provided access through the soft ground and, because the adit sloped downwards to the portal, drained the workings. The military application was the destruction of otherwise unapproachable fortifications by mining under them and using explosives or setting fire to the tunnel supports to collapse the overlying structure. Since both the soldier and the prospector had to live off the country timber was the only tunnel lining material available.

2.1 TIMBERED HEADINGS

A long tradition of this work carried on by men who, by the nature of their calling were travellers resulted in the technique of 'square set timbering' and 'fore-poling' being widely known and the methods of working with them becoming fairly uniform. Two Cornish miners, Robert Vazie and Richard Trevithick, had brought the method to London and driven a drift 305 m (1000 ft) long at a depth of 23 m (76 ft) under the Thames by 1808. This small drift was intended to assist the eventual building of a larger tunnel and in this capacity many more were to be built in the London Clay.

The timbered heading is nevertheless a good example of how a tunnel as such should be built in soft ground such as London Clay given that only timber and manpower are available and that the size of the tunnel need only be large enough to accommodate vehicles that can be moved by a man.
Fig. 2  Timbered heading
The compression members in this lining are built as trapezoidal frames called sets or settings. These comprise a horizontal member on the floor called the sill, a somewhat shorter horizontal piece at the roof called the cap (sometimes collar or bar) supported on two nearby vertical pieces called posts (sometimes legs, sides or props). The posts were tenoned into mortices in the caps. Sometimes the caps had V-notches into which was fitted a sharper V cut into the post (Welsh notch). The last built set was close to the face (or breast to use the older word) which was retained by contiguous horizontal breast boards supported against the set. Typically the clear space inside the set would be 1.4 m (4 ft 8 in) high, 0.8 m (2 ft 7 in) wide at the top and 0.9 m (3 ft) wide at the bottom. The timber would be 10 cm (4 in) in diameter (see Fig 2). The sets were placed at intervals of two thirds to one metre according to the state of the ground. Poling boards 2.5 cm (1 in) thick were lodged on the caps to form a roof and the sides were closed with 20 mm (3/4 inch) thick poling boards.

If running ground was encountered, and in tunnelling in London Clay there is always a chance of finding a buried river channel or a waterbearing silt lens, forepoling was used. Forepoles or spiles were boards sharpened to a chisel point at one end; most of the spiles were also wider at this end. The sharp and broad end was inserted above the cap of the last built set and hammered into the ground. More boards were driven until a continuous sheeting had been inserted ahead of the face. The length of each spile was about two and a half times the spacing between the sets. Because the spile was longer (about two and a half times in the most secure system, than the spacing between the sets and because the cap was thick, the spile had to be driven with an inconveniently steep inclination to the axis of the tunnel. In very loose ground the spile could be tilted inward after
Fig. 3 Forepoling with closely spaced sets
driving, but a safer method was to make every alternate set smaller so that the angle was determined by twice the set spacing and the thickness of the cap. The detail is shown in Fig 3. The roof of the drift was usually the only part forepoled, but if necessary all four surfaces could be so made. If this was done the whole tunnel was effectively made in the ground ahead of the face, and if this were excavated by moving one breast board at a time complete control could be maintained, even with a moderate amount of water flowing out of the face. Short struts were usually put between the corners of one set and those of the next to stop the sets from tilting. For clarity these have been omitted from Fig 3.

2.2 BRICK LINED TUNNELS

Brick had been used for permanent civil engineering works in London—particularly forming culverts over streams that had become sewers. This led to the building of cut and cover brick arched sewers. They could not be deep because they drained into the Thames so tunnelling was not considered.

Used underground, brick could not provide protection whilst the tunnel was driven so that timber was required for temporary support of the ground as well as for centring for the brick arch itself.

2.2.1 The English system of tunnelling

The system of timbering used here was called the English system and is shown in Fig 4. It differed from the earlier French and Belgium systems in that a central core of earth was not left but the whole face was excavated and timbered for a length of twelve feet or so. This left a space clear for the bricklayers to work separately from the miners and facilitated the building of the 'inverted arch' or invert which proved to be necessary in London.
Figure 4  English system of timbering.
Figure 5 English system: brickwork
Clay and Weald Clay. Also the pilot heading, in square set timbering, could be driven at invert level thus providing drainage. Sumps were even included so that the water was available for mixing mortar.

The English system also employed less timber than the European ones; the woods favoured here were the indigenous oak (Quercus robur) and the larch (Larix europaea) which had been introduced into England in the seventeenth century. The main economy in timber was that the 30 cm (12 in) diameter larch timbers called bars which held the poling boards against the clay travelled with the work and were re-used in each length. The bars ran parallel to the length of the tunnel and were spaced at 75 cm (2 ft 6 in) centres all round the arch and held apart by a number of short struts. The bars rested on the finished brickwork at their rear-most ends and on props supported in turn on horizontal sills of 35 cm x 35 cm (14 in x 14 in) square section at the working face.

The invert brickwork was built on the ground which had been trimmed by the miners using a wooden template called a ground mould to check the shape. When the invert and skewbacks were completed other templates called the side moulds were set up to control the side walls. These were built close to the poling boards which were recovered if possible and the space filled with earth. When a bar was reached it could be removed because the brickwork then held the next row of poling boards (see inset to Fig 5). When the lining was at springing level, the centring was placed and succeeding courses rested against the lagging, the line of which thereafter diverged from the line of the timbering. In the crown of the tunnel the space between the two was wide enough for the full thickness of brickwork to be completed underneath the seven or eight crown bars. However, as the work progressed, counter
Figure 6 Drawing the crown bars forward.
Forbes were built in brick between the bars leaving them bricked into long 'cells' (see section of brickwork in Fig 5). When the crown of the length of brick arch had been completed a top heading was driven into the uppermost part of the face. The crown bars were then drawn into this heading and temporarily propped (see Fig 6). Poling and breast boards could be inserted and more of the upper part of the face excavated until the top sill could be put in. The cells from which the bars had been drawn were filled with clay rammed in with a long-handled punner. The top sill was secured against pressure from the face by rakers and 'permanent' props put between it and the crown bars. The new excavation was now safe enough for the side bars and their poling boards to be put in fairly easily and the cycle of building repeated. A 'length' (2.75 m - 3.7 m, 3 or 4 yards) of twin track tunnel usually took about a fortnight to complete (Braithwaite, 1907).

Of the many brick railway tunnels that were to be driven through London Clay by the English method, the two which brought about the greatest changes in the way engineers thought about London Clay were the first, Primrose Hill, at the end of the London To Birmingham Railway, and the deepest at Sydenham on the London, Chatham and Dover Railway.

2.2.2 Primrose Hill and Clerkenwell

The Primrose Hill tunnel 1060 m (1154 yds) long was built between 1836 and 1837. The engineer to the London and Birmingham railway was Robert Stephenson who had served three years as an apprentice in a colliery. He had already completed several railway tunnels in hard rock, but trouble due to running sand in Kilsby tunnel had cost a great deal. After this experience he would have preferred to choose a route into London that avoided tunnelling. However, the landowners and the Grand Junction Canal Company were strongly
opposed to the railway. The route finally chosen was a compromise and included the tunnels at Watford and Primrose Hill.

The tunnels were each lined with a horse-shoe shaped brick arch (Figs 7 and 8). The London Clay in the bottom of the Primrose Hill tunnel heaved and an 'invert arch' of considerable flatter profile than the main arch was added. The clay gave similar trouble in the Camden Hill cutting and was eventually rectified in the same way (Rolt, 1960). No more tunnels were built in swelling clay without an invert.

This trial and error tunnelling although near to London did not damage property because the tunnel was overlain by farmland (see Fig 7). A route through undeveloped land was available because Eton College was vainly expecting to lease their Chalcots Farm estate to a developer who could emulate the standards of Regents Park (Olsen,1973). Part of the tunnel passed under the old Marylebone to Finchley road, now Avenue Road, and the new Marylebone to Finchley Turnpike (now Finchley Road), but the landowner here, Colonel Eyre, was concentrating on high-class development of the southern part of his estate at this time.

After the Chalcots estate had been built it became necessary to duplicate the Primrose Hill tunnel. Presumably with Stephenson's experience in mind, the best methods were used. Nevertheless, W R Galbraith* (1885) commented that the new construction 'did not seem to have been carried out with as little disturbance to buildings as had been achieved in the Underground Railway. From end to end all the buildings were cracked in the neighbourhood of the tunnel and the company must have paid a large amount for compensation.'

* Biographical foot-note on page 80.
Fig. 7 Primrose Hill tunnel, eastern portal, by J.C. Bourne
Galbraith was comparing the tunnelling with that on the Metropolitan and Metropolitan District Railways, which although it is mostly in covered ways, included a number of tunnels driven through the London Clay by the English system. The engineers were Sir John Fowler and Sir Benjamin Baker.

Speaking of the 8.68 m (28 ft 6 in) wide 666 m (728 yds) long Clerkenwell tunnel, built between November 1860 and May 1862, Baker (1885) quoted measurements of the movements at the crown of the tunnel made by his resident engineer, Morton. It was made up as follows:

<table>
<thead>
<tr>
<th>Description</th>
<th>cm</th>
<th>inches</th>
</tr>
</thead>
<tbody>
<tr>
<td>From commencement of the length to getting in the top sill</td>
<td>2</td>
<td>3/4</td>
</tr>
<tr>
<td>From getting in the top sill to completion of timbering</td>
<td>5</td>
<td>2</td>
</tr>
<tr>
<td>Compression of green brickwork</td>
<td>4</td>
<td>1 1/2</td>
</tr>
<tr>
<td>Settlement of arch</td>
<td>3</td>
<td>1 1/4</td>
</tr>
<tr>
<td>Distortion of invert and side wall</td>
<td>6 1/8</td>
<td>1 3/4</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>18 1/8</td>
<td>7 1/4</td>
</tr>
</tbody>
</table>

The cover varied between 25 and 60 ft and the damage to buildings was described as 'slight'. Referring to the parallel running 'Widening tunnel' he observed 'it was driven under the contractor's office in Exmouth Street without any apparent damage to the exterior of the building, and without any apparent damage to the exterior of the building, and with but slight internal indications of settlement. However with the utmost precautions, tunnelling through a town is a risky operation, and settlements may occur years after the completion of the works. Water mains may be broken in the streets and in the houses stone staircases may fall down and other unpleasant symptoms of small earthquakes alarm the occupants'. The long-term effects would be
in part due to Fowler's practice of building in the crown bars, which would later have rotted.

When it is remembered that in some tunnels the crown bars moved down 0.6 m (2 ft) compared with the 7 cms (2 3/4 inches) here and that all the mortar was squeezed from between the bricks which would give a diametrical contraction of one or two feet, this was very good tunnelling by the standards of the day. Indeed, because it was done under the supervision of the finest engineers of their time, and in central London it probably was the best tunnelling by the traditional method.

Four brick-lined tunnels were driven through the London Clay hill on which the Crystal Palace stood at Sydenham. The last and deepest of these was completed in 1863 for the London, Chatham and Dover Railway. The average cover over the crown of the tunnel was 25 m (82 ft), but in parts was as much as 61 m (200 ft).

2.2.3 Sydenham

The engineer was Joseph Cubitt and the contractors Peto and Betts. Their plan was to sink six shafts along the 2010 m (2200 yds) length and connect them by headings driven at crown level. These were to serve as working headings, the usual invert level pilot heading being omitted. In sinking the shafts it was found necessary to timber them very heavily. The settings were framed with timbers 30 cm x 30 cm (12 in x 12 in) placed 75 cm (2 ft 6 in) apart whereas 1.8 m (6 ft) spacing usually sufficed. The poling boards were 7.5 cm (3 in) thick. The clay itself often contained highly polished 'backs' with a throw of several inches.

With these indications of trouble ahead the contractors used 46 cm (18 in)
diameter crown bars spaced at 75 cm (2 ft 6 in) centres, bricked out the invert speedily and placed 35 cm x 35 cm (14 in x 14 in) rakers 7.6 m (25 ft) long, reaching back to special pockets built in the hardened brickwork at this distance from the face. With a view to relieving the load on the green brickwork of the arch, the centres were also made extra strong. Nevertheless, the stout bars bowed and the props sank so that often the crown bars moved down more than the 45-60 cm (1½ to 2 ft) that had been allowed for by raising their leading ends. Sometimes the crown was even close barred.

By these expedients the brickwork could be built satisfactory but often failed after it had been in for a week or so.

Thought was then given to modifying the lining. Only a few yards were built in the original horse-shoe shape with invert section. This is the section 'a' shown in Fig 8, which has five sections a to e for this tunnel, with parts outside the line as cut off. The footings were then widened to the full cross section shown in b (the dotted line is for comparison with other sections); nevertheless length after length failed by coming in at the sides and lifting of the invert. To remedy this the invert was lowered, section c of Fig 8. This section served tolerably well but many failures still occurred. The next experiment was to do away with the broad footings of the side walls and adopt the egg-shaped section d which had the invert the same thickness as the walls and arch. The invert now withstood the pressure but the side walls continued to fail. Eventually after 10,000 m³ (13,000 cu/yds) of brickwork had been broken the nearly circular section e was adopted. This was a complete success and the rest of the tunnel was finished in a year (indeed the whole job only took a little over two years) (Baldwin, 1876).

Twenty-five years seems a long time for the best tunnel to evolve. There
Fig. 8 Evolution of tunnel linings in London

1862-63

J Cubitt

Brick

0 1 2 3 4 5 m
0 10 20 ft

O O C
are many records of broken timbering and brickwork amongst the accounts of tunnel construction during this period, but somehow each problem was overcome. Perhaps this is a tribute to the ordinary miners and bricklayers who were able to make even an unsuitable system work if it were at all possible. It was only when the ground was considerably more difficult than that previously encountered that the engineers were induced to re-think their demonstrably inappropriate designs.

A paradox is presented by the only two canal tunnels in London Clay which appear to have a design ahead of their time. They were both built between 1812 and 1816 to form part of the Regent's Canal which was opened in 1820 to provide a direct link between the London Docks and the Grand Union Canal avoiding the tidal Thames and the crowded Pool.

The Islington tunnel is 880 m (960 yds) long and passes under the ridge of high ground running down to Finsbury. The cover is about 15 m (50 ft). The Iwade tunnel about 4.5 km west is 250 m (272 yds) long and of similar design. They are 5.75 m (18 ft 9 in) high, 3.4 m (11 ft) from water level to the crown, 5.2 m (17 ft) wide at water level, and the brickwork is 36 cm (14 in) thick. The contractors were Fritchard and Hoof of Kings Norton.

James Morgan, the engineer to the canal company, is usually credited with the design of these tunnels. In 1812 he had had no experience of canals, having been assistant to John Nash for 17 years. His new post was due to Nash's desire to provide ornamental water for Regent's Park. The design was probably William Jessop's, since he was one of the engineers who submitted entries to a competition organised by Nash. The prize was not awarded (Sommerson, 1935).
The lining is in the form of an elliptical arch which meets the curve of the invert at a cusp, that is there are no skewbacks between them (Fig 8). This is very similar to one of the improved sections introduced at Sydenham 43 years later.

At first sight it seems that Robert Stephenson could have had no contact with the canal builders when he started the Primrose Hill tunnel in 1836. However, the historical evidence denies this (eg see Rolt, 1960).

It therefore appears that he thought the canal builders' practices were not appropriate to his tunnels. (Also that because of Stephenson's reputation that generation of railway engineers did not question his judgement.) Certainly it must have seemed unlikely to him that the hard London Clay would produce a hydrostatic distribution of pressure or pressures of such magnitude as were to be encountered. The canal and river navigation engineers had on the other hand for centuries had to deal with soft ground heaving upwards at the bottoms of their excavations. The early pound locks were lined and were arched both in horizontal and vertical directions, eg they were boat-shaped. This morphological resemblance between the lock lining and the boat hull must itself have been due to a recognition of the similarity of their structural functions.

2.3 CAST IRON

2.3.1 Origin of circular lining

The circular section would be almost the only one used when tunnels came to be made of cast iron. This material would first be introduced as part of the development of subaqueous tunnelling. Underground water was a nearly insuperable hazard to the early tunnellers and an embarrassment to shaft
sinkers. When sinking deep shafts for collieries it had been the practice to line those parts which passed through aquifers with wooden staves bound with iron hoops to form a watertight cylinder called tubbing because the method of construction was the same as that employed for tubes (Galloway, 1852).

The invention of cast iron tubing was attributed by Galloway to John Dudlle and his father of the same name: 'In the year 1792 a cast iron tub consisting of cylinders the full size of the pit was applied by the elder John Dudlle to dam back a quicksand in the pit A at Wallsend. Similar cylinders were employed by Mr Barnes in the King pit at Walker Colliery about 1795. This form of tubing, however, was only available near the surface, but by dividing the rings into segments it became applicable in any part of the shaft.

Segments of cast iron, having flanges turned inwards for bolting the pieces together, were first used by Mr Dudlle in 1796-7, in sinking the Percy Main pit. An improved form of segment, having flanges turned outwards and without bolts, was introduced by the same gentleman in sinking the Howden pit in 1804-5; an arrangement which has ever since been universally adopted where cast iron tubing is applied under ordinary circumstances'.

2.3.2 Origin of tunnelling shield

Marc Brunel included two kinds of circular iron lining for tunnels in the patent he filed in 1818 which refers to the formation of drifts under the beds of rivers, but neither were actually used by him. Two tunnel shields were also described and one of these, although shown as circular in Marc Brunel's drawings employs the same principle as that used in the shield which the Brunels actually used for the Thames tunnel between 1825 and 1843. This was

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Two versions of this shield were built, both designed by Brunel. The first was made by Holdays and was used between 1825 and 1838. Work was suspended between 1828 and 1835. The second shield, built by Kennicus had refinements making it more suitable for the running ground that had been encountered. In 1835 it was substituted for the first one and was used until the tunnel was completed. The second shield was described and illustrated by Law (1846). Figure 9 is from Law.
Fig. 9 Sir Marc Brunel's Thames Tunnel shield.
built in brickwork of rectangular section 11.5 m x 6.5 m (37 ft 6 in x 22 ft 3 in) and pierced by two archways each 5.2 m (17 ft) high and 4.25 m (14 ft) wide (see Fig 8). The flat section was adopted (Clements, 1970) because borings had shown that the layer of 'strong clue clay' through which the tunnel was to be driven, was not thick enough for a circular tunnel containing two carriage ways. In the event most of the tunnel was driven through very soft material.

Brunel observed that 'It is obvious that the smaller the opening of a drift the easier and the more secure the operation of making the excavation must be'. He concluded from this that in order to make a large tunnel he should split his shield up into a number (36 in the actual Thames tunnel) of small cells each large enough for one man to work in and intended to be forced forward before the timbering which is generally applied to secure the work. The workman thus inclosed and sheltered may work with ease and in perfect security. Each of the ten cells of the proposed circular shield was to move separately; in the rectangular shield actually built, stacks of 3 cells moved as units.

Marc Brunel's shield certainly worked but the contribution of splitting it up as he did must mainly have been to make it possible to advance the shield without using large forces. Since the shield cells were advanced by screw jacks operated by hand, large loads were not available; and because the section had to be taken by green brickwork they could not have been used any way. Control of the face depended on the way the work was done in each of the cells rather than on whether they advanced separately or in a block. Each cell had fifteen poling boards 15 cm (6 in) wide, each held against the face by its own screw jacks, so that although very soft ground was encountered
very little needed to be exposed at a time.

In describing his other proposed shield in 1818 Brunel refers to the wood boring mollusc Teredo navalis. In this species the two valves of the shell do not cover the body but have degenerated to two triangular plates on the front of the head. These plates have an abrasive surface facing forward and can be moved in such a way that the wood in front of it can be rasped to a powder fit to be digested. The animal then secretes a nacreous lining onto the tunnel formed in the wood.

Brunel was impressed by the elegance of continuously depositing a lining in-situ and combining the function of face control and excavation in one pair of full face cutters. One imagines he thought of something like the Haling machine with added provision for placing a shotcrete lining on the ground it had exposed. His 1818 solution was a spiral lining of cast iron plates (anticipating the form of the Bridge system) and a cylindrical shield consisting of a number of 'staves' like a ring of sheet piling which could move parallel to the axis of the cylinder so that at their rearmost ends they fitted the front spiral of the lining. This shield does not appear to have been intended to overlap the lining as was the other circular shield. The forward end of the ring of staves, which were to be advanced by hydraulic rams, met an annulus of plates surrounding a central auger. The plates were hinged so that the annulus could form a shallow helix which Brunel called the greater auger. This greater auger did not, of course, rotate as a whole but the step in the helix moved round as the staves were adjusted. The spoil was to be removed from an opening at the step in the helix.

The idea of a series of interlocking longitudinal staves also embodies in a much later patent by Jennings and Stannard and put into practice in 1890.
when an additional tunnel was required by the Great Northern Railway where this crosses under the Regents Canal at Kings Cross. Their so-called 'needles' were 3.05 m (10 ft) long, 15 cm (6 in) wide, and 5 cm (2 in) deep. They are regarded by Simms as a substitute for the earlier timber crown bars and poling boards. They were only used in the top half of the tunnel, the face and other parts being timbered. The front ends were sharpened and they were driven into the clay by detachable screw jacks which engaged in holes in the needles and thrust off the brickwork. Thus there was no longer any need for a top heading. After the brickwork had been completed under the canopy of needles, they were re-advanced, singly or in threes. The needles were provided with longitudinal tubular cavities through which grout could be pumped as they were driven. Thus no space was left above the brickwork.

This tunnel was an early example of the use of a shallow bored tunnel driven to avoid interference with existing structures (the Regent's Canal, the Great Northern Goods Station, and the old Caledonian Cattle Market) without the necessity for a steep gradient at the portal. The needles only required a 5 cm (2 in) depth above the brickwork whereas timber bars would have required 60 cm (2 ft); thus the tunnel could be completed with only 1.8 m (6 ft) headway (Simms, D. Kinnear Clark, 1896).

2.3.3 Origins of tube railways
The advantages of prefabrication and of watertightness which the use of cast iron could provide in civil engineering structures was being exploited by the second half of the nineteenth century.

In 1851 cast iron cylinders were used as caissons for sinking the piers of Rochester bridge over the Medway by Cubitt and Wright; in 1852 J.R. Brunel
used them for Chepstow bridge over the Wye.

In October 1865 work was started on the Waterloo and Whitehall Railway. Iron tubes 12 ft 9 in diameter were constructed by Samuda Brothers of Poplar for what would have been the world's first immersed tube tunnel. This was to cross the Thames between Great Scotland Yard and York Road (Hadfield, 1966). Repercussions of the Overend and Gurney banking failure resulted in this project being abandoned. Propulsion on this railway was to have been by low pressure air, the train itself acting as the piston in contrast to the more often tried system of a piston moving in a split tube mounted in the track.

In December 1866 the diversion of the Fleet sewer for extension of the Midland Railway to St Pancras was accomplished, at the insistence of the newly formed Metropolitan Board of Works, by means of 15 ft diameter cast iron rings. The engineer was W H Barlow (Barnes, 1966).

However, it was not until 1869 that a bored tunnel in London Clay was lined with cast iron. This was the Tower Subway, which was driven under the Thames between Tower Hill and Vine Street (now Vine Lane) off Tooley Street.

The project was sponsored and engineered by Peter Williams Barlow, elder son of Professor Peter Barlow mathematician and scientist and brother of William Henry Barlow consulting engineer to the Midland Railway. P W Barlow had been principal engineer to the South Eastern Railway. In 1845 he had read a paper to the Institution of Civil Engineers comparing atmospheric traction of the split tube type with rope haulage. The latter method had been used by George Stephenson on the Camden incline from Euston in 1836 and the two methods were considered by Pearson for the Metropolitan Railway which had
opened in 1863. However, because little power was needed the use of small steam locomotives underground was not at first considered objectionable. Now with increasing traffic and more powerful locomotives the ventilation was proving inadequate. Siemens was not to exhibit his electric locomotive until 1879.

P W Barlow believed that the steam locomotive was out of place underground. If the prime mover was separated from the vehicle, an underground railway could be designed with freedom to concentrate on the real problem of metropolitan transport. The railway or tramway could run through deep tunnels which ignored the surface feature including rivers and buildings. The system which Marc Brunel had suggested in 1818 was ideal for such a 'subway' and by 1868 its various components had already been separately tested. Powered lifts were available to take the passengers to and from the deep tunnels.

Barlow planned a subway following a semi-circular route between Tottenham Court Road and the Elephant and Castle with a river crossing downstream of the bridges. The river crossing would be the trial length for a number of reasons: (1) this was the part that the lay shareholders were most anxious about, yet Barlow knew that by going deep, and with the tunnelling shield, it would cause no more engineering difficulties than any other section; (2) a new river crossing could be expected to pay its way as a separate economic unit even before the rest of the railway was built; and (3) little private property needed to be bought for tunnelling rights.

That Barlow was an engineer-planner rather than an innovator of technical ideas is indicated by the title of his patent specification of 1864:

'Improvements in constructing and working railways and in constructing railway
tunnels' and of his paper of 1867 'On the relief of London street traffic with a description of the Tower Subway'. By bringing together a dozen or so ideas which had previously only been used independently or had been published but not tried, and some innovations of his own, Barlow created a system which was to be the basis of a revolution in urban transport (see Table p 31).

In so far as the patent of 1864 deals with the actual construction of a shield the single novelty is that it should move forward in one piece. It is divided into 44 compartments and a cross section is shown with a ring of 12 segments each circumferential flange of which has six bolt holes. From this one might guess that the tunnel diameter was intended to be 25 or 30 ft. However, the longitudinal section of the tunnel shows a lining of six segments and no platforms from which the screw jacks might be operated.

The 1864 patent stresses the potential of the shield method 'particularly where the tunnels are under rivers or under towns and places where the upper surface cannot without serious injury be broken up or interfered with'. He also stated in the patent that: 'It is desirable that the thickness of the iron of the cylinder (tail plate) should be as little as may be in order that the space between the outer surfaces of the rings and the earth which surround them may not produce any subsidence in the land above', and 'The space as it is left between the earth and the exterior of the tunnel may be filled by injecting or running in fluid cement'. This appears almost as an afterthought, but grouting was the very innovation which was to make submetropolitan tunnelling acceptable.

The second patent was taken out in 1868, the same year that the Tower Subway Company was formed. The shield illustrated appears to be smaller. The
Ideas first used interdependently for Tower Subway and subsequently contributing to the success of the 'Underground'

### Tunneling

**Barlow originated**
- One piece shield
- Grouting behind lining
- Water trap or safety curtain
- Front of shield larger than rear (may have been Greathead)

**Others originated**
- Iron lining
- Circular section tunnel
- Shield overlapping lining
- Screw advance of shield

### Underground Railways

**Barlow originated**
- Small vehicles
- Deep tunnels in London Clay
- Upgrades towards stations, downgrades away

**Others originated**
- Cable haulage on inclines
- Vertical cable haulage (lifts)
The longitudinal section shows a lining like that actually used in the Tower Subway. The cross-section shows a lining of six segments. This specification refers to an important innovation: 'A transverse partition or end having through or below its centre and opening which an be either partially or entirely closed as required. ....... By this arrangement should the water at any time break into the tunnel the upper portion of the interior of the tunnel may at all times be kept supplied with air under pressure as the closed end will prevent the air from escaping' (Barlow, 1868).

The bulkhead illustrated in the specification would perform this function, but the one used in the shield actually built would not have been so effective had water been encountered.

2.3.4 Tower Subway

Because the eighteen year long struggle that Brunel had endured under the Thames was remembered, Barlow was unable to find a contractor willing to drive the new tunnel. He put his assistant, J H Greathead, in charge of the work. A small shield 1.45 m (4 ft 9 in) long, 2.2 m (7 ft 3 in) diameter at the tail and flaring out slightly forward was designed by Greathead (according to Copperthwaite, 1905) to the ideas embodied in Barlow's patents of 1864 and 1868. It was advanced by six screw jacks which reacted against the last built ring of lining. The ring was built within a tail plate forming the rearmost part of the shield, and after the shield had advanced a hand syringe was used to inject cement grout into the space behind the lining.

The outside diameter of the cast iron lining is 2.18 m (7 ft 1\(\frac{3}{4}\) in). Each ring is 46 cm (18 inches) wide and consists of three long segments and a key segment. The segments are of the new familiar U-section\(^*\) with flanges

\(^*\)The reader who is not familiar with cast iron tunnel linings or the nomenclature of their parts may find the diagrams between pages 137 and 138 helpful.
running circumferentially which are 29 mm (7/8 inch) thick and 7.6 cm (3 in) deep so that the internal diameter of the ring is 2.02 m (6 ft 7 3/4 in). The skin or outermost part between the flanges is 29 mm (7/8 inch) thick. At the centre of each long segment there is a rib running parallel to the axis of the tunnel and connecting the two circumferential flanges. This suggests that the circumferential flange had to withstand the reaction from the shield screw jacks (see later when the lining of the City and South London Railway is discussed). The cross flanges are deeper than the others, being 10 cm (4 in) deep and 2.4 cm (15/16 inch) thick. Each has two holes by which neighbouring segments in a ring were bolted together. The joint between adjacent rings was made by 1.9 cm (3/4 inch) bolts at 48 cm (19 in) centres. In order to retain the cement grout a rope was placed behind these bolts before the joint was made. It is probable that the cross joints were made with metal to metal contact.

The 410 m (1350 ft) long tunnel was completed in less than a year, a maximum advance of 2.75 m (9 ft) was made in a 24-hour day worked in three shifts of 8 hours (Greathead, 1895). For a small tunnel and solid London Clay, Brunel's original idea of using a circular shield with a cast iron lining had now been shown to be a fast and safe way of tunnelling.

The shafts, 16.3 m (60 ft) deep at Tower Hill and 15.3 m (50 ft) deep at Vine Street are 3.1 m (10 ft) in diameter. In 1869 they contained steam driven lifts. One cable hauled a car running on a 0.93 m (2 ft 6 in) tram track conveyed the passengers between the shafts. The deepest point was at the centre of the tunnel so that acceleration of the car was assisted by gravity. The total journey time was 3 minutes.
Although the Tower Subway was an engineering success and the potential speed and cleanliness of the method of transport had been demonstrated, the project foundered on the lack of capacity and comfort of the car. It was according to the Illustrated London News: 

'a light iron omnibus 10½ ft long, 5 ft 3 in wide and 5 ft 11 in high. This will accommodate fourteen people with the most perfect ease'. After a few months the lifts and tramway were removed and the tunnel was used as a footway until 1894 when Tower Bridge was opened and the London Hydraulic Power Company bought the tunnel.

Clearly the next step was to try again with large enough vehicles in a large enough tunnel. In 1870 he obtained an Act for the Southwark and City Subway and a company was formed in 1871. Crawford Barlow (in Greathead, 1895, discussion p 77) said that this company's prospectus stated that the tunnels would be 50 per cent larger than that of the Tower subway and that the route would be practically the same as that later to be taken by the City and South London Railway. The company however could not be financed and in 1873 an Act of abandonment was obtained.

2.3.5 City and South London Railway

In 1884 J. H Greathead was appointed engineer to a new railway company. Its consultants were Sir Benjamin Baker and Sir John Fowler (of the Metropolitan Railway). It was first known as the City of London and Southwark Subway Company, but later became the City and South London Railway Company.

The new line was to have twin iron tunnels bored by shield at a deep level and incorporating most of P. H. Barlow's ideas such as upgrades to the stations, cable traction and lifts.
The termini were to be in King William Street near the Monument north of the Thames and at the Elephant and Castle south of the river. Before the railway was opened this so-called 'City section' was extended southward to Stockwell and electric locomotives had replaced cable haulage.

The route ran under the streets to avoid the purchase of property or easements. The one easement that was obtained was purchased for £3000 before the Act was applied for, the length of railway involved was about 45 m (50 yards). The City of London and Southwark Subway Act of 1884 contained a clause that 'the company may enter on, take, and use the subsoil'. This did not give the company the right to tunnel anywhere because any owner of property or land affected could bring an injunction to circumvent it or make a claim for the use of land or damage to buildings. However, it is difficult for the offended party to prove entitlement to land forming part of a long-established street, so that in fact no claims for rights to subsoil were made. Nevertheless, the Metropolitan Railway when tunnelling along the Marylebone Road had had to recompense some occupiers of adjoining property.

The penalties that had to be paid for this stratagem were curves as sharp as those in streets laid out for horse-drawn traffic (eg under Arthur Street near the City terminus) and steep gradients where the twin tunnels changed from being side by side to being one over the other under very narrow streets. Such a street was Swan Lane leading down to the river from Arthur Street, between these it was necessary to have a rising gradient of 1 in 40 in one tunnel and a fall of 1 in 14 in the other.

\[\text{Nothing in this Act contained nor any dealing with lands in pursuance of this Act shall relieve the company from liability from compensation under the 66th section of the Lands Clauses Consolidation Act 1845 or under any other enactment!}\]
The lining used on the City section was 3.13 m (10 ft 2 ins) internal diameter and each ring consisted of six long segments and a key piece. The four lower segments were identical, with radial cross joints. The two 'top' segments had radial cross joints where they met the other long segments at shoulder level; but at the crown end the angle between the cross flange and the skin was less than 90°. This enabled each top segment to mate with the parallel sides of the short key segment. The lining of the extension to Stockwell was 3.2 m (10 ft 6 ins) internal diameter. The lengths of the smaller ring was 48 cm (19 inches) and of the larger 51 cm (20 inches). Both had flanges of 11.4 cm (4 1/2 ins) total depth. The skin was 2.5 cm (1 inch) thick on the City section, but a little thinner on the extension.

The cross joint was made with three bolts. The circumferential joint between successive rings was made with 47 bolts at 21 cm (8 1/8 ins) centres. By keeping the centres of the holes the same around the circumference the adjacent ring could be rolled so that the cross joints were not continuous and a stiffer tube could be made where this was thought necessary, i.e. the rings could 'break joint'. The 'top' segments had seven holes in each circumferential flange whereas the 'ordinary' segments had eight. This facilitated their selection in the dark. This arrangement of joints and bolts has persisted in cast iron linings until the present day.

The manner of sealing the circumferential joints against external water pressure is shown in Fig 10 taken from Greathed's (1895) paper. It was the practice when fitting castings together in the early days of mechanical engineering to provide raised faces called 'chipping faces' or 'chipping edges' at the parts where a good metal to metal fit was required. Thus
Fig. 10. Joints of cast iron tunnel linings, half full size
only the raised part needed to be finished by the laborious process of chipping the iron smooth, instead of finishing the whole surface.

Greathead states that an initial seal was provided by a rope of tarred hemp placed between this chipping edge and the bolts. Subsequently space between the rope and the tips of the flanges were pointed with Medina cement. That the circumferential joints of the Tower subway were finished in the same way is confirmed by Harding (1945) who examined the section of the subway lining which was collapsed by enemy action in December 1940 and subsequently rebuilt by Harding in larger iron.

This method of construction is suitable for straight lengths of tunnel but not for curves. Part of the curve under Arthur Street was of only 43 m (140 ft) radius. Greathead said (1895 discussion p 109) 'No special castings were used in the City and South London tunnels in driving round curves, the difference in length between the inner and outer circumferences being made up by filling in the joints'. One of Greathead's engineers, Mr David Hay, stated (1895 correspondence, p 119) 'No special rings of cast iron lining had been used for the curves; consequently iron packings had to be placed between the rings to make the joints radiate from the centre point of the curve'.

Iron packings placed between the chipping edges of adjacent rings would transmit the shield thrust without bending the flanges. Greathead also designed his shield ram shoes to thrust of the edge of the lining rather than the flanges (Greathead, 1895, p 59).

Medina cement was a pozzolanic cement produced by firing the argillaceous limestone which occurs as 'claystones' in the London Clay. Specially the term applied to cement from those claystones dredged from the Solent.
Thus, although Greathead was not using the chipping faces in the manner in which they were conventionally used before the invention of the power planer by Whitworth in 1842, he nevertheless retained it in his castings for three good reasons:

1. to provide a location for the circumferential hemp seal;
2. to avoid bending the flanges by the shield thrust; and
3. to keep the joint open for pointing.

The author believes that the cross joints on the Tower subway had chipping edges, and that the extra deep joint was filled with iron cement. It is difficult to be sure of this without taking a segment out\(^*\) because the iron is rusty particularly at the headwalls which are the best viewing points (see Fig 11a). Three segments with these cross joints would not form an unstable ring, but more would. Hence Greathead would have had to re-design the cross joints for his ring of six segments. In order to employ the full depth of the as-cast iron flanges, he cushioned them with wood packing. This would still allow some angular movement at the joints, hence the provision for break-jointing the rings. Another advantage of the wood packing of the cross joints may have been that tolerances on the circumferential dimensions of the segments would not have been good. Castings shrink by varying amounts if metallurgical control is poor. Sometimes the wood packings had to be left out of the key joints (Curry and Jones, 1927, discussion p 239).

Two plain holes 28.5 mm (1\(1/8\) in.) diameter were cast in each segment for the injection of grout behind the lining. Greathead was dissatisfied with

\(^*\)The repair to the Tower subway had to be done quickly so that there was little time to use this opportunity to study the structure. Also ferrous metal was in short supply so that no iron was kept.
Fig. 11a  Tower Subway showing cross joints of lining with probable filling of rust cement
Fig. 11b  Shaft tubbing at Stockwell station showing fillets on both joint faces.
the hand syringe used for grouting the Tower subway because the lime grout had to be so thin for the syringe that it was not strong enough when placed and when set. Also the pressure attainable did not ensure that all the voids were adequately filled. For the new railway he invented the now well-known Greathead grouting pan which could supply thick grout at 63 kN/m² (80 lb/in²). He experimented with Portland and Medina cements as well as blue lias lime but chose the latter because it was cheap, expanded on setting and because he recognised that 'there is no object in having a shell harder than solid London Clay'.

The several shields used on the City and South London Railway were similar to that illustrated in Fig 12. This one consisted of a cylinder 1.78 m (5 ft 11 ins) long made of steel plates. This was bolted to a strong ring of cast iron at the front end which carried adjustable steel cutters. These could be arranged so that they cut a hole large enough to clear the steel cylinder following them, or the projection of this cylinder if a sharp curve was being driven. To the rear of the ring was bolted the diaphragm or 'face'. Iron castings on the edge of the diaphragm spread the load from the six shield jacks. There were hand pumped hydraulic jacks. To their rams 'were attached long shoes for carrying the pressure on to the solid part of the cast iron tunnel lining without bringing any bending strains upon the rams or undue pressure on the tunnel flanges' (Greathead, 1895, p 59).

The centre of the diaphragm was provided with a rectangular opening which carried channels at its sides into which timbers could be dropped if water bearing gravel was met.

Greathead had not encountered any buried channels while driving the Tower subway although the possibility of dealing with this hazard by the use of
Fig. 12 Greathead shield used for City and South London Railway tunnels
compressed air was envisaged in Barlow's company prospectus. Early in the construction of the City and South London Railway while driving northwards from the Thames at Old Swan Pier shaft he ran out of the London Clay after 55 m (60 yards). A large volume of water flowed from the exposed gravel. However, the door which had been provided was quickly closed with the timbers kept nearby for this contingency. A brick bulkhead and an airlock were then built in the tunnel behind the shield and the next 45.5 m (50 yds) were driven in compressed air.

That he had considered Barlow's (1868) curtain or water trap unnecessary is borne out by the fact that he had already (Greathead, 1874, and 1875, p 66) designed and built a shield with both a water trap and an airlock built into it. This was intended for, but not used in, the chalk under the Thames at Woolwich. In 1889 such a shield was used under the Mersey.

During the construction of the rest of the line other loose water bearing gravels were traversed in a similar manner. The largest section, under the Clapham Road near its intersection with the present Stockwell Park Road, was 183 m (200 yds) wide and under a head of 11 m (35 ft) of water. This is probably a channel associated with the river Effra which is thought to have flowed along the line of the Brixton Road 400 m (1/4 mile) to the east. When the Victoria Line was built a larger and more open type of shield was driven into the same channel, not without incident.

In water bearing ground the Medina cement in the circumferential joints of the linings was replaced by rust cement, iron filings with 0.25 per cent of ammonium chloride added.
The method of working the shield in the clay was first to drive a small timbered heading for about 1.8 m (6 ft) ahead of the face and then to advance the shield. This rather curious practice seems to have been adopted because there was so little room ahead of the diaphragm in which the miners could work. The contractors wanted to use the shield jacking forces to attack the clay but Greathead did not want to risk distorting the shield.

A compromise method of working was adopted by attaching a number of 'wedges or piles' about 0.6 m (2 ft) long to the cutting edge of the shield. When the heading was far enough ahead the rearmost timbers were loosened for the length of one ring. When the shield was advanced the clay between the heading and the ring of wedges was broken off. Greathead claimed that this avoided damaging the shield when meeting bands of claystones and that two shifts could gain 4 m (13 ft 4 ins) or eight rings per day of 24 hours with regularity (Greathead, 1895). One suspects that this method of working the shield would cause an unnecessarily large subsidence.

Large iron linings called tubbing* were used for the liftshafts and stairshafts. The former which were also used as working shafts are 7.7 m (25 ft) internal diameter, the rings are 1.2 m (4 ft) long and consisted of twelve identical segments with no key. The tubbings for the stairway shafts are 4.6 m (15 ft) diameter and also 1.2 m (4 ft) long, also consisting of twelve identical segments. They have a half inch deep chipping face on all four edges (Fig 11b). The rings were built on the surface and the 2.5 cm (1 inch) wide joint was caulked in the dry. The bottom ring is 91 cm (2 ft 6 ins) deep and instead of a lower flange has the skin thickness somewhat increased.

*Possibly they were made from the same patterns as colliery shaft tubbings. Most of them bear no marking, but one was found, above old brickwork, marked 'Stanton'. This company, first formed in 1846, assumed this name in 1877 (Lewis 1959). Stanton & Staveley could offer no information.
to form a cutting edge. A shaft was sunk as far as the clay by weighing it with spare segments as kentledge. Once in the clay the lower part of the shaft was completed in brickwork.

The station tunnels were of brick, those at the intermediate stations being 6.1 m (20 ft) wide, 4.9 m (16 ft) high and 0.9 m (3 ft) thick of 'Christmas pudding' section. They were all in London Clay and constructed by the traditional English method in 2.75 m (9 ft) lengths. Greathead states that some disturbance of the surface was caused by these brick tunnels but not by the iron running tunnels. Since the station tunnels were all under streets paved with small stone setts any disturbance would have to have been severe in order to be noticed. Similarly, the fact that the subsidence due to the running tunnels was not noticed in the roads does not mean that buildings would not have been damaged had the tunnels run directly under them, or indeed, were not damaged by the tunnels under the roads.

2.3.6 Blackwall and Greenwich

Following Greathead's successfully traversing the water bearing gravels in the London Clay, attention was again turned to the possibilities of crossing the Thames further downstream where it has cut deeply into the London Clay. This would lead to more waterproof linings which in turn would influence the design of the linings used in the clay.

Two tunnels were driven through the Thames gravels within a few years, both designed by A R Binnie (later Sir Alexander) for the newly formed London County Council, Blackwall between 1891 and 1896 with Sir Benjamin Baker and J H Greathead as consultants, and Greenwich in 1899.
The Blackwall tunnel of which 0.95 km (47 chains 14 ft) were driven in compressed air and lined with cast iron segments consisted of a number of straight lengths with changes of direction in both line and level at intermediate shafts. The resulting sharp bends have now been radiused off to accommodate the less manoeuvrable vehicles now using the tunnel.

The shield, 8.44 m (27 ft 8 ins) in diameter and 5.94 m (19 ft 6 ins) long, was designed to Brunel's model rather than Barlow's. It had 28 shield rams, two airtight diaphragms and the face was controlled by a number of hinged shutters worked by screw jacks. The working compartment was divided into a number of cells on three levels. It was designed by W H Koir on behalf of Sir Wooton Pearson, the contractor.

The cast iron segments each of which weigh over a ton were erected with the aid of a hydraulic erecting arm. For the shallow parts of the tunnel a somewhat lighter lining was used. The heavier lining (Fig 8) has an internal diameter of 7.62 m (25 ft) and an external diameter of 8.23 m (27 ft). Each 0.76 m (2 ft 6 ins) long ring consists of 14 segments and a solid key 15 cm (6 ins) wide. The skin is 5 cm (2 ins) thick and the flanges taper from 76 mm (3 ins) to 57 mm (2 1/4 ins). The grout holes were tapped and had screw plugs. The segments were made by the British Hydraulic Foundry Company of Glasgow (Haw and Fitzmaurice, 1897).

In both cross joints and circumferential joints the bolts were arranged in two rows, in the cross joints this contributed to the stiffness of the large and heavy rings. The deflection from a true circle of a ring in the tail plate of the shield was 12.7 cm (5 ins) or 1.5 per cent of the diameter.

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"Later increased to 34 rams."
All four joint surfaces of each segment were planed. Cast into the tip of the flange was a rebate 5 cm (2 ins) wide and about 8 mm (5/16 in.) below the surface to be planed. When the joint was made and the planed surfaces mated the two rebates formed a rough-surfaced groove 5 cm (2 ins) deep and about 12 mm (½ in.) wide in the same plane as the joint and open to the inside of the tunnel. This was called the 'caulking groove' and filled with rust cement as soon as possible in order to conserve compressed air. In the Greenwich Footway tunnel the bottom of the groove was first caulked with lead wire which much enhanced the chances of the cement setting in a watertight condition. Also at Greenwich the planed surfaces were smeared with red lead (Copperthwaite, 1902, 1905).

Of the 11 km (12000 yds) of caulked joints in the Greenwich tunnel only twelve points needed making good after the air pressure was reduced; the leak rate is only 1091 litres/day (240 gallons) or 0.10 litres/metre/day, or 0.16 pints per yard/day, compared with 0.29 litres/metre/day (0.47 pints per yard per day) at Blackwall (from quantities measured in 1904 - 3500 gallon/day over 60,000 yards of joint).

Summarising: these impressive results were obtained because:

(1) the tunnel was designed in straight lengths;

(2) the segments were accurately machined to build these straight lengths;

(3) the shield could be steered accurately to make a hole that was also straight.

2.3.7 The Underground, Londoners and Parliament

A great many more deep underground railways were to be built on the pattern of the City and South London railway. Although ultimately this was due to
needs of the people of London it was despite rather than because of the public representations of the LCC that the London Underground system developed. The cut and cover construction of most of the Metropolitan and Metropolitan District Railways had caused considerable disruption and aroused stiff resistance from property owners and leaseholders and parish councils. At the same time the streets were becoming more congested and by the 1870's it was becoming both politically and logistically impractical to use the cut and cover method in London. Other engineering solutions were sometimes presented (overhead, covered ways, immersed tube, and even the shield method) but the schemes failed, usually because the Bill failed, but sometimes because the opposition resulted in clauses being included in the Act which created engineering difficulties; if these were solved expensively money would be lost, if they were solved ingeniously investors became cautious and the money was not available. By no means did this difficulty disappear once the first successful railway was eventually built. That the local problem of London's railways was dealt with in a rather cumbersome way by our national Parliament under Private Bill Procedure was viewed with disfavour by progressive local politicians, complacency or even comfort by reactionary ones and as a gamble by the entrepreneurs.

Success in winning Parliamentary approval for the first tube railway was due in part to a judicious choice of route, but also to Greathead's effectiveness when giving evidence to the Committee. Briefed in this by Baker and Fowler and with a natural flair for exposition of technical matters to a lay audience, his first-hand knowledge of the shield method could be put to full

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*Resulting in 1902, by dint of pressure from the LCC, in the Royal Commission on London Traffic under the Chairmanship of Sir David Miller Barbour. On 17 July 1905 it produced an eight volume report containing much sound and well intentioned recommendations. Unfortunately the practical effect was that for three years Parliament was reluctant to pass any London railway Bills. This accounts for the period between 1901 and 1904 when only one line was built and that between 1906 and 1908 when nine schemes were completed. By now the competition of the motorbus was beginning to be felt.*
use in assuaging doubts as to the engineering feasibility of the scheme
and its potentiality for minimum disruption of everyday activities. He
overcame the City's ban on cartage of spoil through the narrow crowded
streets near the Thames by sinking working shafts in the river itself.
From these the clay was taken down river in barges.

Whereas other schemes, including Barlow's, having been enacted failed through
lack of confidence of potential investors. Here the City and South London
was fortunate in having a chairman, C G Mott; he himself provided much of
the capital and with it, of course, financial confidence.

Although its civil, electrical and mechanical engineering were satisfactory and
it was soon carrying large number of passengers, quickly and cheaply, daily
to and from the overcrowded City, it did not pay its promoters large
dividends. This was foreseen by promoters who applied for an Act for a
new deep underground railway from the Bayswater Road to the City end of the
City and South London Railway. The revenue from this route would have been
much higher because there would have been travellers throughout the day.
The Bill was vigorously opposed especially by the Metropolitan Railway
because it cut across the Inner Circle and Parliament had virtually promised that no railway would be built within the Inner Circle. From the point of
view of local and national government it would do little to relieve over-
crowding in the City. The Bill was passed through the Commons in May 1890
but was rejected by the Lords. It reappeared for the 1891 session as the
Central London Railway running from Shepherds Bush to Cornhill, and with a

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The 1863 Select Committee of the House of Lords on Metropolitan Communications
endorsed proposals of the 1846 'Royal Commission on Railway Terminals within
or in the immediate vicinity of the Metropolis' to keep railway out of the
centre. In 1864, a Joint Select of both Houses of Parliament approved the
route of the Inner Circle.
commitment to run three trains each way, morning and evening at a 1d fare for the whole distance (16 miles). This version gained the Royal Assent on 5 August 1891.

This Parliamentary volte face encouraged a number of proposals for the 1892 session so that the Commons instructed that a joint Select Committee of both Houses of Parliament should consider the best way of dealing with these schemes.

Six schemes were considered, two were for alterations or extension of existing powers (Central London Railway and City and South London Railway), three for underground routes connected geographically and administratively with existing surface railways (Great Northern and City Railway (Finsbury Park to Moorgate), Waterloo and City Railway (Waterloo to Bank) and the Baker Street and Waterloo Railway); and one newcomer - a Hampstead, St Pancras and Charing Cross Railway.

All were regarded as useful by the committee and were approved and built within a few years, including the last which in a slightly altered form became the Charing Cross, Euston and Hampstead Railway. Not only did the Committee state 'They are required to relieve the overgrown passenger traffic along the chief thoroughfares, to provide for the natural expansion of London, and to check the congestion of our metropolitan population by facilitating cheap communication outwards to a circumference which tends constantly to recede' but also added 'More such lines of communication are required with existing suburbs; and there is growing need of their extension still further into the country'. (Report of Select Committee, paragraphs 8 and 9.)
Amongst the witnesses were representatives of the City of London, the London County Council and, of course, J H Greathead who was concerned with all six schemes. The LCC wished the size of the tunnels to be sufficient to allow interchange of traffic with existing railways. It also said that 'the underground lines should not follow existing streets but go from point to point, the depth below the surface being such as to avoid injury or inconvenience to buildings in the line of the railways' (Cripps, II l in evidence p 393). Lord Portman's agent spoke against this proposal.

Greathead strongly opposed the LCC's first proposal on the grounds that main line stock would require large tunnels because of its side-hung doors and this virtually wasted space would be an unnecessary expense for a purely local underground railway.

The Committee accepted Greathead's view, except in the case of the Great Northern and City Railway whose Bill proposed a diameter of 4.9 m (16 ft), they hoped it would lead to a trans-Metropolitan main line rail link (the 'Widened Lines' route involved sharp bends and steep gradients). A standard diameter of 3.36 m (11 ft 6 ins) was recommended (Report, paragraph 16).

With regard to routes the Committee was unable to do anything better than endorse Greathead's approach as embodied in the two previous Acts and the six Bills before the Committee. It reported that it was 'desirable that companies should be allowed to acquire a way-leave, instead of purchasing the freehold of the land, subject to the terms of the Lands Clauses Acts as to compensation' (Report, paragraph 14).

'In the case of public streets the Committee think it expedient that the companies should be empowered to pass under the streets at sufficient depth
without compensation for the way-leave. In consideration of such free passage the Committee advise that the companies should be put under obligation to furnish an adequate number of cheap and convenient trains!.
(Report, paragraph 15.)

Consequently it continued to be expensive to tunnel under private property and underground railways continued to be built with sharp bends (as little as 61 m (200 ft) radius).

Only as the next tube boom of 1906-7 was ending and the property owners were becoming sufficiently confident that the construction and operation of underground railways could be carried on without causing nuisance did they start to allow way-leaves at reasonable charges.

2.3.8 The Waterloo and City Railway

The Waterloo and City railway was instigated by the London and South-western Railway to link their main line terminus with the City. It remains today in much the same form that it was built, two stations connected by 3.5 km (1 mile 46 chains) of twin tunnel. The engineers to the subsidiary railway company were J H Greathead, W R Galbraith and R F Church. A young engineer from the drawing office of the LSWR, R H Dalrymple-Hay was appointed as resident engineer. Tunnelling began in 1894 and the line was opened in July 1898.

A long section is under Queen Victoria Street including the City station (now Bank Station) and its associated large cross-over tunnel. The Waterloo terminus is under LSWR property and the river is crossed diagonally. To connect these sections required some curves of 100 m (5 chains) radius.
The nominal internal diameter chosen for the running tunnel was 3.66 m (12 ft). An allowance for 'wriggle' (for consecutive rings not being built truly coaxially) of 44 mm (1 3/4 inch) brought this diameter up to 3.70 m (12 ft 1 3/4 ins) and with an outside diameter of 3.96 m (13 ft 0 ins) gave a full depth of flange of 130 mm (5 1/8 ins). When the tunnels were completed the space between the flanges was filled with concrete in the expectation of deadening the noise. It did not prove very effective. On the five chain curves, to allow clearance for the centre throw of the cars, rings with an internal diameter of 3.88 m (12 ft 9 ins) were used. The rings of cast iron lining were 0.508 m (20 ins) long and made up of seven segments and a key. Unusually the top segments had the same number of bolt holes as the ordinary segments, eight; making 57 bolts between each ring. These were 25 mm (1 inch) diameter and fitted through slotted holes elongated circumferentially. This may have been because Greathead and Galbraith disagreed about the cross joints. Greathead wished to use the wood packing that had been used on the City and South London lining while Galbraith wanted the planed joints that Sir Alexander Binnie had used in the Blackwall tunnel (Dalrymple-Hay, 1899, discussion p 155). Slotted holes would have allowed the same castings to be used whether the cross joints were planed or packed. In the event they were planed across the whole depth for use in clay (see Fig 10). A caulking groove was machined as well if the segments were to be used in gravel.

The circumferential joints were not machined, every joint had creosoted yellow pine packings. The thickness of the packing was varied to allow for curves in the tunnel and for inaccuracies in driving the tunnel. It appears that the shield was not steered as accurately as had been the hand jacked shields used with an iron packed lining. The packings came to within 20 mm (3/4 inch) of the flange tips, the remaining space was pointed with cement.
If the tunnels were dry. If they were wet the circumferential joint had a 19 mm (3/4 inch) diameter rubber card placed between the bolts and the chipping face and the rest of the joint was made up with rust cement. This tended to cause the tunnel to incline downwards; this difficulty was overcome by first building the tunnel with hardwood packing and then, before the air pressure was reduced, cutting out the packing and caulking with rust cement.

These changes so far from improving the system worked out by Barlow and Greathead were mostly retrogressive. Unfortunately they have survived until today except that asbestos cement caulking compound has been substituted for rust cement. Two reasons for this are the early death of Greathead and the considerable influence of Dalrymple-Hay*, who was engaged in the design and construction of the Bakerloo, the Hampstead and the Piccadilly lines. In 1902 he became consultant to the London Underground Electric Railway and later to its successor the London Passenger Transport Board.

Dalrymple-Hay (1899, discussion p 156) believed that wood packing in the cross joints 'was one of the principal causes that tended to produce settlements' also that 'his experience extending over some four to five years showed that the face of the clay was absolutely inert, it did not move'. His agreement with the desirability of a caulking groove in place of the wood in the cross joint but acceptance of several layers of wood in the circumferential joint for a lining in water bearing ground is equally odd.

A summary of the effects of the changes made at this time are:

(A) Bad effects due to changes to circumferential joint.

* Knighted in 1933
(i) Shield thrust no longer carried through chipping edges and skin; it is now able to bend the circumferential flanges.

(ii) Steering of shield prejudiced by re-acting against layers of springy wooden packing.

(iii) Packing has to be removed to caulk.

(iv) Caulking only held by narrowing of gap between rings due to chipping edge at back.

(v) Necessity for 'block joint' in caulking at corner to bring caulking at chipping edge level up to level of caulking groove in cross joint.

(B) Gains due to changes in circumferential joint.

(i) Curves in tunnel made without special segments.

(ii) Bad steering of shield can be corrected.

(C) Bad effects due to change to cross joint.

(i) Bending moments in segments due to iron to iron edge contacts (see Section 3.3.5 page 152).

(D) Gains due to change of cross joint.

(i) Circumferential dimensions of ring determined at foundry.

(E) Gains not achieved by change of cross joint.

(i) Easier caulking.

(ii) Stiffer ring. This may or may not be desirable. To achieve it the cross flange would have to be stiffened as well as planed and the bolting arrangement would need to be changed.
The shields used for most of the work were of the same design used for the City and South London railway; the dimensions were a total length of 2.14 m (7 ft) and an outside diameter of 4.05 m (13 ft 2 ins) for the 3.72 m (12 ft 13/4 ins) tunnel. They were also worked in the same way with a square heading driven ahead of the face in the clay and a close poled round heading used with compressed air in the ballast. The diameter of this was greater than the shield which was in fact driven through it. The gravel outside the poling boards was grouted. Greathead adopted this procedure on the City and South London Railway because he did not want the gravel to collapse onto the lining when the shield was advanced, thus causing settlement at the surface. Dalrymple-Hay had been using this technique for some months before he reached York Road where the route passed under the LCC sewer and some buildings. Fearing that the poling boards left in the ground would rot and cause subsidence later he contrived a new way of tunnelling through gravel.

Under-river tunnels such as the Blackwall tunnels had been driven through ballast, without a heading ahead of the face, allowing the ballast to fall back on the lining; but the thrust necessary to push the cutting edge into the ballast was very high. The 28 20.7 cm (8 ins) diameter runs of the Blackwall tunnel shield developing 51,500 kN (5,165 tons) thrust had had to be augmented by the addition of six more. This accounts for the very substantial lining used for the Greenwich footway tunnel. Since the shield and lining for the Waterloo and City had already been made, Dalrymple-Hay's problem was to adapt the shield and find a method of working that would preclude subsidence yet require no more shield thrust than that normally used for clay. This very successful solution is known as the 'hooded shield and clay-pocketing method'. Shields made for working in London Clay nowadays usually have this hood.
The hood fitted to the Waterloo and City tunnel shield consisted of a part cylinder of rivetted steel plates 6.35 cm (2½ ins) thick extending 0.61 m (2 ft) forward of the original cutting edge and itself acting as a cutting edge. The bottom sixth or so of this extension was cut back to the original cutting edge. It thus formed a chamber under the cover of which the miners could work in safety. Without the hood the space between the diaphragm and the cutting edge was only 38 cm (15 ins) long. Because the lowest part is not extended the ballast can be entirely removed from the bottom part of the invert so that the shield could advance easily without fear of being damaged by hidden boulders or claystones. Radial pressures on the hood are transmitted by a number of webs onto a ring girder fixed inside the skin ahead of the diaphragm.

The shield was worked by close poling and grouting the greater part of the face, which was in the plane of the cutting edge of the hood, but timbering the periphery with small easily removable boards. Commencing at the highest part of the face a board was taken out and a hole 30-38 cms (12 to 15 ins) wide and 0.56 m (22 ins) long was raked out of the gravel. This hole was immediately plugged with soft remoulded clay which tended to be held in place by the escaping air. A similar hole was made next the plugged one and the process repeated until a ring of clay with an outside diameter some 5 cms (2 ins) greater than the shield had been built up. When the shield was advanced the hood became buried in this ring of soft remoulded clay. The timbered face was now inside the hood where it could safely be set forward ready for the next cycle of the operation. Because the clay adhered to the ballast more than to the outside of the shield a tube of clay was formed outside the shield by successive advances. When this tube of clay was left behind the tail plate it formed a membrane which retained the air to that a pressure to hold the ballast back while the space was grouted was available.
Progress with the new system was 18 rings per week in a full face of ballast compared with 12 to 14 rings per week using the close poled grouted heading. If the hooded shield entered the clay its performance is not recorded.

The 7.05 m (23 ft) internal diameter tunnels for the City station and crossover were in London Clay, cast iron lined and shield driven. The shield had twentytwo 1.78 cm (7 ins) diameter hydraulic jacks equally spaced around the shield. The Hydraulic Power Company supplied water at 5,200 kN/m² (750 lb/in²) which was increased to 13,800 kN/m² (2,000 lb/in²) by hydraulic intensifier behind the shield. Because of the large number of rams the three straight tunnels were steered quite accurately. As at Blackwall a hydraulic segment erecter was used. No timber heading was used, the tunnel being excavated full face. This shield was 3.05 m (10 ft) long so there may have been more working space at the face than in the smaller shields.

Disturbance of the surface over some of the tunnels through the clay is reported but the size of tunnel is not mentioned. Deacon (1899, discussion p 11) mentions cracked buildings and Moir (1899, discussion p 124) records that the curbstones in Victoria Street tilted. There are several reasons why settlement should have occurred:

(1) the shield was not used as Barlow intended it should be, but had a heading driven ahead of it which was then collapsed;

(2) on sharp bends it was necessary to excavate up to two inches of material at either side of the cutting edge in order to get the shield to turn;

(3) in order to negotiate these tight bends the clearance between the outside of the lining and the inside of the tail plate was large; in combination with a poor seal for the grout this could have resulted in the system being less effective than intended.
2.3.9 The Central London Railway

In its final version the Act for the Central London Railway was for a 9.3 km (5 miles 6 furlongs) long line of 3.50 m (11 ft 6 ins) diameter twin tunnel running from Shepherd's Bush to Bank. There were eleven intermediate stations and the Bank station was to be separate from the City station of the City and South London Railway. The tunnels are deep, between 18 and 33.5 m (60 and 110 ft). They are almost entirely in the London Clay, although the invert is in the Woolwich and Reading Beds between Berners Street and Red Lion Street, Holborn (Engineering, 18 March 1898). There was clay beneath the Fleet Ditch but compressed air was used so presumably it had softened.

The engineers were J H Greathead with Sir John Fowler and Sir Benjamin Baker of the Metropolitan Railway. When Greathead died on 21 October 1896 his place was taken by Basil Nott* who had worked with Greathead on the City and South London railway and whose uncle was its chairman.

The running tunnels were of nominal diameter 3.50 m (11 ft 6 ins), the size favoured by the 1892 Parliamentary Committee. (Greathead himself, in evidence, recommended 3.66 m (12 ft).) The actual internal diameter of the rings was 3.56 m (11 ft 8 1/4 ins) giving a wriggle allowance of 57 mm (2 1/4 ins) compared with 44 mm (1 3/4 ins) allowed on the Waterloo and City railway. The increase was achieved by making the full depth of the flanges 124 mm (4 7/8 ins) instead of the 130 mm (5 1/8 ins) used on the Waterloo and City.

Sir Basil Nott (1924) said that the traffic department wished to use larger cars and that 'after careful consideration it was decided to reduce the flanges of the cast iron tunnel to a minimum'. Other explanations of

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*Knighted 1930, elected FRS 1932.
comparable credibility have been given. They probably arose because there was some mystery about the diameter at the time, contemporary reports in the technical press giving it as 3.50 m (11 ft 6 ins). Probably the company thought it inadvisable to advertise that the 11 ft 6 ins given by their 1891 Act of Parliament had been rather liberally interpreted. The first fifteen rings next to each station headwall were encased (disguised, perhaps) in concrete. What is certain is that the internal diameter of 3.56 m (11 ft 8 1/4 ins) that became standard on the 'Underground' for many years, and rings with 124 mm (4 7/8 ins) deep flanges proved structurally sound.

The ring of the now well known pattern of six long segments and a key segment and 0.51 m (20 ins) long, assembled with 22 mm (7/8 in.) diameter bolts with circumferential bolt holes arranged to allow the possibility of the rings breaking joint. They were mostly built, as they are nowadays, with continuous cross joints and with key segment placed centrally at the crown.

Whereas the L&SWR engineers had had a hand in the Waterloo and City Railway, Greathead was unopposed here and the new railway was built according to City and South London methods. The cross flanges were unplanned and the joints were packed with creosoted pine and pointed with Medina cement. The temporary seal of tarred hemp rope was used between the bolts and the fillet of the circumferential joint, to exclude wet grout. After grouting this joint was also pointed with Medina cement. In wet places rust cement replaced the pointing (Copperthwaite, 1905). Curves as sharp as 61 m (200 ft) radius were made using iron packings between the fillets of adjacent rings, two to each segment, to ensure that shield trust was transmitted through the skins of the segments. Tapered rings were used for bonds in the 2.44 m (8 ft) diameter footway passages at stations.
The shield driven station tunnels were also of a size (6.46 m (21 ft 2½ ins) ID and 6.86 m (22 ft 6 ins) OD) and pattern that was to become adopted as a standard. Each ring was 0.46 m (18 ins) long and consisted of twelve long segments and a key segment. The circumferential joint had 83 bolt holes and the rings were built with 28 mm (1½ in.) diameter bolts so as to break joint. The three bolts of the cross joint were staggered, to make a stiffer joint so that the rings would build more truly to their circular shape. At the Bank terminus the two station tunnels were successfully built on curves of 100.7 m (5 chains) and 91.5 m (300 ft) radius by inserting hardwood packing up to 33 mm (1½/16 in.) thick in the circumferential joints (Copperthwaite, 1905).

Three contractors (strictly speaking they were sub-contractors to the Electric Traction Company) shared the work: George Talbot (Bank to Post Office), Walter Scott & Co of Newcastle on Tyne (Post Office to Marble Arch), and John Price (Marble Arch to Shepherds Bush). Walter Scott had been responsible for the Elephant to Stockwell section of the City and South London and had taken over on the City section when Edmund Gabbutt fell ill. Two of these contractors experimented with mechanical excavators.

Thomas Thompson, Scott's agent, designed the bucket excavator (Thompson, 1897) shown in Fig 13a (there are photographs of it in the tunnel reproduced in Engineering for 22 April 1898). The machine was mounted on a 'Goliath' type carriage (its 0.61 m (2 ft) diameter wheels are not attached in the illustration) which ran on rails 1.91 m (6 ft 3 ins) apart, and gave a vertical clearance of 1.73 m (5 ft 8 ins) over the space between the rails. Ordinary skips running on 0.61 m (2 ft) gauge rails could be run into or through this space, either collecting clay that fell from the rear of the
Fig 13a  The Thompson excavator 1897

after p 58
bucket ladder or taking iron through to the face.

The machine was attached to, and could rotate slightly on, a stout king post projecting from near the centre of the top of carriage. An electric motor of about 20 c.v. (20 h.p.) situated near the rear of the machine drove, through worm gearing, a horizontal shaft on the centre line. From this various winches were driven by bevel gears and cone clutches, and at the foremost end of the shaft, bevel gears drove the bucket chain itself. The arm carrying this was 5.18 m (17 ft) long and protruded through the shield where its front end met the face. In order to move the cutting parts vertically up the face the winch near the top of the machine tightened chains which ran along the extended side plates and down to the bucket ladder. To move the cutter horizontally, the whole machine was slewed on the carriage by cables between the two. At either side of the tunnel, cables were attached to the completed tunnel lining ahead of and behind the carriage. These passed round the winch barrels protruding through the sides of the carriage. By operating these winches the whole equipage could be advanced, carrying the cutter further into the face. When building rings, or in the event of a breakdown, it could all be moved back some 3 m (10 ft). The Thompson excavator was built by Messrs E Scott and Mountain of Newcastle.

The shield used with this machine was short (2.15 m (7 ft ½ in.)) compared to its diameter of 3.86 m (12 ft 8 ins), but so was the proper Greathead shield of this period (1895-1903). It differed in having the diaphragm, which was an essential part of the shield as envisaged by Barlow and Greathead, reduced to a mere annulus between the cast iron segments of the cutting edge and those carrying the rams. In fact, the area of face available with the full diaphragm would not have been much less than that obtained with the
modification. Perhaps the excavator buckets themselves began the removal of the diaphragm. The control with cone clutches may have been limited. Nevertheless, a fashion was set for reducing diaphragms and so reducing security and safety. With the methods of construction used at the time the modified shields were also less rigid. Another modification to the shield was the provision of a small sloping apron to prevent clay which fell off the buckets filling the bottom of the shield.

The progress with combinations used by Thompson was three rings in a 10 hour shift with eight men at the face (Engineer, 22 April 1898). Copperthwaite said that the number of men was just over half the number required for hand mining and that the progress was about double. Perhaps he means maximum progress because on the previous page we find ... '...the frequent failure of the electrical motor and connections causing so much loss of time that the increased rate of excavation when the machine was working was made of no effect'.

The machine was versatile enough to cope with the large claystones found in the basement beds of the London Clay, although a few of the cast-steel buckets were broken. These were replaced by gun metal buckets.

John Price's machine (Price, 1896) was a rotary machine which cut over the whole surface of the face at once. Rotary machines had been made before this date but were made for cutting rock. The most famous of these was used by Colonel Beaumont and Major English in 1884 to drive 2.14 m (7 ft) diameter pilot headings in the Lower Chalk for a Channel Tunnel; its maximum progress on the English side was 22 m (72 ft) in 24 hours, in the more argillaceous material on the French side it achieved 25 m (82 ft) in 24 hours (Bramwell, 1885).
In its original form, as used between Shepherds Bush and Marble Arch in 1897, Price's machine did not work very well. It was vulnerable to the large claystones and its attachment to the shield made removal of the stones by hand difficult. Its long central shaft was not stiff enough to drive sharp curves. The drive was applied directly to the shaft.

Probably the potentially more versatile Thompson design was conceived as an answer to these shortcomings. However, improved versions of the Price machine were soon to be built (see page 64) whereas development of the Thompson concept was postponed for over 75 years.

The six or so miles of the Central London Railway running tunnels were completed between April 1896 and November 1898. The railway was opened by the Prince of Wales on 27 June 1900 and then closed for trial running. The locomotives, of American design, weighed 44.7 tonnes (44 tons), 34.5 tonnes (34 tons) of which was unsprung because the gearless motors were directly on the axles. The rails were of too light a section and mounted on longitudinal sleepers. These factors combined to cause vibration of sufficient magnitude to cause the Board of Trade to set up a committee of investigation under Lord Raleigh and to make property owners shy of granting way-leaves for the next six years. The vibration problems were solved but underground railway tunnels continued to have 5 chain curves.

2.3.10 Iron and bricks

The Great Northern and City Railway, engineered by Sir Douglas Fox and built by Pearsons, was envisaged as an extension of the JIR to carry their full-sized suburban trains through a 5.5 km (3 miles 3 furlongs 3 chains) long tunnel* from Finsbury Park to the City. In fact it functioned as an

*Except for Drayton Park Station which is in a cutting.
independent railway between its completion in 1904 and its take-over by the Metropolitan in 1913. It now forms part of LTB's Victoria Line between Finsbury Park and Drayton Park, the remaining portion between Drayton Park and Hoorgate being regarded as a spur of the Northern Line.

Commenced in 1898 after six years of bickering with the parent company, the 4.88 m (16 ft) diameter tunnels are of a unique construction (Fig 14). The rings were of the usual 0.51 m (20 ins) length but consisted of eight long segments with two short key segments, one at the crown and one at the invert. The flanges had a full depth of 15 cm (6 ins) giving an OD of 5.19 m (17 ft). The cross joint faces were not planed, wood packing was inserted into the joint as was Greathead's practice. The circumferential joints had the usual wooden packing. The tunnel was driven with a shield using these rings in the normal way. Some way behind the shield a gang followed which removed the invert iron, and the layer of grout and then excavated a further layer of clay. More was taken from invert than from axis level so that a slightly elliptical shape was formed. A three-ring lining of blue vitrified brick was built onto the clay. The junction between the brick and the iron was made with a shoe which spread the hoop load in the iron over the whole width of the brickwork, about 35 cm (15 ins). Flat bottomed rails were laid on longitudinal timbers on a wedge shaped fillet on the brickwork. With the rails so close to the lining the impression was created that the tunnel was much more elliptical than in fact it was.

The advantages claimed for this system were that iron was saved, the noise in trains was reduced and the vibration felt at the surface was reduced. The saving in money was estimated at £30,000. With the increasing cost of
Fig. 14  Lining of Great Northern and City Railway tunnels.
labour it would not have been economic to have used this method subsequently. Possibly iron may become so scarce in the future that something similar might be done with a concrete invert.

The large tunnels on the Great Northern and City Railway which contained the cross-over roads were also unique at the time. They had a horizontal diameter of 9.15 m (30 ft), but were built flat in the invert. They were hand built without a shield and recently the method has been used quite satisfactorily for similar cross-over tunnels at Brixton on the Victoria Line extension and at Heathrow Central on the Piccadilly Line extension.

2.3.11 The Hampstead tube and the Price excavator

The Charing Cross, Euston and Hampstead Railway was the last of the underground railways considered by the 1892 Committee. After four different Bills had received the Royal Assent it was finally financed by the American promoter C T Yorke and the Anglo-German banker Edgar Speyer. It did not actually become part of their Underground Electric Railway until 1910.

Sir Douglas Fox was appointed as consulting engineer with W K Galbraith. Their resident engineer was A W Donaldson. H H Dalrymple-Hay was now engineer to the Underground Group and kept a watching brief on its behalf. The contractors were Price and Reeves. The design of the linings (Figs 8 and 10), an amalgam of ideas from the Waterloo and City and the Central London Railways became the standard for the 'Underground' network. That for running tunnels was 3.56 m ID, 3.81 m OD (11 ft 8½ in ID, 12 ft 6 ins OD). The two top segments had eight circumferential bolt holes, the key one and the other nine making a total of 53 around each circumference of the 0.51 m (20 ins) long ring. The circumferential joint face had a 22 mm (7/8 in.) deep fillet at the back but was not used, all adjustments of tunnel direction being made
with wood packing. The cross joint faces were machined and had a caulking groove. The joint was made with the usual three 22 mm (7/8 in.) bolts.

The route was an ambitious one, 10 km (6 miles) from Charing Cross, the South Eastern Railway station forecourt where the Fleet line station is now being constructed, to Golders Green with 10 intermediate stations and a spur from Camden Town to the foot of Highgate Hill (now called Archway station). There is a 83 m (272 ft) rise between Charing Cross and the portal near Golders Green station. At Hampstead Heath the cover is 76 m (250 ft) above rail level. It also has the deepest station on the system, Hampstead, 59 m (192 ft) down.

The project took five years to complete (July 1902 to June 1907). During this time the 'Bakerloo' was not only completed but extended south to Elephant and Castle and north to Marylebone and the Great Northern Piccadilly and Brompton Railway (Hammersmith to Finsbury Park) was started and nearly finished. All this work used the standard 'Underground' methods finalised on the Hampstead tube. Motor buses were now profiting from the LCC's road improvements and no new line was built in the face of this competition.

By the time the various other lines were extended they too were part of the Underground group or, after 1933, LPTB.

John Price's tunnelling machine was improved for work on the Hampstead tube and in its improved form was used on the Piccadilly and Bakerloo lines and some of the extensions. The outside was a Greathead shield a little longer than usual, 2.24 m (7 ft 4 ins), and with the diaphragm removed (Fig 13b). The interior of the shield was bridged by two stout beams (23 cm x 8 cm x 1.6 cm (9 ins x 3 ins x 5/8 in.) steel channel section) mounted horizontally.
Fig 13b The Price rotary excavator Mk II 1907
just above axis level. One was in line with the fronts of the ram cylinders, the other in line with the backs; in fact, two of the ram cylinders were modified so that they formed the attachment between the beams and the shield skin. As well as stiffening the shield they had bearings hung from them which carried the 203 mm (8 ins) diameter axial shaft. The rear mounting had a thrust bearing as well as journal bearing. The front end of the shaft protruded beyond the cutting edge of the shield and carried a large cast metal hub to the front of which were bolted six radial arms; another six were bolted to the back of the hub. Circumferential plates bolted to the arms tied them together to form a wheel of considerable rigidity. The back of the wheel had an annular rack about 3.2 m (10 ft 6 ins) in diameter, its teeth facing towards the shaft. This meshed with gear train ending at the shaft of a 37 to 45 kW (50 or 60 h.p.) electric motor. Motor and gear train were both carried on the 23 cm x 7.6 cm (9 ins x 3 ins) beams at the left hand side.

Digging tools could be bolted to the arms in positions chosen by the engineer to give a suitable distribution of cuts over the whole face. Spoil cut by the digging tools fell to the invert from whence it was removed by six buckets each one attached to the leading edge of an arm. From the part of the bucket which was radially inward and to the rear there led a small, short chute. While the bucket was picking up muck from the invert this chute pointed upwards, but after the wheel had turned a little more than half a turn it was pointing downwards and the muck could fall from it into a large chute below. This chute led down and back just past the rear beam. From here it was carried away by a belt conveyor; this was rail-mounted so that it could be withdrawn when iron was being erected. From this arrangement of the buckets it will be appreciated that the excavating
wheel was only rotated in one direction although the tines were symmetrical. To counteract the machine's rolling under the torque reaction a 'plough' protruded from the shield, angled slightly to the direction of advance.

To enable ground to be cut in front of the hub, this carried a V-shaped toothed plate. This, with the points of the other digging tools, was about 0.46 m (18 ins) ahead of the shield cutting edge. An improvement suggested and incorporated by the manufacturers, Markhams, was to have the peripheral tines, which were mounted at an angle to the rotational axis, able to slide in their mountings. By means of push rods led to a cam or eccentric on the centre shaft they could be made to protrude further for part of the revolution, thus making a deeper excavation one side into which the machine could be turned when a curve was being driven.*

Running at 500 rpm the motor turned the excavating wheel at 1½ rpm. The operator balanced the advance due to the shove rams against the torque exerted on the wheel by watching an ammeter measuring the current taken by the motor.

In regular use on the Hampstead tube, 360 rings were erected in four consecutive weeks and 108 rings a week (55 mm (180 ft)) could be expected in good ground. Access to the face was restricted so it would not have made good progress in ground with claystones large enough to damage the cutting tools. In the deep section of the line under Hampstead Heath the face slipped and buried the cutting wheel. The excavating part of the machine was dismantled and the shield was driven on by hand.

Apart from progressing at twice the speed of the shield used with hand tools,

*Overcutting can, however, cause more steering trouble than it solves (see page 110).
there would have been a greatly reduced settlement now that a heading was not driven ahead of the face. This advantage probably passed unnoticed by the tunnellers at the time but eventually paid dividends in the form of more readily obtained way-leaves when it was realised that less damage was being done. Had the lessons learned with the Thompson prototype been applied to the development of a version as rugged as the Price machine, even more careful tunnelling could have been done. With the Thompson there was the possibility of advancing the shield and cutter together and if this had been done, there would have been no cause of disturbance ahead of the cutting edge (the Price cutting head protruded rather far). Also, if work had to stop, the Thompson machine could be withdrawn and the face properly timbered.

2.3.12 The City and South London Railway and the clay spade

The first major tunnelling work after the war was to enlarge the City and South London Railway tunnels to the now standard diameter of 3.56 m (11 ft 8\1/4 ins)\(^{11}\). Between Moorgate and Clapham Common, where the linings had been of 3.098 m (10 ft 2 ins) or 3.200 m (10 ft 6 ins) diameter, this was done, two rings at a time, by rebuilding the 3.20 m (10 ft 6 ins) iron with five short, parallel-faced segments 251 mm (9\7/8 ins) long inserted into the radial joints (ie in all the cross joints except those next to the key segment). Because the cross joint faces were unmachined the new joints, like the original ones, were made with wood packings included in them. The rather flexible ring this produced was quite satisfactory (Jones and Curry, 1926).

A shield was used; this, like those used with excavating machines on the Central London Railway, had most of the diaphragm cut away, leaving a circular

\(^{11}\)Between August 1922 and December 1924.
hole 3.25 m (10 ft 8 ins) in diameter. Not only did the trains pass through this but all the cables and tunnel services as well. Because of the difficulty of gaining access to the clay with the traditional clay pick, pneumatic clay spades were introduced. Pneumatic tools were also used for breaking out the old concrete invert and re-threading old bolts (Jones and Curry, 1926, p 233; F N G Taylor, op cit, p 223).

The availability of pneumatic tools for soft ground mining, aided no doubt by the post war shortage of timber, meant that it was no longer worthwhile driving a heading forward of the face. Neither the extra working space, nor the power assistance of the shield were required. This would have considerably reduced the settlement caused in tunnelling, but the rates of progress were not increased. Mr Langfield of Charles Brand and Son who kindly gave the author information on pre-war practice and the Brand's tunnelling machine, quoted three rings per shift. He attributes this to the desire of the pre-war engineers to achieve maximum accuracy of construction, and their belief that if the gangs had been allowed to build a greater number of rings in the shift, the standards would have deteriorated. Groves (1945) quotes four rings per shift.

The other improvement to the City and South London Railway to be started in 1923, was to extend the line from Clapham Common to Horden. The contractor for the Horden to Tooting Broadway section was the Foundation Co Ltd and between Tooting and Clapham, Charles Brand and Son. Ten Price rotary excavators were employed driving 427 m (1400 ft) of 3.56 m (11 ft 8 1/4 ins) tunnel between them (Jackson and Croome, 1962, p 183). The route was at the junction of London Clay and Woolwich and Reading Beds. The excavators are remembered as being difficult to steer but in the hands of experienced

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*This railway maintained its identity until 1926.*
crews they did not deviate more than 12 mm (½ in.) from the desired line and level.

Between 1930 and 1932 the Piccadilly line tunnels were extended by 12.7 km (7.9 miles) from Finsbury Park to Arnos Grove. Using pneumatic clay spades and Greathead shields a weekly advance of 51 rings (26 m; 85 ft) was attained (Jackson and Groome, 1962). This is a little more than half the rate that had been obtainable with the Price machine.

Charles Brand's again used a Price machine shortly before the 1939-45 war. This was manufactured by Markham (Engineer, 15 June 1951, pp 779-780) but was to the Price design. 2.96 m (9 ft 8¾ ins) long and 4.66 m (13 ft 2¾ ins) O.D. it was intended for use with 4.42 m (12 ft 3 ins) I.D. iron lining. It was employed between 1936 and 1939 (together with six Greathead shields using 3.66 m (12 ft 0 in.) I.D. linings) for the extension of the Northern line from Highgate (now Archway) station to East Finchley. A rate of progress of 100 rings (51 m; 167 ft) per week was achieved. Brand's used the same machine for two of the forty drives for tunnels completed in 1939 for the eastward extension of the Central Line (Groves, 1945).

G C Brand (op. cit, discussion p 43-44) explains that, although only a 3.66 m (12 ft 0 in.) I.D. tunnel was required, better progress (again 100 rings per week) could be achieved if the extra 76 mm (3 ins) of diameter was available as an allowance for wriggle. The average rate of progress with Greathead shields and clay spades was 45 rings (23 m; 75 ft) per week here.
2.4 CONCRETE

Mass concrete had certainly been used for lining tunnels in self supporting ground by 1890 (W R Galbraith *, 1893, 1904). This led engineers to consider saving iron by removing the invert segments of shield-driven tunnels in London Clay and replacing it with a mass concrete invert before the track bed was laid (eg Moir, 1895). Greathead's (1895) response to this suggestion was: 'There were localities no doubt where concrete could be more economical than iron but it would probably be found best in such cases to build the tunnel of concrete with the shape of moulded segments or blocks, thoroughly set and hardened before use. Concrete in this form would be capable at once of resisting pressure and excluding water'.

2.4.1 Tunnels with grouted concrete linings

2.4.1.1 The McAlpine lining

In 1903 reinforced concrete was used by the contractors Robert McAlpine and Sons for the lining of a 2 m (6 ft) sewer tunnel in Glasgow (Easton, 1911). An improved version of this lining was first used in London Clay under the Norwood Junction station, Croydon, for a pedestrian subway (Carter, 1911); (McAlpine, 1910). This tunnel of 2.89 m (9 ft 6 ins) internal diameter was driven without a shield. The rings were 30 cm (1 ft) long and consisted of seven segments, 17.8 cm (7 ins) thick, and a key. The cross joints between the segments had a tongue and groove form. The tongue,

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* W R Galbraith was engineer in charge of new works for the London and South Western Railway from 1862 onwards; between 1880 and 1890 he was Consulting Engineer to the North British Railway. From 1892 onwards he was concerned with underground railways, starting with Greathead on the L & SWR owned Waterloo and City Railway. In discussion in 1893 on Rickard's 1900 paper he said that he had lined tunnels on the North British Railway. In the discussion on a paper by Wood-Hill and Pain in 1904 he said he had lined two tunnels on this railway, and some in the chalk on the L & SWR, with mass concrete.
which was about one third of the thickness of the segment, bottomed in the groove so that the outer thirds of the joint remained as open gaps. Later the inside gap would be pointed and the outside one grouted. The circumferential faces of the ring each had a groove of semi-circular section running round them. When the circumferential joint was made, a ring of steel rod of 2.9 cm (1⅛ in.) round section was inserted, in two pieces, into the space between the segments. Channels led into this space so that when the ring was grouted any clearance between the rod and the segments was filled and the rod acted as additional circumferential reinforcement as well as being a shear plug for the joint.

The rings were made to breakjoint except for one key joint because successive rings were rolled in opposite directions by the width of the key segment. White glazed tiles were applied directly to the interior surface. The concrete was a rich mix, 3:1, and was made with 2 cm (¾ inch) granite aggregate. The segments were cast in cast iron moulds on a shaking table and not removed for three days.

In 1931 McAlpine and Sons won a contract for sewers for the West Middlesex Main Drainage (Watson, 1937). Their tender included as an alternative, a proposal to use the McAlpine lining. The Government had informed the County Council that a grant in aid of financing the work would be conditional upon (1) an early start; (2) completion within four years; (3) 90 per cent of the labour force being recruited from local labour exchanges; (4) 70 per cent of these to be men originally from the 'depressed areas'. By this time sewers of circular section with vitreous brick inner linings had been decided upon.
The County Council therefore chose McAlpine linings for 38,624 m (42,240 yards) or nearly half of the tunnelled sections because:

(i) untrained labour could be used in the concrete casting plant;

(ii) where the new lining replaced brick tunnels the demand on brick manufacturers would be eased, shortening the time over which deliveries were made;

(iii) where the new lining replaced cast iron, it would be easier and quicker to lay the inner lining of vitreous bricks saving time and skilled labour.

The lining was made in thirteen sizes ranging from 1.2 m (4 ft) I.D. to 3.2 m (19 ft 6 ins) I.D. With better concrete technology the segments could now be made cheaper and faster because a strength of 2500 lb/in² at 7 days could be obtained with a cheaper mix; also flint aggregate was suitable and readily obtainable locally, and the moulds were occupied for less time. Reinforcing was omitted from the thicker segments. The circumferential steel rods between rings were now between 1 cm (3/8 in.) and 1.6 cm (5/8 in.) diameter according to the ring size. Rings of all diameters were 30 cm (1 ft) long, those below 7 ft 9 ins diameter consisted of five segments and a key segment; above this size more segments were used. The 2.36 m (7 ft 9 ins) size had segments 12.7 cm (5 ins) thick. The rings were rolled as at Croydon.

The concrete lining was only used for tunnels in the clay. The whole area consists of London Clay overlain by 3 to 6 metres (10 to 20 ft) of flood plain or terrace gravels. The scheme was designed so that most of the tunnelled sewers would have a clay cover. The tunnels were driven without a shield, except for a short experimental length at Hayes driven with a
Greathead shield. The rings were grouted to refusal at 550-690 kl/m$^2$ (80 to 100 lb/in$^2$) after the joints had been pointed. It was found that the grout layer was 5 to 7.5 cm (2 to 3 inches) thick – about twice as thick as that produced when a shield is used. It was also found necessary to provide grout holes in the brickwork, which unlike London practice, extended round the whole 360° of the intrados. When the centres and laggings were struck the single brick inner lining often separated from the concrete at the crown, leaving the space which needed grouting.

Watson states that the linings could be erected 'with a tolerance of $\frac{1}{2}$ inch in line and level'. He does not say how truly circular the tunnels were or how the shape of the rings was controlled. The tunnels were watertight even before the brick lining, which was chosen for its resistance to abrasion and hence constant hydraulic friction, was placed. The brick lining could therefore be built in long lengths which resulted in more satisfactory work.

An example of the use of the McAlpine segments without a secondary lining is the tunnel driven through London Clay from the Shell Building to a shaft in the bed of the Thames. This tunnel is 3.05 m (10 ft) internal diameter and carries two 0.69 m (27 ins) cooling water pipes. The concrete of the segments is reinforced (Measor and Williams, 1962).

2.4.1.2 Bolted reinforced-concrete linings

In 1937 whilst the then London Passenger Transport Board was extending the network of tube railways the demands of the nation's rearmament programme began to affect the supply of iron. The Board and its consultants began to investigate the possibilities of replacing the cast-iron segments with
reinforced concrete. As a result a bolted reinforced concrete lining was designed and 4 3/4 km (2 3/4 miles) of tunnel on the Ilford branch of the Central Line between Red Bridge and Newbury Park stations were lined with it before the start of the war. A version of it was also used to line tunnels used as deep air-raid shelters (Groves, 1943; Anderson, 1938).

A ring of the lining is 3.73 m (12 ft 3 ins) ID; 3.98 m (13 ft 3 3/4 ins) OD with 31 holes around the circumference. It is 51 cm (20 ins) long and consists of six long segments and a key 43 cm (15 1/4 ins) long at the intrados.

The segments have an unreinforced skin 5 cm (2 ins) thick. The circumferential flanges have a full depth of 4 7/8 ins and are 10 cm (4 ins) wide. They are reinforced with four 1 cm (3/8 in.) diameter steel bars with 0.6 cm (1/4 in.) diameter stirrups. Also included in the flanges are steel ferrules to line the bolt holes. The cross flanges have similar reinforcement and ferrules. Parallel to the cross flanges there are four ribs which were to resist jacking forces. These have a trapezoidal section being 5 cm (2 ins) wide at the intrados and 10.5 cm (4 1/8 ins) wide where they meet the skin, and are reinforced with two 1 cm (3/8 in.) bars. The reinforcement constitutes 2.1 per cent of the total volume of the segment and although the concrete made good quality test cubes $\sqrt{41.4 \text{ ksi}} = 28 \text{ days}$ the surfaces of the reinforced ribs and flanges show many fine cracks. In tests by the Building Research Station on similar segments this defect is reflected in the very low flexural modulus measured (Ward, 1966).

In building the lining more than usual care was required in handling the
segments and 3 mm (1/8 in.) thick bitumen-hessian packing was placed in the cross joints. Also grommets were used under the bolt washers. The rings were erected with a two hole breakjoint by placing the key successively two holes right, central, two holes left and so on. Conventional soft wood packing was used between the rings.

This concrete lining bears a strong resemblance to the iron lining which it replaced. The reasons given for this are:

(i) that the Board wished to use the same cable brackets, lighting and telephone wire fixtures that were used in the iron lined tunnels. Since these fittings were attached by the circumferential bolts in the iron lining then the concrete lining would have to have similar bolts.

(ii) The signalling apparatus was fitted 'in the belly of the iron' ie between the flanges.

(iii) The same shield that was used for the iron lining of 3.66 m ID and 3.9 m OD (12 ft 0 in. ID and 12 ft 9 3/4 ins OD) was to be used for the concrete lining.

After some development work had been done with a concrete ring of the same ID and OD as the iron lining, it was found that requirements (ii) and (iii) were incompatible. If the ring was only 51 cm (20 inches) long to suit the stroke of the shield rams then the signals would not fit in the belly of the iron. In order to clear the structure gauge the ID of the lining was increased to 3.73 m (12 ft 3 ins).

Because of the cross ribs the pitch of the circumferential bolt holes was 39 cm (15.3 ins) compared with 22 cm (8.8 ins) in the iron lining. These
distances do not have a small lowest common multiple so presumably the
cable brackets did not fit either.

Hence none of the operational requirements was met and one is tempted to
accept the simpler explanation that the old form of lining was merely copied
in concrete, which as it is weaker required thicker sections. Some
difficulty was experienced getting in the large, heavy key segments. A
whole ring of lining weighs 5280 kg (5 tons 4 cwt) compared with 1680 kg
(1 ton 13 cwt) for conventional iron.

On the Central Line these segments were only used in the London Clay and
since 'tube' tunnels are very well ventilated by the piston effect of the
trains no difficulties were experienced due to corrosion of the reinforcement.

The saving in cost reported by Groves was 25 per cent in comparison with the
cost of equivalent linings at that time. Dr Anderson founded an industry
when he designed this lining for despite its shortcomings it provides a
cheap lining which can be used with or without a shield and is available in
a range of sizes virtually 'off the shelf'. The contemporary product has
an adequate concrete cover over the reinforcement, a high standard of finish
and is frequently used in water-bearing ground. It now has a caulking
groove (Fig 20a).

Bolted concrete segments are often used for small tunnels built without a
shield. Examples are sewers, cable tunnels and pedestrian subways which
may consist of short straight sections meeting at sharp elbow bends. The
rings of a small tunnel are stiff enough to be built onto the end of the
finished length of lining and hold their shape in an irregularly shaped
Fig. 20a  Bolted reinforced-concrete segment of 1974 with cast-iron segment of 1886 from the City and South London Railway
Fig. 20b Segments of Mott, Hay and Anderson flexibly jointed cast-iron lining
excavation. Once the ring has been grouted and become part of the tunnel however, the bolts are unnecessary. The disadvantage of the ordinary bolted rings for tunnels such as sewers is that a secondary lining is required to make the interior smooth.

A lining combining the smooth bore of the McAlpine lining with the versatility of the bolted lining was introduced by Spun Concrete Ltd in 1956 (Guymer and Dee, 1961). Superficially the segments resemble those of the McAlpine lining, but the finished tunnel rings contain no steel. Each plain concrete segment is sent to the face with a curved steel channel bolted to its circumferential centre line (Fig 20c). Coarse screw threads are cast into the concrete for this purpose. The ends of the steel channel are closed by stout plates positioned radially and in line with the ends of the segment that form the cross joints. The radial steel plates have a ground surface of about 10 cm x 10 cm (4 ins x 4 ins) with a hole at the centre. These holes enable a number of these channels to be bolted together to make a very stiff former ring. By bolting them together with the concrete segments attached to them a stiff tunnel ring is built. The key segment of the concrete ring has a corresponding key segment of the former ring bolted to it. The adjacent 'top' segments of both the concrete lining and the steel former ring are shaped to fit their respective key segments. Holes in the side of the steel former segments enable tie bars to pass from one ring to the next. The tie bars are threaded so that nuts may be used to tighten the tie bars and close the circumferential joints between successive rings. The ring is grouted in the ordinary way and when the grout has hardened the steel former ring may be dismantled and returned to the surface for re-use.
Fig. 20c 'Flexilock' lining by Spun Concrete Ltd.
showing segmented former-ring
Knuckle joints are used both circumferentially and longitudinally. In order to avoid a feather edge on the concave part of the joint, the knuckle does not reach the edge of the segment. At the extrados it is relieved to provide clearance and at the intrados it runs into a caulking groove. Between the concrete surfaces of the knuckle joint there is a strip of soft rubber-bitumen compound. This cushions the joint against high contact stress due to minor imperfections of the mating surfaces or inaccurate assembly. It also provides a primary waterproof seal which withstands slight changes of the shape of the ring during the early life of the tunnel.

This lining is made in various sizes from 1.37 m (54 ins) internal diameter, consisting of ten segments 8.3 cm (3\(\frac{1}{4}\) ins) thick, up to 3.67 m (12 ft) internal diameter made up of eighteen segments 17.8 cm (7 ins) thick. The latter ring weighs 3138 kg (6918 lbs). All the rings are 56 cm (22 ins) long. This lining has been used in the London Clay at many places including St Pancras, Croydon, Hounslow, Harrow, Ealing, Hammersmith and Southend.

Since 1956 a number of designs have appeared which solve the same problem. Kinneir Hoodie (Concrete) Ltd (now Charcon Tunnels) produced a 'Rapid' lining in 1962 (Cardwell, Guthrie and McBean, 1963) and the 'Universal' lining with circumferential tie bars in 1972 (Tunnels and Tunnelling, May 1972, p 188) (see Fig 20d). In the latter, the cross-joint bolts are replaced by circumferential tie bars joined by pairs of linked nuts. A pair of bars is attached to the links of the last placed segment and another segment is passed over them. The next two pairs of linked nuts are then screwed on to the threaded ends of the bars which protrude into
Fig. 20d  Kinnear Moodie 'Universal' lining showing a segment of circumferential tie bar with one of its connecting links.
'counter bores' at the ends of ducts in the segments. Tightening the nuts against the bottoms of the 'counter bores' tensions the rods and closes the cross joint at the other end of the segment. Since the bars cannot be a close fit in the ducts in concrete, the resulting assembly is rather springy. Since there is no equivalent of circumferential joint bolts, the newly built ring can gain no support from a previous one that is already grouted. Hence the lack of rigidity of the cross joints is a nuisance. Charcon Tunnels point out that these segments may also be built by the method described in section 2.4.2.

William Rees Ltd's solution for very small tunnels of 1.0 m (39 ins) and 1.2 m (47 1/2 ins) is to make a plain concrete lining of only three segments with only three knuckle joints. This, of course, provides a stable structure. Each segment has four longitudinal V-grooves equally spaced on both the inside and outside surfaces. These act as stress raisers so that as the tunnel deforms from its originally circular shape additional 'joints' form by fracture. Since the ring is in compression and the fracture surfaces are rough, displacements do not occur along them. The use of slow-setting weak grout assists the process. Initially the overbreak is filled with pea gravel, hence there is no risk of the tunnel moving in the semi-fluid grout (Rees, Garnett and Richardson, 1969).

William Rees manufacture a shield for this so-called 'mini-tunnel'. The demand for a small bore segmental tunnelling system has arisen because the previously used alternatives of pipe laying in open trenches or in timbered headings have become unacceptable due to the greatly increased use of the roads in one case and increased cost of labour and timber in the other. Also,
of course, the security of the shield system and the confinement of noise
favour this as they do other applications of tunnelling in urban situations.

2.4.2 Tunnel lining without grouting

The Thames has only intermittently been a source of potable water for
London; the conflicting requirements of transport, fishery, milling and
sewage disposal have demanded at various times that 'sweet waters be brought
from afar'. The City Conduit conveyed water from springs at Hampstead at
a very early date. The demands for quantity overcame those for quality
and the London Bridge Waterworks was built under London Bridge by 1581 using
waterwheels to raise the water. In 1613 Sir Hugh Myddleton's New River
(Davidson, 1948) brought water 62 km (39 miles) from springs at Chadwell and
Amwell in Hertfordshire where the River Lee" had cut down into the Chalk.
The first call on water from the Lee itself was for navigation, this
interest having been protected since 1425 by Royal Statute (2 Hen VI and
3 Hen VI c5) although in 1738 the New River Company was given Parliamentary
powers to divert some of the flow.

When the springs could not longer meet demand, boreholes were sunk through
the London Clay to the Chalk directly beneath. Those bores were mostly
privately owned by breweries and later, laundries, the proportion of the
total supply coming from this source reached 18 per cent (Davies, 1939).

Water taken straight from the Thames was first filtered in 1829 by James
Simpson (Davidson, 1948) but it was not until after the cholera epidemic of
1848-9 that in 1852 the Metropolitan Water Act demanded that water be taken

"Although 'Lea' is the spelling adopted by the Ordnance Survey, Lee is found
in the oldest statutes and is preferred by the Metropolitan Water Board.
from beyond the tidal limit and also filtered. The intakes of various companies appeared further up the river and the mains got longer and longer.

In 1867 the East London Water Company obtained powers to take water from Sunbury-on-Thames, a 0.91 m (36 ins) main took filtered water to a reservoir at Finsbury Park.

As London became larger, more water was required but the difficulties of finding routes for new mains increased. Eventually the Metropolitan Water Board decided to build large storage reservoirs in the Lee Valley in order to capture the winter flow. The last of these was completed in 1939. In 1943-44 the depletion of the reservoirs during the hot summer was not made good during the dry winter that followed. After the war it became apparent that these reservoirs would have to be refilled in the winters with raw water from the Thames. By now the usual solutions of an open aqueduct or a pipe in trench were virtually impossible. Sir William Halcrow suggested to the then Chief Engineer to the Metropolitan Water Board, H F Cronin (1952) that a tunnel be used. From the lowest Thames water intake at Hampton a straight line route of 39 km (24 miles) could be taken to the King George V Reservoir at Chingford. The pumping station built at Hampton in 1937 could transmit the necessary 110 million litres (24 million gallons) of water per day through a 1.9 m (75 ins) main.

2.4.2.1 The Donseg Lining

Sir William Halcrow's firm was commissioned to study the problem of putting this main in a tunnel. Their H J Donovan (1947) had invented a cheap concrete lining called the Donseg lining*. It was proposed to drive a

*Scott (1952) called the segments 'Don-Segs'; Cuthbert and Wood (1962) introduced the preferred term 'Donseg'.
tunnel very little bigger than the main, sections of which would be welded together in the tunnel. The small space between the outside of the steel main and the inside of the tunnel would be filled with concrete. Thenceforth the tunnel lining would serve no further purpose (Scott, 1952).

The Donseg lining was to be used with a tunnelling shield but not inside it. Instead the tail of the shield was to be left off and the lining built in contact with the clay. Unlike previous tunnelling operations this would be a complete tunnelling system in which the shield, the lining and its method of erection all interacted. Hence for its development it was wisely decided to build a trial length of tunnel (Scott, 1952). Kinner Hoodie Ltd was commissioned to do this on a cost plus fee basis. The same firm also designed a special shield and worked out the method of casting the plain concrete lining segments.

The Donseg lining is shown in Fig 15. Each ring consisted of ten identical segments, each of which, instead of being symmetrical about the circumferential centre line of the ring, tapered longitudinally. Alternate segments were built into the ring facing in opposite directions. If, around a circumferential face of the ring, all the wide ends stood proud, the circumference of the ring would be small. On pushing the wide ends into the same plane as the narrow ends the ring would expand circumferentially. If this were done in a hole in the clay which was slightly smaller than the finished ring, both the ring and the clay would be compressed.

The 305 m (1000 ft) long trial tunnel built at Stoke Newington in 1950-51 was 2.266 m (7 ft 6 ins) I.D. and 2.588 m (8 ft 5 7/8 ins) O.D. The rings were 0.533 m (21 ins) long and each segment was 135 mm (5 15/16 ins) thick.
The Donseg Lining

Fig 15
and weighed 152 kg (3 cwts). The external arc at the narrow end was 0.750 m (2 ft 5½ ins), at the wide ends it was 0.876 m (2 ft 10½ ins) giving a taper of 1 in 4.2. With five segments moving, the ring would expand diametrically 0.379 in. per inch of longitudinal movement.

So that the cross joints should fit regardless of the segment being displaced in the direction parallel to the axis of the tunnel, all transverse sections of segments are bounded by radial lines at the cross joint faces. Hence the cross joint faces are not plane but have a 'wind' on them. This was one of the problems of manufacture that had to be solved by Kinnear Hoodie (Concrete) Ltd. The concrete mix, 3:1½:1 of 9.5 mm (3/8 in.) flint, sand and Portland cement and with a water/cement ratio of 0.42 gave a strength of 42 to 52.5 MN/m² (6,095 to 7,590 lb/in²) at 28 days.

The shield made by Arthur Foster (East Ham) Ltd was to an unusually rugged design because the jacking forces were unknown. Also distortion of the cutting edge which would have been acceptable in a grouted tunnel could not be tolerated here because the Donsegs were designed to fit a perfectly cylindrical hole. It was 1.803 m (5 ft 11 ins) long including a 30 cm (12 ins) long hood. There was provision for attaching a tail plate should bad ground be encountered, ordinary bolted segments would then have been used. There were ten shove rams - one for each segment. The shoes extended over an arc a little less than that of the narrow end of a segment so that the shield could roll a little. The rams had an unusually long stroke 0.711 m (28 ins). The diameter of the ring calculated from the dimensions of the segments is 2.586 m (8 ft 5½/16 ins). Scott does not say whether a bead was used on the cutting edge but since the drive was straight any bead may have been quite small. The diametrical clearance appears then to have been about 4.7 mm (3/16 in.).
After excavating and mucking a 0.53 m (21 ins) length, which took 90 minutes, the five invert segments were laid on the clay. A steel ring 2.21 m (7 ft 3 ins) diameter was placed on these and the arch built on this with the aid of chocks. There were also pins which could be withdrawn from the ram shoes to stop the leading edge dropping.

Expanding the ring was not just a simple matter of applying pressure to all the rams because as the ring expanded the segments had to move circumferentially against the friction of the concrete on the wooden ram shoes. Hence a certain amount of joggling was required before the ring could be finally tightened against the clay.

The usual wood packing was used in the circumferential joint before the next ring was built. Tolerance on line, level and circularity was regarded as satisfactory. The average progress was 21.3 m (70 ft) per week or four rings per shift of 12 hours.

In the early part of the trial, hydraulic pressures of 13.8 kN/m² (2000 lb/in²) were used and the corners of the segments were broken. Assuming the ram diameter to be 178 mm (7 ins) the thrust on a segment would be 350 kN (35 tons). By rearranging the order of placing the segments the thrust was halved. Finally, the segment dimensions were modified to give a ring 1/16 inch less than the exact diameter of the excavation. Pressures were reduced to 200 to 250 lb/in² but with no diminution of the tightness of the lining against the clay. Grease was used to reduce the concrete-concrete friction at the cross joints, but later a coat of bitumastic paint, previously applied, was found efficacious.
The thickness of steel necessary to contain the water pressure was 4.7-6.4 mm (3/16 to 1/4 in.) according to the position along the length of the main. However, prefabricated lengths of 1.91 m (75 ins) diameter pipe of these thicknesses could not be handled, let alone transported from Middlesborough and taken into the tunnel. This difficulty was resolved by making the steel 12.5 mm (1/2 in.) thick, spinning a 25 mm (1 inch) thick layer of concrete onto the inside and adding supporting spiders (for transport only). The units used at Stoke Newington were 4.4 m (14 ft 6 ins) long and weighed 241 kg (4 cwt 3 qtrs). Their spigot and socket joints were welded in the tunnel and pressure tested to 1.035 MN/m² (150 lb/in²) before they were grouted with concrete. The welded joint was covered by the Gunite process.

The trial length of tunnel was quite satisfactory but 22,353 tonnes (22,000 tons) of steel would now be required for the whole tunnel and at this time there were serious doubts whether this could be made available for this job. The Metropolitan Water Board then considered leaving out the steel lining altogether and relying on the fairly impermeable London Clay to retain the water. This idea became feasible when it was decided to make the main a gravity one by putting the pumps at the Lee end (at Lockwood) and increasing the diameter to 2.59 m (102 ins). There was a risk that should the pumps fail a surge would occur which would cause the hole in the clay to expand so much that the Donsegs became dangerously loose. A suitable syphon intake was designed, but clearly more knowledge was required about the Donseg lining and whether a higher pre-stress could be jacked into it and the surrounding clay.

At this time the Metropolitan Water Board required a pressure tunnel between
The Queen Mary reservoir and their new filtration works at Ashford Common. They enlisted the help of the Building Research Station of the then Department of Scientific and Industrial Research and engineers from the soil mechanics laboratories of both organisations designed and carried out suitable experiments in this tunnel (Tattersall, Wakeling and Ward, 1955).

The tunnel is 574 m (1883 ft) long and lined with Donsegs 15 cm (6 ins) thick giving an internal diameter of 2.54 m (100 ins). The tunnel runs between two shafts with its axis at a depth of 27 m (89 ft) below ground level. The cover is 21 m (70 ft) of London Clay overlain by gravel. Top water level of the reservoir is 10 m (33 ft) above ground level. The theoretical overburden pressure therefore exceeds the static water level by about 45 per cent; but the earth pressure on the tunnel lining would probably be less and the water pressure could momentarily be more than these figures.

Instruments installed by Tattersall, Wakeling and Ward included remote reading diameter gauges, load cells to measure hoop loads and radial earth pressure, and gauges to record any opening of the cross joints.

The investigation showed that by suitable lubrication of the segments and application of the jacking forces, hoop loads as high as 350 kN (35 tons) could be applied. The load equivalent to the overburden acting hydrostatically was 400 kN (40 tons). Although it was not known how far from the ring the zone of compression extended, it was evident that the clay stayed strong enough to contain this compression because this high hoop load remained in the ring.
When the tunnel was filled with water under the normal working head of 37 m (120 ft) the tunnel diameter increased between 2.5 mm and 5 mm (0.1 and 0.2 in.). There was no measurable movement between adjacent segments. The pressure of the clay on the outside was reduced by about two thirds of the internal water pressure. There was no significant leakage.

On the basis of these investigations the then Chief Engineer of the Metropolitan Water Board decided that 27 km (17 miles) of the Thames to Lee tunnel would be in Donsegs with no steel lining (Cuthbert and Wood, 1962). The first main drive was begun in January 1955 and the last was completed in June 1959. The main was commissioned in September 1960.

Observation of the building of many rings of Donsegs at Ashford Common showed that the gangs generally placed nine segments in the positions that they would finally occupy in the ring, and used only the last segment to actually expand the ring. This avoided the initial joggling of the segments to prepare the ring for final compression. Tattersall, Wakeling and Ward therefore recommended that only one segment should have the double taper and it should be placed as a key between two 'top' segments shaped to fit it. With only one moving segment a greater amount of expansion would be required from this one 'wedge'. Hence it should be shorter than the other segments in order to allow it to travel further. It would be inserted from the leading edge of the ring. Later to be called the Wedge-block lining, this design was tried out in the Thames to Lee tunnel; 864 m (2835 ft) of tunnel were driven with an ordinary hand shield. It was successful and subsequently the Wedge-block lining replaced the Donseg lining.
2.4.2.2 The Bridge System

Another groutless tunnelling system was demonstrated in 1958 by Aubrey Watson Ltd (Hammond, 1958). Invented by E K Bridge (1954) it was designed for building a pre-compressed lining of concrete blocks in London Clay having a shear strength of 540 kN/m² (5 tons/ft²). A pre-compression sufficient to allow an internal hydrostatic head of 61 m (200 ft) of water was envisaged. A short length of tunnel of 1.83 m (6 ft) diameter was built at shallow depth in sandy clay at Tolworth. Because it was for demonstration purposes it was only driven intermittently, but a high speed of advance was intended and the creditable speed of 1.83 m (6 ft) per hour was demonstrated.

The concrete voussoirs, all identical, were about 13 cm (5 ins) thick, measured 30 cm (12 ins) in the axial direction and 23 cm (9 ins) in the circumferential direction and weighed about 18 kg (40 lbs). They were built as a two start spiral, behind a special shield with a pair of circumferential jacks of 425 kN (42.5 tons) capacity in addition to the normal longitudinal shove rams. The circumferential jacks provided the pre-stress in the spiral and also rotated the shield. The resistance to rotation was used to cut the clay. The advantages available from a spiral configuration were:

1. the lining process was nearly a continuous one and it did not interfere with the excavation and mucking which could be continuous too (they were also automatic).

2. The maximum area exposed behind the face was the area of two voussoirs, only 0.14 m² (1½ ft²). Only half of this could be in the roof so the likelihood of 'the top dropping out' (roof falls) was very small.
3. Compression was provided immediately after the block was placed (when the next block was placed it was held in a special device).

4. The size of the hole that was to be lined (see page 104) was not critical so that it was not necessary to have detachable cutting edge beads or make any other provision for matching the hole and lining.

5. The shield could be short (0.9 m, 3 ft) (see page 211).

A conical cutting head was attached rigidly to the front of the shield. It consisted of twelve or so arms spaced apart where they joined the shield, just inside the cutting edge, inclining towards each other and joining at the point of the cone. Each arm bore a number of tines which cut the clay. The centre of the cone could be equipped with a stationary pointed cutter, the altitude of which could be changed to facilitate steering of the shield. As with an ordinary shield, steering was normally done by differentially altering the thrusts of the shove rams which acted on the concrete lining via rollers. The diameter of the circular excavation was determined by the cutting edge which was the widest part of the shield and was provided with serrations so that while slowly rotating (one revolution in ten minutes, giving 0.305 m (1 ft) of advance) it acted like a hole saw. The whole shield was powered by a 5 cv (5 hp) compressed air motor.

The muck was removed by a shovel conveyor the shovels of which filled themselves at the face, and emptied themselves at the portal.

Despite the intermittent operation the face did not collapse during months
inactivity because the arms of the cutting cone held it. Had the trial been conducted at greater depths, in hard London Clay, the system would have worked better rather than worse then it did because as the resistance to cutting was increased so was the pre-stress in the lining.

With all these potential advantages the reader may be surprised that the system was not developed. The explanation does not lie in how well or how badly the trial length was executed. The successful line of soft ground rotary tunnelling machine began with a Price machine producing only 36 badly aligned rings in one week (Dalrymple-Hay, 1908, in discussion on Tabor, pp 215-217) yet was achieving 100 rings per week correctly aligned to 6 mm (1/4 inch) (Galbraith op. cit) before it was superseded. The reason must surely be that the role that would have been fulfilled by the Bridge system was already taken by the Donseg lining and therefore improvements by small steps taken with this lining (Wedge block and machine excavation) were more easily initiated than a complete change of system. The situation is analogous to that in biological evolution when the development of an emergent species is arrested because the ecological niche to which it is suited is already occupied.

2.4.2.3 Potters Bar tunnel

Most of the tunnels on mainline railways out of London were duplicated at an early date. The Great Northern Railway had had operating and financial difficulties arising from its dual role as a national railway and a London commuters' railway. It could not afford to duplicate all the five tunnels on its route across the South Herts plateau. The solution eventually adopted was to by-pass the bottle neck by extending the Enfield Branch to form the 'Hertford Loop'. This includes the 2.4 km (1 1/2 mile) long
Ponsbourne tunnel built in 1913. This was the last traditional brick two-track tunnel driven in London Clay.

In 1953 there remained a 4 km (2.5 mile) long length of twin-track on the main line. This included three tunnels: Potters Bar, Hadley Wood North and Hadley Wood South. British Rail decided that these must be duplicated; but with a forty year gap in their own and contractors' experience in tunnelling in London Clay by traditional methods, sought the advice of consultants. Sir William Halcrow and Partners suggested that the new technique of a concrete lining expanded directly against the clay be adopted for these large tunnels. This was successfully used for all three tunnels which are 1110 m (1214 yards), 212 m (232 yards) and 351 m (384 yards) long, and were completed on 3 May 1959. They are straight and are spaced at 15.2 m (50 ft) centres from the brick tunnels built in 1849-50.

The lining is circular and of 8.08 m (26 ft 6 ins) internal diameter and 0.685 m (2 ft 3 ins) thick. The rings are 0.457 m (1 ft 6 ins) long and consist of 20 segments with the middle third of any joint face raised or depressed 7.5 cm (3 ins) to form a shallow tongue and groove joint with its neighbour. The cross joint surfaces were painted with bitumen. Nineteen of the segments were short voussoirs subtending angles of 14° or 16° at the tunnel axis (Fig 16a). These were in plain concrete and weighed 760 kg (15 cwt) each. The invert consisted of a single reinforced concrete unit 6.7 m (22 ft) across the outside chord and weighing 3.3 tonnes (31/4 tons). It was the same thickness as the voussoirs except at the ends where two horizontal pedestals were cast on (Terris and Morgan, 1961). During construction these provided a surface on which a 33.5 m (110 ft) long platform could slide. This was towed behind the shield and carried the hydraulic
Figure 16a Precast concrete lining used at Hadley Wood and Potters Bar
pump for the shield jack, the belts for loading a train of skips with the muck taken from the face, a crane for handling segments etc.

The shield was 9.548 m (31 ft 6 3/8 in.) diameter over the cutting edge (Barter, 1962) weighed 152 tonnes (150 tons) (Terris and Morgan, 1961), was 2.74 m (9 ft) long and had provision for bolting on a tail plate 0.61 m (2 ft) long. The shield had 25 shove rams, six of which thrusted against the invert block. The remaining 19 rams each thrusted against the centre of one of the 19 voussoirs. The total thrust available was 9960 kN (1000 tons), but a thrust of 3984 kN (400 tons) usually sufficed. The shield was fitted with two hydraulically operated erector arms for placing the voussoirs above axis level, and also a lift for the key voussoir. The shield had three decks divided to form compartments and was fitted with nine face rams designed to retract as the shove rams moved the shield forward.

The first dozen or so rings of each tunnel were ordinary cast iron rings erected in the ordinary way in the tail of the shield. A buttressed pile structure was built outside the portal to provide reaction for the shove rams. When the tunnel had advanced far enough into the hill that the cover was sufficient, the tail plate was removed and the concrete lining was started. It was built up to axis level in the normal way, the last voussoirs being specials. They were a little longer than the others (subtending 16°) and had pockets in which hydraulic jacks were fitted. Another special voussoir was placed next followed by five ordinary ones each side, built with the erector arms (Fig 16b). The last ordinary voussoir was then inserted into the ring from the front. The jacks at axis level were then extended in order to expand the ring. They were left
Fig. 16b  Erector arm, loaded with concrete segments, in Potters Bar tunnel
extended while the space either side of them was packed with dry mortar. When this had hardened the jacks were removed and that space too was filled. There is no information about the jacking force used or whether this was varied according to the overburden load, nor is a longitudinal section given (Terris and Morgan, 1961; or 'The Engineer', 14 Feb 1956). The approximate average cover over the crown is 10 m (30 ft) (O.S. map and photographs). The stress in concrete is stated to have been about 6.9 MN/m² (1000 lb/in²). Considering that the concrete had a strength of 41.4 MN/m² (6000 lb/in²) at 28 days and was of mix chosen to be resistant to locomotive exhaust gases, the lining appears too thick. Some allowance has to be made for stress concentrations at the many refuges, cross passages and ventilation ducts.

The miners worked three shifts nominally of eight hours each, usually they built three rings in 6 hours. Thus average daily progress was 4.1 m (13 ft 6 ins) (Bulman, 1962). The 2345 concrete rings of the Potters Bar tunnel were built in 60 weeks.

2.4.2.4 Heathrow Cargo tunnel

In 1967-8 a large diameter shallow tunnel was built by similar methods in the London Clay underlying London (Heathrow) Airport again to the design of Sir William Halcrow and Partners. The facilities at the Central Terminal, which is completely surrounded by runways and taxi-ways, were no longer adequate to deal with the freight which is unloaded there from passenger aircraft. A link was therefore required with the Cargo Area to the south of the airport, and with little room for approach ramps it would have to be shallow. It was calculated that a single tunnel would be cheaper than a pair of twin tunnels.
The tunnel of 10.9 m (33 ft 9 ins) internal diameter, with a cover over the crown of 7 to 8 m (23 to 26 ft) of which only 2.5 m (8 ft) was London Clay, was driven under two runways with negligible disturbance. The lining was 30 cm (12 ins) thick, much thinner (Fig 8) than that used at Potters Bar and Hadley Wood, also the crown joint surfaces were slightly cylindrical with a radius of 3.175 m (125 ins). This more satisfactory design arose out of work done in 1962 for the Victoria Line of London Transport. The rings were 0.61 m (2 ft) long and consisted of 27 plain concrete segments. Quite a small invert segment was used. The ring was expanded by jacks of 350 kN (35 tons) capacity situated at axis level and used in the same way as described above.

Load cells were inserted into one ring. These indicated that the loads increased during the first two months after building and thereafter remained virtually constant. The values reached varied according to the position of the gauge in the ring; expressed in percentage of the hoop thrust at each position for the full overburden pressure acting on the lining they were 100 per cent (460 kN) at the crown, 60 per cent (570 kN) at axis level and 55 per cent (620 kN) at 25° from the invert.

The shield, as used, without a tail plate, was 3.1 m (10 ft) long and propelled by 28 shove rams - one for each segment except the large one at the invert for which there were two. Each ram was rated for 800 kN (80 tons) thrust at 31 MN/m² (4,500 lb/in²). The shield was divided into five levels by four decks. These were each subdivided into four boxes by vertical girders. The boxes were equipped with a total of 56 face rams and also platform rams. These were individually controlled by the miner from the box he was working in. The load at which the face rams retracted
Figure 16c  Heathrow cargo tunnel construction
during a shove was adjusted by relief valves which were set for each level in the shield (Wood and Gibb, 1971).

The segment erector was designed so that it could to some extent take over the duties of the missing shield tail. It consisted (see Fig 16c) of a two-pinned three-segment arch, attached to the rear of the shield and fitted with vertical and inclined hydraulic rams so that in the retracted position there was sufficient room to assemble the top half of the next ring of segments on it. Care was taken to see that such a half ring was correctly assembled before the shove was made. Then, immediately the shove rams were withdrawn, the erector arms could be extended and the roof supported within a minute. The invert segments could then be placed and the ring expanded in safety. Total surface settlement above this tunnel was 12 mm (½ inch), half this had occurred when the tunnel face was immediately below the point of measurement (Smyth-Osbourne, 1971). The horizontal diameter of the tunnel increased about 2.5 mm (0.1 inch). Experimental work (carried out elsewhere) to be described later, suggests that these very satisfactory results were due to the shortness of the shield in relation to its diameter and to the speed with which the rings were erected as well as to the shield face rams.

2.4.3 Re-appearance of the tunnelling machine

Another innovation tried on the Thames to Lee tunnel was a hydraulically driven shielded tunnelling machine designed by Kinnear Moodie Ltd for use with the Donseg lining and built by Arthur Foster (East Ham) Ltd. Two machines were used in five drives totalling 8.8 km (5½ miles) (Cuthbert and Wood, 1962).
In designing a tunnelling machine of 3.01 m (118 1/2 ins) diameter, later to be made only 2.79 m (111 ins) diameter, the pre-war underground railway practice of mounting a full-face cutting head on a central shaft was unacceptable because there was no room between this and the motor and gearing for disposal of the excavated muck. The solution adopted was to mount the cutting head on the forward end of a quite large cylinder through which the broken and bulked clay could pass. The cylinder, or 'drum' as it came to be called, rotated with the cutting head. There was only a narrow space between the drum and the shield where the power could be applied. This problem was solved by using four hydraulic motors supplied with oil at 13.8 MN/m² (2000 lb/in²) pressure through pipes from a pump situated some distance behind the shield. The four motors drove a peripheral gear ring on the drum through reduction gearboxes, pinions on the ends of these engaging a large gear ring on the drum. With the large velocity ratio thus obtained it was possible to rotate the drum at 4 rpm but even at this modest speed the hydraulic oil had to be pumped at such a high rate of flow that, with the rather hastily designed hydraulic circuit, it became excessively hot. Thus the hydraulic design was working at the limit of its capacity and the advantages of hydraulic motors, that torque and speed are independently variable, were not available.

This so-called 'KM Drum digger' had an overall length of 2.67 m (8 ft 9 ins) (1.1 m, 3 ft 9 ins longer than the hand shield). Of this length the rear-most 2.37 m (7 ft 9 ins) was surrounded by a conventional skin plate (including tail plate) and the front 36 cm (12 ins) were occupied by the cutting head. This consisted of four radial arms spaced at 90°; two of the arms, on a common diameter, were linked by a bridge. Thus the front of the

*The motors were 'Derry-sine' rotary vane type. Motors were supplied by Denison-Derry (now Abex-Denison Ltd).
machine at first glance appeared to carry a large cross. The arms and bridge all carried tines of about 2.5 cm x 2.5 cm (1 ins x 1 in) section at about 15 cm (6 ins) centres. These were mounted in a direction parallel to the axis of the tunnel and, when new, were chisel pointed, the sharp edge being arranged circumferentially, ie in the direction of motion of the tine. No attempt was made to provide rake or clearance as with a coal cutting pick. Hence they rubbed rather than cut kerfs in the face and not only did the tip of the tine wear but most of its shank as well. They needed replacing after about 1.6 km (1 mile) of driving.

The reason for this arrangement of the tines was that, because it was symmetrical, the cutting head could rotate in either direction so that any roll developed by the machine could be corrected. The sides of the arms were shaped so that excavated clay which fell on them tended to be tipped towards the centre of the cutting head under the bridge. Similarly any clay which fell to the bottom of the face was picked up and directed to the centre. Paddles in this part carried the muck backwards through the drum to a belt conveyor.

The thrust of the shield rams bearing on the last ring of lining was transmitted from the frame of the shield to the cutting head by a phospher bronze thrust ring at the front end of the drum. Side forces were carried to two bearing rings at either end of the drum, these revolved on roller attached to the frame. A later version of the machine is shown in Fig 17.

The whole of the drum and its associated paddles, cutting arms, hydraulic motors could, if necessary, have been removed from the shield while the machine was in the tunnel. Thus, according to Cuthbert and Wood, had
running ground entered the face the heading could have been saved and the
drive completed by hand using the shield in the ordinary way.

Auxiliary equipment was carried on a sledge towed by the machine; this
included a 90 kW (120 hp) electric motor driving the pump supplying power
to the drum, another motor and pump for the shield rams and two oil
reservoirs.

The maximum sustained rate of progress on the machine drives was about 110 m
(360 ft) per week or twice that of the hand drives.

2.4.4 Marriage of the tunnelling machine and the groutless lining -

Victoria Line

Between January 1960 and July 1961 a mile length of twin tunnel was built
as a trial for the construction of what is now the Victoria Line. These
tunnels now form part of the line between Finsbury Park and Seven Sisters
stations.

London Transport Board bought two tunnelling machines. There were scaled
up versions of the 'KM drum digger' developed for the Metropolitan Water
Board and were designed and constructed by the same team. The new machines
were 4.27 m (14 ft 0 in) and 3.99 m (13 ft 1 in) diameter to suit two
kinds of experimental lining both to be expanded directly onto the clay.
The larger machine had fourteen shove rams and the smaller one twelve. In
other respects the machines were alike having shields 2.95 m (9 ft 8½ in)
long when the tail plate was detached and drums with an internal diameter
of 2.28 m (7 ft 6 ins) - over half the diameter of either shield.
Figure 17

a Kinnear Moodie drum digger Mk II

b McAlpine centre shaft digger
The four time bearing arms of the prototype were increased to six. One diametrically opposed pair was made to cross the whole diameter by means of a central bridge as before. The muck deflecting paddles were now integrated with the arms. The head extended 0.61 m or two feet beyond the cutting edge of the shield. It rotated at the same speed powered by the same Derry-sine motors with the same reduction gearing arrangement.

The shove rams of 152 mm (6 ins) diameter and 0.913 m (2 ft 8 ins) stroke were supplied with hydraulic oil at a working pressure of 13.8 MN/m² (2000 lb/in²) by a pump driven from a 9.3 kW (12½ hp) electric motor. Power for rotating the drum and cutting head was from another motor of 150 kW (200 hp). These motors, pumps, transformer, muck disposal arrangements etc travelled on a long platform towed by the machine. The hydraulic gear of these machines suffered one of the disadvantages of the smaller prototype in that it became too hot.

The larger machine was used with a concrete lining 23 cm (9 ins) thick (Fig 18a) consisting of fourteen identical segments each two feet long. These left a 18 cm (7 ins) wide gap at the crown which was occupied by a pair of folding wedges made of reinforced concrete; a small hydraulic jack pushed these with a force of 100 kN (10 tons) and produced an expanding face of up to 250 kN (25 tons)* in the ring. The wedges had sufficiently small taper that despite lubrication with bitumen paint they just did not move back after the jacking force had been removed.

The other machine was used with cast iron rings also 0.61 m (2 ft 0 in.) long but with vestigial circumferential flanges of a full depth of 63.5 mm.

*The stress measured in the ring corresponded to a force of only 80 kN (8 tons).
(2\(\frac{3}{4}\) ins) giving an internal diameter of 3.86 m (12 ft 8 ins). The rings were expanded by jacks at each of the knee joints producing thrust of up to 120 kN (12 tons) (Dunton, Kell and Morgan, 1965). Part of the axis level segment was cut away to accommodate the jack. Tapered packing pieces were put in the gap that was made in the knee joint so that the jacks could be removed immediately after expansion of the ring (Figs 18a and 18c).

The use of a fast reliable tunnelling machine with the rapidly erected expanded lining proved a successful synergistic combination. The lining required a well cut circular hole with no overbreak and this could be provided if the excavating–mucking–building cycle was performed rapidly. Soft ground tunnelling made an advance comparable with that produced by P W Barlow's marrying the tunnel shield to the prefabricated tunnel lining.

Both linings were equally successful in making a neat and structurally sound tunnel and the costs were the same. The rate of tunnelling was higher using the iron lining. This was partly because 14 per cent less muck needed to be excavated and partly because one segment weighing 260 kg (5 cwt) and forming one sixth of the ring could be erected by hand regardless of its position in the ring. The concrete segment weighed about the same but only lined 7 per cent of the ring and needed winches and a roller track erector arm because not enough men could get round it to handle it easily. Edmund Nuttall Sons & Co Ltd, the contractor using the iron lining established a record rate of soft ground tunnelling by completing 285 m (934 ft) of tunnel in two weeks – an average speed of 1.07 m (3\(\frac{3}{4}\) ft) per working hour.

2.4.4.1 Hinging cross joints
At the time of the Finsbury Park experimental tunnel, evolutionary progress
Fig 18a Linings used on the experimental length of the Victoria Line
Top: flexibly jointed cast-iron, by Mott, Hay and Anderson
Below: pre-cast concrete, by Sir William Halcrow and Partners
Fig 18b  Concrete linings used on the Victoria Line
Top: Mott, Hay and Anderson design
Below: Sir William Halcrow design
Fig 18c  Details of flexibly jointed cast-iron and concrete linings by Mott, Hay and Anderson
resumed on another aspect of tunnelling in London Clay - the design of the cross joints. The cast iron lining had a simple knuckle joint consisting of convex and concave cylinders of 31.25 mm (1.25 inch) radius machined in the 62.5 mm (2³/₄ ins) wide 0.61 m (2 ft) long thickened end of the casting (see Fig 18c). At first sight (Fig 18a) the concrete lining appears to have a similar knuckle joint, but in fact the radius of the convex part was 199 mm (7.85 ins) and of the concave part 207 mm (8.15 ins). This arrangement retained the advantage of the fully mating knuckle in that it kept segments aligned with respect to each other and to the tunnel axis, while in addition the geometry of the contact area was changed. This was nominally a central line contact. The relative radius of the two radii of the rolling knuckle was 8.4 m (213 ins) so that taking the elasticity of the concrete into consideration a reasonably wide bearing area was obtained. The relative radius had nevertheless to be sufficiently small so that when the ring of segments deformed from a true circle, and angular movement occurred between the segments, the contact patch would not move to the edge of the joint.

When a ring of segments that had been instrumented by the Building Research Station was dismantled, the area of contact, or rather the area covered by those parts that had been in contact at some time during the 2½ years the ring was in the tunnel, could be seen. This was possible because the machining marks on the convex part of the mould were at right angles to those on the concave part. These marks were transferred to the concrete castings which were then painted with bitumen paint. The area of parts that had been in contact was traced out as a reticulate pattern. These showed that generally the area extended the length of the joint and was about 5 cm (2 ins) wide (Fig 19). Sometimes, however, particles of clay
Fig. 19  Segment from experimental length of Victoria Line showing contact patch on rolling knuckle type of cross joint face
had got into the joint during expansion and sometimes the segments had been sufficiently skew to each other for the area to be divided between the two ends. Nevertheless none of these segments spalled. Indeed, the only damage that was noticeable in the tunnel was to the key wedges. Dunton, Kell and Morgan (1965) reported that out of 33,000 segments only 10 were seriously damaged. The concrete was a 5:1 mix using 38 mm (1\frac{1}{2} in.) aggregate that gave 28 day strengths of around 55 MN/m² (8000 lb/in²).

The stresses in the 23 cm (9 ins) thick concrete segments were measured by Building Research Station and found to be only about 3.1 MN/m² (450 lb/in²). Clearly the designers could reduce the thickness. Unfortunately a number of other modifications were introduced for the Sir William Halcrow designed segments that were to be used later on the Victoria Line. The relative radius of the rolling cross joint was regarded as too large for the 15 cm (6 ins) segments and was reduced to 1.59 m (62\frac{1}{2} ins). This was achieved by making both surfaces to a convex radius of 3.175 m (125 ins). Without the knuckle shape to retain the segments concentric to the tunnel, the top segments tended to drop at their leading edges. This resulted in modified point contact instead of modified line contact in the cross joint and unsatisfactory formation of the circumferential joint. The latter was due both to the circumferential joint face not being square to the axis of the tunnel and to the point contacts on the cross joints allowing the segment to rotate about a line radial to the tunnel axis.

The argument the designers used in defence of their adopting the convex-convex cross joint was that self-alignment would not work well enough with a 15 cm (6 ins) wide rolling knuckle and that the rings would be badly built in any case. The author would look for a solution using a more
slippery coating on the cross joint faces than the bitumen that was tried. A coating containing graphite or molybdenum disulphide would be better.

Having decided that bad building was inevitable and that greater relative curvature is needed to keep the contact area within a 15 cm (6 ins) wide joint when it hinges, the problem is that the contact line now moves more for the same radial displacement between segments. The change from convex-concave to convex-convex makes the joint less sensitive to radial displacements (Bubbers and Morgan, 1969, p 382, Fig 3.7) although the same relative radius is used.

The other argument sometimes used against the knuckle joint in concrete is that if errors in design or building caused overstressing, the resulting fractures might occur at the back of the lining and could not be seen.

The design by Messrs Mott, Hay and Anderson for the iron lining used in the experimental length of tunnel at Finsbury Park was first used as part of shield driven 4.27 m (14 ft) internal diameter cooling water tunnel at Belvedere Generating Station on the Kent bank of the Thames in 1958. It was modified slightly for the 3.86 m (12 ft 8 ins) tunnel as Finsbury Park and was used for part of the later Victoria Line. Two factors contributed to its eventual disuse. Both consultants designed bolted cast iron linings which could be used inside a tail plate if bad ground should be encountered. The one that was made to be interchangeable with the 152 mm (6 ins) thick concrete lining was 3.99 m (13 ft 1 in.) O.D. to fit inside the tail plate, and had a flange 76 mm (3 ins) deep and a skin 25 mm (1 inch) thick giving an internal diameter of 3.83 m (12 ft 7 ins). This was used in the over-breaking ground at Brixton (see Chapter 3). That for the expanding
12 ft 8 ins iron was 3.91 m (12 ft 10 ins) O.D., had a flange of 79 mm (3⅛ ins) deep and a skin with the traditional thickness of 22 mm (⅞ in.) giving an internal diameter of 3.71 m (12 ft 2 ins). At this time London Transport Board decided that 3.81 m (12 ft 6 ins) should be the smallest diameter on the Victoria Line. If the diameter of the expansible lining were increased to make the contingency lining acceptable its advantage of thinness would be lost. Also thinner concrete lining was now shown to be safe and the price of iron was increasing so that the iron lining became relatively uneconomic.

The expansible iron lining was redesigned in concrete and used on the Victoria Line (Fig 18b). A ring was 0.61 m (2 ft 0 in.) long, had an internal diameter of 3.81 m (12 ft 6 ins) and an external diameter of 4.14 m (13 ft 6 ins). The twelve 152 mm (5 ins) thick segments had knuckle cross joints of 102 mm (4 ins) radius. The feather edge of the concave half was taken off so that the two halves mated over about 60°. There was some steel reinforcement under the concave half. The joint hinged satisfactorily and spalling did not occur.

2.4.4.2 Methods of expanding the rings

One of the difficulties to be overcome by the designer of an expansible tunnel lining is that the size of the hole depends on a number of factors: time, swelling property of the clay, stress in the ground etc. Time is the dominant variable for any one job because the machine may be driving at maximum speed or it may be unservicable for days. The wedge-block lining can be supplied with 'key' segments of various sizes but the fine adjustment is by the distance the wedge shaped key is pushed and for this to be varied

*Except at the invert joint which was radial.*

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the key must be shorter than the rest of the ring. This leaves a gap which is filled by a hessian bag of dry mortar dipped by the moving key. A similar facility was provided by the folding wedges of the 229 mm (9 ins) thick experimental lining used at Finsbury Park except that here the reinforced concrete wedges often broke under the determined effort of the miners to 'get the key in'. In view of the complicated shape of these wedges which had to fit the convex and concave surfaces of the segments, cast iron would have been a better material.

Perhaps because of the untidy look presented by the crown of these otherwise near-perfect tunnels, the linings used on the Victoria Line reverted to expansion at the knee joint as in the Potters Bar and Hadley Wood tunnels.

The Mott, Hay and Anderson design in concrete used the same principle that had been used in their expanded cast iron lining so that the jack was immediately available for re-use. Tapered concrete packing pieces bore on a slanting plane surface on the pocketed segment and on a 'half-round' piece of cast iron filling the concave cross joint surface of the segment below (see Fig 18c). Thus the joint could still hinge during the early life of the tunnel before the filler block was mortared into the jacking pocket. No spalling occurred.

The concrete lining finally designed by Sir William Halcrow and Partners for the Victoria Line was also expanded at the two knee joints but in order to avoid filling gaps with mortar, a packing piece with radial cross joints having the same 3.175 m (125 ins) radius rocker was inserted into each knee joint from the front of the ring. To provide for variation in the size of the excavation, the nominally 203.2 mm (8 ins) wide packer was made in
15 sizes varying in 1.6 mm (1/16 in.) increments between 188.9 mm (7/16 ins) wide and 211.1 mm (85/16 ins) wide. The jacks had to bridge across these packers, and claws attached to them entered pockets in the adjacent segments. The claws only bore on the concrete at the bottom of pockets so that the thrust would be on the centre line of the ring. The intended procedure at this stage was to pressurise the pairs of jacks at the two knee joints to give a thrust in the ring of 350 kN (35 tons), select the appropriate sized packers and then let the load onto the packers, losing 50 kN (5 tons) per 1.6 mm (1/16 in.) contraction of the circumference. In the first stage the arch of the ring was to be suspended on the two pairs of jacks. If the difference in thrust between the two pairs exceeded the friction of the lining against the clay the lining would move as one jack opened and the other closed. This in fact happened and the miners modified the building procedure so that the equipment provided could be used.

At Potters Bar the friction around the ring was large and the jacking loads were smaller. At this time the idea seems to have been less to prestress the concrete against the ground than to make it a good fit in lieu of grouting. The wedge shaped packers in the Mott, Hay and Anderson system could be fed in to follow the growing gaps as the ring was jacked. With four packers each, the gaps could easily be kept equal and even if the wrong sized packers had been chosen initially, coarse adjustment could be made by changing them.

The load in three instrumented rings in a section of Halcrow lining was measured and found to be 120 kN (12 tons) rather than the 360 kN (36 tons) indicated by the Bourdon gauges in the oil line to the jacks. The rams of the jacks were eccentrically loaded which would have introduced additional
friction. Also having four hydraulically independent jacks in the expanding system, when the double convex cross joints already gave the ring an unusual amount of freedom, resulted in badly built rings.

For erecting the concrete lining for the Piccadilly Line extension tunnel between Hatton Cross and Heathrow stations the segments were built on a former hoop similar to that used to improve the shape of the ring when the Denseg was first tried. The segments of the upper half of the ring were temporarily secured to the former by the holes that are cast in the segments for the later attachment of cable brackets etc with expanding dowels. In this way a good shape is given to the half ring before it is offered up to the clay surface. The invert of the ring is easier to build correctly.

This new lining, of 3.81 m (12 ft 6 ins) diameter and 15 cm (6 ins) thickness designed by Sir William Halcrow and Partners consists of 22 segments. The cross joint faces have a radius of 3 m (118 ins) in the plane of the ring and a radius of 13 m (512 ins) in the plane at right angles. One imagines that this would suffer from the same building faults as the Halcrow Victoria Line ring, but to a greater extent, if it were not built with the help of the former ring. However, having decided to use a former ring at all, no doubt the increased articulation caused no further trouble. The advantage of a segment which only has an arc of 18° and which has cleanly shaped joint faces lies in potential ease of manufacture. The contractor John Mowlem opted to press the segments. The only modification to the segment that this required was to make the three dowel holes parallel to each other, ie the two end ones could no longer be strictly radial. The

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*A former was used for the London Airport Cargo tunnel but there the primary objective was instant support. Nevertheless, it probably made it easier to build well and in the present case the rapid support is an additional bonus.*
Fielding and Platt concrete press required rather expensive moulds but
because it could produce segments at the rate of one a minute and they are
used in the tunnel at the rate of about one every five minutes, the
necessary number of conventional moulds would have cost about the same.
An additional factor in favour of the press is that conventional moulds
often suffer damage at the hands of the low grade labour used in demoulding.

The 0.6 m (2 ft) long ring is built up from four different kinds of segment.
The invert consists of five segments of the basic type; then, at each knee
joint, there are three segments by which the ring is expanded. These three
are used in exactly the same way as the 'key' and two 'top segments' of the
Wedge-block lining. The wedge shaped key is narrower, about 20 cm (8 ins)
at the wide end, but the taper is the same. Eleven more of the basic type
of segment complete the ring. The knee joints would be the best place to
expand the ring in view of the fact that the upper part of the ring is
pre-assembled on the former hoop.

The rate of progress with this lining and a machine yet to be described was
on average 120 rings (73 m) (240 ft) per week. The finished tunnel does
not look so neat as, and the cross joints show considerably more roll than,
the ex-Victoria Line, Mott, Hay and Anderson, lining in the twin tunnel.
In a railway tunnel errors in roll are a nuisance because they make it more
difficult to lay cables in straight lines; cable brackets have to be made
slotted and then adjusted in the tunnel.

The argument in favour of using the double curve on both halves of the cross
joint is that with short segments the ring is very likely to be built so
that the circumferential joint faces of the segments do not lie in one plane;
then if the joint were single convex to single convex, point contact would still occur at the ends.

2.5 MODERN TUNNELLING MACHINES

2.5.1 Victoria Line machines

For the Victoria Line between Victoria and Walthamstow (Hoe Street) (excluding the experimental length completed in 1962) London Transport Board bought some twenty shields of various sizes. Eight were shielded tunnelling machines. Two were the 'MH drum diggers' from the experimental length reconstructed as 'MK I machines'. Two more, called Mark II machines, employed a new slow speed, high torque, hydraulic motor. This was the Staffa radial motor made by Chamberlain Industries and developed with the help of Central Engineering Establishment (now the Mining Research and Development Establishment) of the National Coal Board. In about 1956 their Mr II Monk redesigned the original Staffa motor as a monobloc unit which could work at high pressures and slow speed. The power to size ratio now attained enabled the motor to replace the geared pneumatic motors, used with coal conveyors in confined spaces. Since then over 50,000 of these motors have been made and their application has spread to augers, winches, dredgers, cranes etc. The Kinnear Hoodie Mark II drum digger employed four of the five cylinder motors then rated at 50 hp each, pinions on the motor shafts directly engaged a gear ring on the drum giving a reduction in speed of a tenth. The drum was usually rotated at 4 rpm (see Fig 17).

The four other tunnelling machines were built by Sir Robert McAlpine and Sons and also employed Staffa hydraulic motors. These were larger units having seven cylinders and were rated at 44.7 kN (60 hp). At 13.8 m/s² (2000 lb/in²) the torque available from the unit was 9 kN/m (6650 lb/ft)
compared to 6.25 kl/m (4600 lb/ft) from the smaller unit (Engineering, 1959, vol 186, pp 421 and 569).

Four of these motors, which could be used at lower speeds than the five cylinder version, bore pinions which drove a large pin wheel at the rear of the cutting head, the reduction being 3:20 (see Fig 17).

The McAlpine machine had originated as a 5.33 m (17 ft 6 ins) O.D. mechanised shield complete with tail plate and used with a 4.9 m (16 ft) I.D. cast iron lining in varved clay and glacial till for driving tunnels forming part of the Toronto subway (Bartlett, Koskieicz and Ramsay, 1965).

A simple modification was tried here to overcome the difficulty of changing beads on the cutting edge whilst determining the optimum clearance over the shield skin. An edge tooth cutter was put on the cutting head so that it could cut 6 mm (1/4 in.) or so beyond the skin. The bottom quadrant of the skin bore a sole plate 6 mm (1/4 in) thick so that the shield did not 'dive'. When the overcutting method of providing skin plate clearance was tried on the Victoria Line the machine became 'directionally unstable' (Clark et al, 1969).

There are two aspects to the steering problem, both of which are relevant to the change from Toronto and London, the shield and the lining. A shield is steered by using only, or mostly, those shove rams which are on the outside of the intended curve. The ram thrust is resisted by the forces on the cutting edge and the resultant of these lies on, or approximately on, the axis of the shield. This resultant force is not in line with that of the chosen group of rams so a couple is produced which turns the shield. If the clay ahead of the shield has already been cut by an oversized cutting
head, the resistance of the cutting edge, and hence the couple, is small. It may be too small to overcome the friction between the skin and clay and between the ram shoes and the lining. If the hole is undercut the necessary resistance is available just as with a hand shield. The other aspect is steering both the shield and the tunnel itself by inserting packing in the circumferential joints of the lining. The cast iron lining at Toronto had corrections to line and level "made with machined tapered rings, with 1 inch taper on the 17 ft external diameter. These rings had been surprisingly cheap" (Bartlett et al). Wood packing introduces steering difficulties because of its compressibility. The side which is packed moves more when the shield ram pressures are exerted, both because there is more packing and because more thrust is used on the side the miner wishes to advance. Also, movement is not confined to the last ring of packing.

There was also some anxiety lest the edge of the sole plate should cut a step in the clay and cause difficulties in the use of an expanded lining. Hence the final decision was to undercut by 36 mm (3 ins) and trim to size with the cutting edge fitted with a bead in the normal way.

The McAlpine machine not only had more powerful motors than the 12 ft Kinnear Hoodie machine but it had shield rams of 216 mm (8½ ins) bore compared to 178 mm (7 ins). The number and arrangement of these depended on the lining with which they were used. The arrangements shown in Fig 17 were both suitable for either the 3.81 m (12 ft 6 ins) 12-segment concrete lining designed by Hett, Hay and Anderson or the 3.86 m (12 ft 7 ins) cast iron lining of 6 segments and a key. Also there was enough power to spare to justify a more sophisticated hydraulic circuit which linked the
showing rate with the cutting rate. This had been tried on the first Kinnear Hoodie machine but was abandoned. Shoving and digging were balanced by hand when the Kinnear Hoodie machine was used on the Victoria Line.

The cutting head of the McAlpine machine was in the form of a cross, bearing chisel pointed picks of rather stouter section than those on the Kinnear Hoodie machine. A boss at the centre was studded with picks. The head was carried on a shaft mounted on bearings in stout horizontal beams crossing the shield. The conveyor occupied a position above the shaft hence the muck had to be lifted to the top of the machine and then passed back to the conveyor. This was achieved by making the periphery of the head in the form of two frusta of steel metal cones arranged concentrically. The annular space between them was divided by radial plates. Muck which fell on the inner cone at its lowest part slid forward to the face and then back in to the annular space (labelled 'annular pick up buckets' in Fig 17). It was then carried up to the top of the machine where the segment of 'bucket' was upside down. It was then free to fall down onto the other side of the inner cone which deflected it backwards onto the conveyor.

Early experience with the Kinnear Hoodie machine (Cuthbert and Wood, 1962) had been that it overbroke the ground more than did hand shields. This was thought to be because the cutting head protruded ahead of the cutting edge and could transmit shear forces to the surrounding ground. Hence the McAlpine and subsequent machines have been designed with the cutting head set back in the cutting edge. The results confirm the views of Cuthbert and Wood. This was achieved with very little increase in the length of the shield skin plate and resulted in superior manoeuvrability of the McAlpine machine.
The difference in design philosophy of the two machines gave rise to a difference in approach to maintenance. The drum digger had its working parts shielded from the muck; the greater difficulty of servicing it was offset by its lesser need for attention. The McAlpine machine, particularly the pin wheel, was vulnerable to abrasion by the silty clay but could be cleaned easily. The centre shaft bearing could be easily greased, the ingoing grease expelling any dirt that had entered. These difficulties were illustrated when by chance, or mistake, it happened that one of the McAlpine machines was used on a Kinnear Moodie contract. The shaft bearing only just lasted the drive.

Nevertheless, both designs are proved to be basically sound by the fact that machines of both types are still used and differ only in detail from the 1962 versions. Recently (summer 1972) a centre shaft machine was used for a 2.54 m (100 ins) tunnel in the London Clay at Southend in contravention of the idea that first brought hydraulic motors to tunnelling machines. This was achieved by reducing the depth of the main cross beam and putting a double crank in it so that the part holding the bearings is surrounded by the cutting head. The shaft no longer restricted the placing of the conveyor.

Access to the face was difficult in both machines. It was more dangerous to be at the face in the KM machine because two feet of ground were not only unsupported but more disturbed. If the clay was wet and sticky, as under the Regent's Canal and under the River Lee, it was necessary to go forward to clean the muck deflecting paddles which the accreted layer of clay made the wrong shape, or, in the other machine, to avoid the danger of the outer cone binding on the shield skin.
In a hand shield it is usual to timber the face when it is left unattended. With the difficulty of access in these machines, together with the awkward shape of the spaces between the arms, this precaution was neglected, it being hoped that the arms themselves would offer some support. On two occasions runs of water bearing ballast occurred and digger shields were buried for months, whereas when a run occurred during a hand drive the face was recovered and suitable action taken fairly quickly (Morgan and Lubbers, 1969).

It has been frequently suggested that minors should investigate potentially bad ground by boring ahead and upwards from the face, but again this would have been difficult from either machine. Moreover, there are few things that can be done if, say, a very shallow cover of clay is found. The practical choices seem to be the extreme ones of putting the tunnel under compressed air or carrying on as fast as possible so that weaknesses in the clay do not have time to develop. This would mean not stopping for weekends, not doing any maintenance nor even any further probing ahead. The conclusion reached by London Transport Board was that these machines were not sufficiently versatile to use in ground where buried channels might exist and when the Victoria Line was extended south of the river they were not used.

The realisation of this problem has led to an evolutionary branching in the design of tunnelling machinery. For tunnels with a large depth/diameter ratio, powerful, fast, full face machines have continued the line of development from the Price machine through the '4½ drum digger' to (for instance) the Uttall-Priestley machine (see later). On the other hand where machines are required for relatively shallow tunnels, there has been
a return to the Thompson concept of making the shield and excavator easily separable.

2.5.2 Modern machines for shallow tunnels

Machines for the Piccadilly line extension under London Airport and for the Fleet line fall into this category. The shield continues to cut the edge of the hole and in the case of an expandable lining to form the shape to which the segments are expanded. It also tows the sledge carrying the usual power packs, muck disposal gear etc and also has space for the now mobile excavator. Segment erecting gear (see page 107) is also closely associated with the shield and can be designed so that, when loaded with the segments which are to form the top half of the ring, it provides protection against a 'back run' to which the tailless shield is vulnerable.

Moving the excavator back onto the sledge also gives the designer a chance to make the shield shorter which, in addition to making it easier to steer, will later be shown to be advantageous in connection with supporting the clay. The Priestley shields for the Piccadilly line are 2.43 m (8 ft) long whereas the McAlpine digger shield of the same circumference is 3.05 m (10 ft) long.

Two different excavators are in use in the twin tunnels at Heathrow. The tunnel for the eastbound road is driven with an Anderson-Mavor 'road header'. This machine was designed for excavating rock. Its basic component is a boom, trunnion-mounted on the axial line of the tunnel, and capable of being moved through a solid angle large enough to allow the forward end to reach any part of the face. The boom is restricted from moving beyond this angle by the circular hole in the diaphragm of the shield which has
been modified with this requirement in mind. The boom is moved by hydraulic rams. On the forward end of the boom is a cutting head about 1 m (3 ft 4 in) diameter which is rotated at 30 to 50 rpm in one direction by a shaft running along the boom. The boom and shaft are telescopic so that, although the face cut by the rotating head is normally a segment of sphere, a plane surface could be cut. The shaft is powered by a 44.7 kW (60 hp) electric motor at the other end of the boom behind the trunnion mounting so that it counterbalances the head. An electric motor is preferable to an hydraulic one because the small heavily built head is robust enough to withstand shock loads and shock loads are needed to break clay stones. The electric motor itself has some inertia and when momentarily overloaded meets the high power demand with a high current which is acceptable for a short time. The hydraulic motor on the other hand has low inertia and stalls when overloaded. The frame on which the road header is mounted has gripping pads which can be jacked out against the completed tunnel lining to steady it. The muck which is cut off the face falls onto a forward sloping pan in the bottom of the shield. Two sets of paddles, arranged like the spokes of a wheel with no rim, mesh and counter-rotate in a plane just above the pan. These convey the muck up the slope of the pan and over its rear edge. It then drops into a conventional belt conveyor which transports it to shuttle cars.

The small cutting head requires only a small torque reaction so, without the necessity to provide for the reverse rotation required of the full face cutters, the picks may be correctly angled. Hence the power required for this machine is only one quarter of that for the McAlpine machine. Also the picks only wear on the tips which are designed for cutting, having a coating of tungsten carbide. The replacement rate is only three or four
picks per 300 m (1000 ft) drive.

The machine in the 'westbound' tunnel is the Hisladorachs made by Westfalia Linnen. This is a scraper-conveyor mounted on a track laying vehicle in such a way that its end can move over the face in a manner similar to that described above. The machine is not so well married to the shield and one imagines more skill is required of the operator.

Average progress with these machines and shields has been 120 rings of 0.6 m (2 ft) size per week. Occasionally a rate of 160 rings per week has been attained.

A recent (1969) solution to the problem of using a mechanised shield near the clay-gravel interface, which is suitable for small tunnels, is that of Streeters of Godalming Ltd (Mazzotti, 1973). Their so-called clay-armed digger shield has the cutting edge divided into two semi-circular pieces and attached to the rest of the shield by pivots at the ends of the semi-circles. These may rotate about vertical axes until they meet on the centre line of the shield, sweeping out a dome-shaped surface as they do so. They are much more robust than the normal cutting edge and each one has a 230 kN (23 tons) hydraulic jack attached at axis level which reacts against the inside of the skin plate, also at axis level. In bad ground the two halves are in the normal cutting edge position or 'open' and there is plenty of space between them to work the face with poling boards. When good ground is met the two halves are 'closed' with a clawing or grabbing motion so that they bite out a hemispherical face. The broken material is then disposed of by hand shovelling or mechanical conveyor according to the size of the tunnel. The cutting edge halves are then retracted, the shield
shoved a little and the cycle repeated. In practice mucking can be carried on continuously while the excavation cycle is proceeding.

Since 1969 six machines have been built for use with 1.5 (5 ft) I.D. lining, one for 1.67 m (5 ft 6 ins) and one for 2.2 m (7 ft 3 ins). In the London and Hampshire Basins they have been used for flood relief schemes in the Roding Valley, also at Chertsey, Farnborough and Havant. Overall average progress on the Roding Valley scheme has been 124 m (407 ft) per week.

Also with this machine very little power is expended in cutting the face. Indeed, a saving of 90 per cent is claimed. Presumably this is because the muck can be broken off in very large pieces.

2.5.3 Modern machines and lining for deep tunnels

Improvements in hydraulics enabled motors to be run at increasingly high pressures and by 1969 the Nuttall Priestley machine, which superficially resembles the Kinnear Moodie 'Drum Digger', could be powered by four Chamberlain B200 motors each capable of providing a torque of 10 kN.m (7350 lb/ft) when working at a pressure of 20.7 M.N./m² (3000 lb/in²). The 2.82 m (111 ins) O.D. machine, used in 1969-70 for the 19.75 km (12 miles 2 furlongs 40 yards) long tunnelled section of the Ely Cut to Essex aqueduct in Gault clay with a Wedge-block lining, broke the world record for soft ground tunnelling by driving 434.6 m (475 yds 10 ins) in one week.

The average speed was about 183 m (200 yds) per week using 0.54 m (24 ins) long rings. These high speeds reflect: (i) the noteworthy reliability

\*The firms involved were Edmund Nuttall and Sons (London) Ltd civil engineering contractors; Robert L Priestley Ltd of Gravesend, Kent, tunnel shield and tunnelling machine manufacturers; Chamberlain Industries Ltd of Leyton, London, manufacturer of hydraulic pumps and Staffa motors.
of the machine; (ii) the ergonomic design of systems for passing the
segments to the face and bringing the muck away from it (all carried on
a sledge towed by the machine); and (iii) the rapidity with which the
Wedge-block lining could be erected. With practice the miners could erect
a ring in two and a half minutes, including the time to lubricate the clay
with soft soap or bentonite slurry. Shoving the shield and mucking for
the next ring took six to seven minutes. Removal and replacing of the
spoil conveyor from the face took only seconds. Nevertheless, minor
contingencies usually increased the time for the whole cycle to 15 minutes.
Nevertheless, the average progress was around 30 rings per 12 hour shift
at the time the author saw the work. The discrepancy is due to a delay
which even the careful planning of Binnie and Partners and Edmund Battall
had failed to foresee. Nearing the end of a long drive the clay had
become more closely fissured and perhaps the cutting tools had become
somewhat worn. This resulted in the size of the lumps decreasing to about
that of a fist and the consequent bulking of the broken clay required the
addition of another skip to the muck train. Because very large skips had
been chosen the weight of the train became too much for the electric
locomotive to haul on the gradient on this section. Thus extra journeys
were required and the machine could not be kept working fully.

The Wedge Block lining used for the northern-most 14081 m (8 miles 6 fur-
longs) of the Ely Ouse to Essex aqueduct tunnel, where the overburden
varies from 28 to 45 m (92 to 147 ft), is 14 cm (5½ ins thick). For the
southern-most 5669 m (3 miles 4½ furlongs) where the overburden varies
between 45 and 80 m (147 to 263 ft) the thickness was increased to 18 cm
(7 ins). The Wedge block itself was thrust with a force of 330 kN
(33 tons) for the normal lining and 420 kN (42 tons) for the thick one.
For the Metropolitan Water Board's Southern Tunnel Main, a treated water main some 20 km (12 miles) long connecting the treatment works at Hampton-on-Thames and Walton-on-Thames to Merton, the Wedge Block lining was 14 cm (5½ ins) thick. This lining has been improved in detail since it was first conceived and is now used for nearly all tunnels for the MWB which has the segments made (mostly a standard size of 2.54 m (100 ins) ID) by Charcon tunnels, formerly Kinnear Hoodie (Concrete) Ltd, under a licence agreement (based on Tattersall, 1955). To ensure accurate reproduction, cast iron master segments have been made. Also under a licence agreement MWB will provide 'know how' to contractors or consultants wishing to use the system and will prepare a set of contract documents. By these means the expertise that MWB has acquired since 1952 does not become diluted by uninformed imitation.

The 2.54 m (100 ins) ID lining is nominally 140 mm (5½ ins) thick and a ring is 68.6 cm (27 ins) long and consists of twelve segments. Nine of these are identical and two are shaped to fit the tapered key which has cross joint faces like a Donseg with slightly helical surfaces. The key is 5 cm (2 ins) shorter than the ring in order to provide tolerance on finished circumference. The gap is filled by nipping a bag of dry mortar between the newly driven key and the previous one. The shield and ring are matched either by trying various sizes of key (as recommended by Tattersall) or by varying the bead on the shield of the machine. The original Kinner Hoodie drum digger, rebuilt for driving the Hampton end of the Southern Tunnel Main, was fitted, after several trials, with a 3 mm (⅛ in.) bead extending round the top 180°. The end sections of bead were faired into the shield cutting edge.
The cross joints of the ordinary segments are radial and nearly plane, having only $0.5 \text{ mm (0.020 in.)}$ rocker. If the surface is truly cylindrical, this corresponds to a radius of $9.6 \text{ m (378 ins)}$. They are coated with a compliant compound which increases the area of contact and so lowers the stress. The sliding surfaces of the key and top segments are coated with a compound containing graphite to lubricate them. The clay surface is lubricated with soft soap when the ring is expanded. For the Southern Tunnel Main a hoop thrust of $140 \text{ kN (14 tons)}$ was obtained by pushing the key with a force of $400 \text{ kN (40 tons)}$. No spalling was seen at the axis level cross joint or any other joints. In general a squat of about $3 \text{ mm (1/8 in.)}$ had developed in about six months.

Much higher loads were jacked into the rings near shafts by a method of repeated jacking. If the load was very high, say equivalent to that of the full overburden acting hydrostatically, $\gamma h$, it dissipated with time to about 60 per cent of its highest value. M
B engineers were unable to detect any squat at all, in the rings put in with a high initial load.

The Southern Tunnel Main traverses the Malden to Deptford fault. A change in lithology was noted (the fault has a throw of about $46 \text{ m (150 ft)}$) but the actual fault surface had not caused any shattering or softening of the clay to affect the tunnelling.

The Kinnear Hoodie machine working from Hampton was regularly progressing $257 \text{ m (845 ft) per week (75 of the longer, 27 ins, rings per day)}$; the maximum progress was $276 \text{ m (900 ft)}$ in one week. The more modern Huttab-Priestley machine at the other end of the tunnel was performing even better.

The Huttab-Priestley machine was also used in London Clay with the Wedge-
block lining for the 'Three Valleys' aqueduct. The latter is a raw water tunnel shared by the Colne Valley, the Rickmansworth and Uxbridge Valley and the Lee Valley Water Companies which calculate that they will need an extra 320 ML/day (70 million gallons per day) of treated water by 1999.

The 6900 m (4 miles 2 furlongs 64 yards) long tunnel runs from Sunnymeads on the Thames upstream of Old Windsor Lock to a treatment works at Iver where the Paddington to Bristol railway crosses the river Colne. The tunnelling machine, muck handling, segment transfer and sledge were all those used on the Ely Ouse to Essex tunnel (Collins, 1972). The cutting tools, although symmetrical with respect to the direction of rotation, were 'T' shaped and provided with rake and clearance so that they peeled off the clay rather than just knocking it. The average cover was 20 m (65 ft) and the tunnel crosses over two EWB tunnels 15 m (50 ft) lower down. The tendency for overbreak that EWB experienced with a Kinnor Moodie machine when driving these particular tunnels was not found with the Nuttall-Priestley machine, probably because the tools really did cut through clay and so transmitted less torque to the face as a whole. Also the cutting head does not protrude so far from the cutting edge of the shield. A 9.5 mm (3/8 in.) bead was used. The cutting tools were carried on four arms made up as a cross. Between the arms of the cross were paddles to deflect the excavated clay to the centre and directly onto the main conveyor which can be extended right through the inner drum (see Fig. ). This inner drum does not rotate as did that of the original KM drum digger. All the bearings and gear rings are behind the fixed inner drum and only the head carrying the cutting arms and muck deflecting paddles rotates. Thus there is no risk of the conveyor catching a revolving part and being twisted. No dimensions are given, Priestley's claim that the inner drum is larger than that of the Kinnear Moodie machine
of the same size and access to the face is better.

For about 500 m (550 yds) before the Iver shaft the machine ran into a particularly wet layer of claystones. The water softened the clay and caused it to stick onto and along the cutting head and the conveyor. With a slower rate of progress and a greater tendency for the clay to swell, 30 per cent more pressure was required to drive the key blocks but no damage was caused (Collins, 1972).

The tunnel was driven in 8 months; despite many (drier) layers of claystones an average rate of 212 m (700 ft) per week was achieved in the machine drive. This rate is comparable with the average rate achieved in the fault where there were no claystones.

2.6 THE IMMEDIATE FUTURE FOR EXPANSIBLE CONCRETE TUNNEL LININGS

The author has not walked the whole length of the Victoria Line but of the parts he has seen, the Halcrow designed 152 mm (6 ins) thick concrete lining was the most damaged.* The damage was mostly spalled edges and chipped corners; occasionally a segment had a crack right across it parallel to the tunnel axis. This last fault is the most difficult to see, it is also the least dangerous amounting to no more than an extra cross joint. The spalling was potentially more dangerous because a train could hit a falling piece of concrete. However, the time taken to build the railway once the tunnel was driven was over two years† and in this time the

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* But see reservations about attributing causes to damage seen, page 127.

† Work on the main contracts was started in March 1963, the last running tunnel drive (north of Victoria) was completed at Kings Cross in September 1966. The line was open for passenger traffic between Vihthamstow and Highbury on 1 September 1968 and through to Victoria on 7 March 1969 (Follenfant, 1969).
earth movements which could precipitate further spalling of the damaged segments have virtually ceased.

To what extent should the damage in a particular tunnel be used as criterion for judging the success of that particular lining? Partly this depends on when the damage occurred, either:

1. In transport between factory and site
2. In transport between shaft top and face
3. During erection
4. After erection.

Probably the designer considers these factors in the reverse order.

The concrete lining which appeared to be least vulnerable to damage during building was the Halcrow lining having segments which are convex at both ends. The damage that it in fact suffered was probably not inevitable. One cause was the use of the hydraulic power to place the heavy segments; if the segment did not fall neatly into place in the ring it would be forced and consequently damaged. In the author's opinion the self-positioning knuckle joint of the Mott, Hay and Anderson prevents this kind of damage to that lining. One is puzzled that, once the trouble in erecting the Halcrow lining was discovered, steps were not taken to rectify it. For instance, the controls of the hydraulic gear could have been provided with 'feel' or the supervision of the work could have been improved.

In seeking an answer to these questions or the question of whether the same design of lining suffered the same damage on different drives, one is led to consider the organisation of this industry. The parties involved on the Victoria Line were the client, London Transport Board, two consulting
firms of the highest repute and experience in soft ground tunnelling,
Sir William Halcrow and Partners and Messrs Mott, Hay and Anderson, nine
tunnelling contractors, four manufacturers of lining, two manufacturers of
tunnelling machines of running-tunnel size and three manufacturers of
Greathead shields of various sizes. To this should be added the face
gangs themselves for although nominally part of the contractors' organisa-
tions they often worked as autonomous units and changed employers at short
notice.

A face is usually worked by three gangs working one stint in each 24 hour
period for a five day week, the weekend being used by the contractors for
maintenance and the engineers for surveying. The gang is paid for each
ring of tunnel completed, so that it has an interest in speed and in not
leaving any work partly completed. Thus, at best, a nominally eight hour
shift is rounded down to the time to complete a whole number of rings.
With a fast machine the leading miner may take the gang out earlier if he
feels that fatigue will cause the next ring to be built slowly or danger-
ously. If circumstances prevent the men from working, payment is negotiated;
hence the gang has little interest in caring for the machinery which can be
tested severely unless the contractors' pit bosses or foremen are both con-
scientious and vigilant.

Segments are occasionally damaged during their transport to the face through
derailments due to a bad road. One does not see why a contractor should
tolerate this erosion of his own efficiency when the whole route is designed
and controlled by him.

The arrangements for off-loading at the face on the other hand, often have
to be devised at the start of the job to suit a tunnelling machine supplied by one manufacturer and a lining supplied by another and to be used by men who have seen neither before. The gear has to be fitted in with a variety of hydraulic pumps, transformers, cable troughs and muck disposal arrangements. Once the job has started there is usually little opportunity to modify the arrangements.

The traditional iron segment comprising one sixth of a circle was usually winched off the bogie train on its back and because of its upturned ends usually toboggan across the work space sustaining only glancing blows on the way to its place in the ring. The modern concrete segment weighing about the same but of half the length tends to dig its corners into soft timber or have them knocked off by hard machinery. It is easily moved on roller tracks but can get out of control on slopes and collide with other segments causing damage. It is best moved hanging from a mono-rail.

Damage in transport to the site is not rare, though, because it is clearly the carrier's responsibility, the knowledge that the segments will be inspected should deter carelessness. The risk to iron segments is greater because they are more likely to be brought long distances.

The client is represented at the site by the consultant's resident engineer and his staff and by the client's own inspector. At the face for most of the time only the latter is present, because those of the RB's staff who are in the tunnel have gone there to perform specific duties and these are often hindered by the other work, leaving little opportunity for other interests. Therefore, if the RB is to exercise any supervisory function as the rings are actually built he or his staff must make special visits.
This will strengthen the moral position of the inspector who will be more willing to act early and often. If by some lucky accident the relations between the RE and the contractor's agent, between the inspector and the contractor's men and so on are all harmonious, then these visits will not be taken amiss; the job will go well and a by-product will be that the lining and the associated machinery will be used as their designers intended. On the other hand the personalities involved may confine themselves to their strictly contractual relations. Any gross faults will be put right without anyone being regarded as having neglected his duty. Faults of marginal significance once incorporated in the work will not be put right. This is obviously unsatisfactory from the point of view of that particular client; from the viewpoint of the author trying to make an assessment of various linings it provides a trap. If he only sees the finished tunnel he does not know whether a good tunnel could have been built with that lining or whether the design is such that blemishes are certain to appear. He is only on sure ground with tunnels he has seen built.

These remarks hardly apply to the older, iron-lined, running tunnels because it is much easier to bolt six segments together to form a true circle than to build one from twelve or so concrete voussoirs. If no shield is used the shape may not be so well held though. Cracked flanges are the most common failure in iron linings. These may not show up until rust stains appear. If they are manufacturing faults they usually contain bitumen.

Not all tunnels are built with the division of responsibility described above, but where public accountability is involved such a system is likely. Assuming that the designer cannot alter the system what are the lessons for him?
(i) Firstly that the way a particular design of lining was treated on one contract is little guide to how it will fare on the next because there are too many variables.

(ii) He should seek to be allowed to design for particular machines and handling gear even if he cannot choose the organisations that use them.

(iii) He should make a design which is easier to use in the correct way than the wrong way.

(iv) Lastly, he may consider sacrificing qualities which will only be of avail if the lining is perfectly built in order to provide some provision against misuse, eg impact resistance may be more important than a good finish. A material that is weak but gives clear evidence of failure is to be preferred to a stronger one that half fails.

2.7 EVOLUTION CONTINUES

The subject of this chapter has been called evolution because it describes how the methods of tunnelling in London Clay have changed and it will have been noticed that the more outstanding changes have occurred in response to influences outside engineering and technology. This is saying more than that 'necessity is the mother of invention'. It is rather that unless an invention is mothered it will die in infancy.

There is a remarkably close parallel between this engineering evolution and biological evolution which, although it was often in the writer's thoughts, has not been discussed in this already rather long story. In concluding this chapter it is however necessary to stress that this sort of engineering will continue to be a response to human needs and economic pressures; but
whereas the story started during the industrial revolution when new sources of raw materials and energy were being unearthed, we are now at the beginning of a biological revolution and at time when the population is about to, or in some parts of the world, already has, become out of balance with renewable resources and is running out of the finite ones. This will bring changes in what is demanded of the civil engineer which are unlikely to be the kind already experienced. The only certainty is that extrapolation from the present situation will prove an unreliable method of forecasting. The science-fiction that resulted from following numerous possible trends has therefore been omitted.

Evolution is concerned with success and progress. These are not necessarily dependent on each other. An evolutionarily successful form is one that occurs in large numbers and continues to do so for a long time. It does not have to be very advanced - just sufficiently advanced to have an edge on its competitors. Since tunnels do not die a suitable measure of success is the mere mileage of a particular kind of tunnel. Bolted iron of which there are over 160 km (100 miles) in London is obviously a successful form. That it is declining now, but did not do so immediately the technology was available to replace it, lends itself to an evolutionary explanation.

Progress is a concept which is much more difficult to measure. Technical sophistication is not what is meant, nor in evolutionary terms is it necessarily an asset, for it often implies a lack of versatility and the consequent inability to withstand changed conditions. Many commentaries on progress in tunnelling use speed (rate of advance or rate of excavation of volume). This is easily measured, has often been recorded, and, of course, has increased dramatically. With this parameter however (if it is
to have any engineering meaning) it is necessary to compare like with like. This does not give us a broad view of the evolution of the whole art of tunnelling. It would be far more pertinent to be able to demonstrate the way settlement over the surface of tunnels has been decreased. A suitable non-dimensional measure would be one related to the damage caused to overlying buildings, say, the maximum slope of the subsidence trough. Unfortunately records from which this can be done are sparse.
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Terms relating to position in a ring of tunnel lining

Terms used to denote parts and dimensions of a tunnel lining
3.1 INTRODUCTION

In 1967 the Building Research Station had discussed with Stanton and Staveley Ltd, who manufacture cast iron tunnel linings, the possibility of designing new linings and testing them in full-size tunnels specially built for this purpose. Stanton and Staveley were already using spheroidal graphite cast iron* for pipes and could produce tunnel segments in the same material. London Transport was then planning the extension of the Victoria Line southward to Brixton and agreed to a short length of 3.8 m (12 ft) diameter running tunnel being used for such a trial. Because of the likely presence of buried channels in the London Clay south of the Thames, hooded hand shields would be used for the 3.8 m (12 ft) tunnels here instead of the tunnelling machines used for the rest of the line.

The traditional segments which form one sixth of the circumference of a 3.8 m (12 ft) ring of iron lining are cast in sand moulds which are prepared by hand. Smaller segments could be moulded automatically with new plant which Stanton and Staveley was soon to commission. The Station thought that for building by hand inside a tunnel shield, a ring consisting of a greater number of smaller segments would be acceptable and recommended that designs for a twelve-segment ring be prepared.

Three designs suitable for casting in spheroidal graphite iron, were submitted to London Transport who offered to allow a length of thirty rings of

*See page 141, bottom.
the most conservative design to be built into a temporary tunnel. The Building Research Station agreed to instrument one ring of the new lining and one ring of the lining which was to be used for the rest of the tunnel. The instrumentation and the site are described in Section 3.4.

3.2 OBJECTIVES

Stanton and Staveley were mainly interested in the mechanical properties of the new lining. A trial length of tunnel was expected to demonstrate that:

1. These segments would not break in handling. SG iron has a very high resistance to impact in comparison to ordinary flake graphite grey cast iron.

2. The lightness of the ring would contribute to the ease of and speed of building it. The thickness of section used (see Fig 22) could be half that of the '12 ft 7 ins iron' used on the Victoria Line because SG iron is twice as strong as ordinary cast iron.

3. The flanges would not break under the shield ram thrust, in spite of the light section employed, because SG iron is not only strong but has a high elongation.

Instrumentation of two rings could be expected to indicate that:

i. the factor of safety against failure in service was higher in the SG iron lining;

ii. a ring of twelve small segments, which could be made in automatic plant, would be as structurally sound as the traditional number of six segments per ring.
Fig. 21 The two kinds of cast-iron lining used at Brixton, the smaller segment is of spheroidal graphite cast-iron
Whereas Stanton and Staveley were mainly interested in the extent to which their rings behaved differently from the ordinary ones, the Station was also concerned with the behaviour of tunnels as whole structures acting with the ground. This led to a greater interest in the similarities between the behaviour of the two linings.

Observation of the instrumented rings would provide the Station with more data on:

1. the rate of increase in hoop thrust in a grouted tunnel;
2. the distortion of a single grouted tunnel as it aged;
3. the distortion of a tunnel by a similar tunnel driven very close to it;
4. the behaviour of the ground ahead of a large tunnel face;
5. the mechanism by which bending moments are developed in a circular segmented tunnel ring.

3.3 LABORATORY EXAMINATION OF THE SEGMENTS

3.3.1 Description, measurement and theoretical properties

Segments of both kinds were sent to the Building Research Station for testing of their structural properties and for preparation for the field experiment. The segments were as cast, the usual bituminous coating having been left off at the Station's request to facilitate instrumentation. The two kinds of segment are shown in Fig 21. The larger segment superficially resembles the traditional segments used to line most of the Underground system. It differs from them in having a full depth of flange of only 102 mm (4 ins) compared to the usual 124 mm (4 7/8 ins) and in having a thicker skin, 25 mm (1 in.) instead of 23 mm (7/8 in.). These are retrograde changes and employ a similar amount of metal to make a section which is less stiff and less strong than the traditional one (Tables 2 and 3 refer).
Fig 22 Cross-sections of the two kinds of segment used at Brixton. Top 12'7" in ordinary cast iron. Bottom 12'8" in SG iron.
Fig. 23 Photomicrographs of the two kinds of grey cast-iron, approximately x 200 magnification.
Top: ordinary cast-iron showing the graphite as flakes. Bottom: S.G. cast-iron showing the graphite as spheroids.
Six of these segments with planed cross joint faces, make, with a 76 mm (3 ins) wide solid key segment, a ring of 3.83 m (12 ft 7 ins) internal diameter. This is more than is required to clear the structure gauge so the reason for the small flanges remains unexplained. The outside diameter of 4.04 m (13 ft 3 ins) was chosen to match the outside diameter of the expanded concrete lining which was to be used with the same shield in good ground. The material was ordinary grey cast iron of grade 12 of BS 1452.

The experimental rings were of the same outside diameter but of 3.86 m (12 ft 8 ins) internal diameter and were 0.61 m (24 ins) long instead of 0.51 m (20 ins). They consisted of twelve segments, with machined cross joint faces, and a solid key segment. The thickness of both the flanges and the skin was approximately half that of the other segments. The cross sections are compared in Fig 22. The circumferential bolt holes were at roughly 0.34 m (13½ ins) centres in both rings but on pitch circle diameters differing by 25 mm (1 inch). The material, spheroidal graphite iron, differs from ordinary grey iron in that the graphite occurs as spherules instead of flakes (Fig 23) and consequently has increased strength, elongation and stiffness.* The degree to which a batch of iron has these properties can be controlled at the foundry. The material of these segments was roughly twice as strong and twice as stiff as the flake graphite iron. The properties are compared in Table 1.

*The term spheroidal graphite iron is usually abbreviated to SG iron. This iron is also known as nodular graphite iron. The British Standard (BS 2789) uses both terms. The American tend to use 'ductile iron' and this has virtually been adopted as a trade mark by Stanton and Stavely in this country. The more appropriate word 'malleable' has, of course, another meaning in the metallurgy of cast iron.
**Table 1**

Structural properties of the two types of iron

<table>
<thead>
<tr>
<th>Type of grey iron</th>
<th>flake graphite</th>
<th>spheroidal graphite</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specification</td>
<td>Grade 12 to BS 1452:1956</td>
<td>SIG 32/7 to BS 2789:1961</td>
</tr>
<tr>
<td>Tensile strength $\text{MN/m}^2$</td>
<td>$&gt; 185$</td>
<td>about 540</td>
</tr>
<tr>
<td>Tensile strength tons/in$^2$</td>
<td>$&gt; 12$</td>
<td>about 35</td>
</tr>
<tr>
<td>Young's modulus $\text{GPa}$</td>
<td>80-95</td>
<td>about 170</td>
</tr>
<tr>
<td>Young's modulus $10^6\text{lb/in}^2$</td>
<td>12-14</td>
<td>about 24</td>
</tr>
<tr>
<td>Charpy Impact mN</td>
<td>1.4</td>
<td>19</td>
</tr>
<tr>
<td>Charpy Impact ft lb</td>
<td>1</td>
<td>14</td>
</tr>
<tr>
<td>Elongation (per cent)</td>
<td>$\frac{1}{3}$</td>
<td>8-15</td>
</tr>
</tbody>
</table>

3.3.2 Strength of the segments

A few segments of both kinds were tested in the laboratory. They were subjected to bending in the plane at right angles to the tunnel axis (Fig 24). Loads, deflection and local strains were measured. The points on the cross section where strains were measured are shown in Fig 25.

Some of the strain measurements were made with a 'Demec' gauge (Morice and Base, 1953), others were made with vibrating wire strain gauges actually to be used in the tunnel and described later under field instrumentation. On the centre line of the segment the gauge was a two wire gauge (Thomas, 1966) used for measuring bending with only one pair of points of attachment. From these tests values of flexural rigidity and moment of resistance at failure were obtained.
Fig. 24  Bending test on S.G. iron tunnel segment
The moment of resistance of an ordinary cast iron segment is of practical interest long before it is actually built into a tunnel, because failure can occur simply by knocking a segment over onto its back. A measure of the capability of a segment to withstand such accelerations is obtained if the moment of resistance is divided by the moment in the segment at rest due merely to its own weight as it lies on its back only supported at its centre. This moment is \( \frac{m g l}{8} \) where \( m \) is the mass of the segment, \( g \) is the acceleration due to gravity and \( l \) is the chord length of the segment.

Table 2 shows how the short, light and strong segments of SG iron are superior to those of ordinary iron. Of the last, the segment used on the Victoria Line is inferior to the traditional one.

Moments of resistance were calculated from the dimensions taken from the manufacturer's drawings and from the specified minimum strength. Measured moments of resistance at failure differed from this because the dimensions of the segments were not to the drawings. The ordinary iron segments were too light and the SG iron segments of too thick a section, particularly in the skin. The prototype SG iron segments were hand moulded.

The variation between calculated and measured values of the stiffness, the modulus of flexural rigidity \( E I \), were much greater. One of the SG iron segments had a second moment of area, \( I \), which was twice that calculated from the specified dimensions and given in Table 3.

Also, of course, the stress strain curve of cast iron, particularly the ordinary flake graphite form, is far from straight and if the specimen is reloaded subsequent curves are of different shapes. Hence any value taken for Young's modulus, \( E \), must be regarded with caution.
### TABLE 2

<table>
<thead>
<tr>
<th>Size of lining</th>
<th>Type of iron</th>
<th>Moment of Resistance calculated</th>
<th>Moment of Resistance measured</th>
<th>Weight</th>
<th>Chord length</th>
<th>Self-weight moment</th>
<th>Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>12 ft 6 in</td>
<td>ordinary</td>
<td>M = 30.4 kN m</td>
<td>M₀ = -</td>
<td>2.75</td>
<td>2.01</td>
<td>0.69</td>
<td>44</td>
</tr>
<tr>
<td>12 ft 7 in</td>
<td>ordinary</td>
<td>25.3 kN m</td>
<td>16.5 kN m</td>
<td>2.67</td>
<td>2.03</td>
<td>0.68</td>
<td>37</td>
</tr>
<tr>
<td>12 ft 8 in</td>
<td>SG</td>
<td>32.4 kN m</td>
<td>38.0 kN m</td>
<td>0.85</td>
<td>1.04</td>
<td>0.11</td>
<td>290</td>
</tr>
<tr>
<td>12 ft 6 in</td>
<td>ordinary</td>
<td>tonf in</td>
<td>tonf in</td>
<td>620</td>
<td>79</td>
<td>2.74</td>
<td>44</td>
</tr>
<tr>
<td>12 ft 7 in</td>
<td>ordinary</td>
<td>100 tonf in</td>
<td>65 tonf in</td>
<td>600</td>
<td>80</td>
<td>2.68</td>
<td>37</td>
</tr>
<tr>
<td>12 ft 8 in</td>
<td>SG</td>
<td>128 tonf in</td>
<td>150 tonf in</td>
<td>192</td>
<td>41</td>
<td>0.44</td>
<td>290</td>
</tr>
</tbody>
</table>

### TABLE 3 - Section properties derived from manufacturers drawings

<table>
<thead>
<tr>
<th>Size of lining</th>
<th>iron</th>
<th>Used</th>
<th>Area of section A</th>
<th>Second moment of Area I</th>
<th>Distance of neutral axis from extrados</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>mm²</td>
<td>mm⁴ x 10⁶</td>
<td>mm</td>
</tr>
<tr>
<td>12 ft 6 in</td>
<td>ordinary</td>
<td>traditionally</td>
<td>17400</td>
<td>20.6</td>
<td>32</td>
</tr>
<tr>
<td>12 ft 7 in</td>
<td>ordinary</td>
<td>Victoria Line</td>
<td>17700</td>
<td>11.1</td>
<td>25</td>
</tr>
<tr>
<td>12 ft 8 in</td>
<td>SG</td>
<td>experimentally</td>
<td>10100</td>
<td>4.25</td>
<td>17</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>in²</td>
<td>in⁴</td>
<td>in</td>
</tr>
<tr>
<td>12 ft 6 in</td>
<td>ordinary</td>
<td>traditionally</td>
<td>27.0</td>
<td>49.4</td>
<td>1.28</td>
</tr>
<tr>
<td>12 ft 7 in</td>
<td>ordinary</td>
<td>Victoria Line</td>
<td>27.5</td>
<td>26.6</td>
<td>1.00</td>
</tr>
<tr>
<td>12 ft 8 in</td>
<td>SG</td>
<td>experimentally</td>
<td>15.7</td>
<td>10.7</td>
<td>0.66</td>
</tr>
</tbody>
</table>
3.3.3 Structural behaviour of the segments in flexure

To ascertain whether the laws of simple bending were applicable to a tunnel segment, a value of $E$, determined from tests on small samples, was multiplied by the calculated value of $I$ using the actual dimensions of a segment. This theoretical $EI$ was then compared with the stiffness measured in a bending test.

Results from the pairs of strain gauges could be regarded as gradients of strain $\frac{e_1 - e_2}{d}$ where $e_1$ and $e_2$ are two strain changes and $d$ is the distance between the strain gauges in the plane of bending. The results would be expressed as microstrain per inch of depth of the section. Equally well the expression $\frac{e_1 - e_2}{d}$ which has the dimensions (length)$^{-1}$ could be regarded as a measurement of curvature.

For a beam subjected to simple bending the theoretical relation is

$$M = EI \times \frac{1}{R}$$

where $M$ = applied bending moment

$1/R$ = curvature

$E$ = Young's modulus

$I$ = second moment of area.

However bending tests on the segments gave values for the moduli of flexural rigidity which were low compared to the calculated ones. This discrepancy was more marked in the case of the short, wide, light-sectioned segments in S3 iron. Also the modulus of flexural rigidity became even lower as the applied bending moment was increased.
This suggested that a tunnel segment did not bend in the way a simple beam does. Experimental investigation showed that neither did it behave as does a straight piece of material of U section and with open ends.

When a tunnel segment, curved and with flanges across its ends, was loaded by forces tending to straighten it (positive bending), it developed the expected tensile strains in the tips of its circumferential flanges with compression of the roots of these flanges. Instead of shortening with the flange-root, the relatively flexible skin of the segment bowed out, particularly near the circumferential centre line. No compressive stress occurred here, hence the skin was not contributing to the stiffness of the segment as a whole.

When a segment was bent in the opposite sense the central part of the skin straightened under negligible tension as the cross flanges tilted apart.

3.3.4 Flexure of SG iron segments with back pressurised

In order to investigate the effect of ground pressure on the bending characteristics of the SG iron segments, a uniform pressure was applied to the back of one by means of a flat rubber bag which was pressurised with water (see Fig 26). The test, up to 413 kil/m² (60 lb/in²) which is equivalent to the overburden pressure, showed that the flanges of the segment bent negatively (increase of curvature) with increasing pressure. 6.89 kil/m² (1 lb/in²) produced a curvature corresponding to -0.02 kilm (-0.8 ton in.) bending moment (see Fig 27).

The mechanism of the bending test was now operating in reverse - cause and effect being transposed. The uniform pressure moved the centre part of
the thin skin radially inwards inducing a circumferential compression in it. The skin was framed by the relatively rigid flange root which was thrown into tension. The tip of the circumferential flange which was on the other side of the neutral axis of the segment section went into compression, ie the flange was bent negatively. The extra strain gauges on the flanges also suggested that the part of the skin near the flange root acted with the flange. The width of this part may be determined by equating the moment calculated from the observed curvature of the flange and the stiffness of the increased section with the moment given by the skin stress acting over the remaining area at the appropriate distance from the neutral axis of the flange.

Thus \( \frac{1}{R} x E I = E e x \text{ active area of skin x h} \)

where \( \frac{1}{R} \) = curvature indicated by flange gauges
\( I \) = second moment of area of flange and nearby part of skin
\( h \) = distance of neutral axis of this flange section from middle of skin
\( e \) = average strain in skin.

I and h for the flange section and the active area of the skin may be expressed in terms of the single unknown - the width of the skin which acts with the flange. This has been found to be 90 mm (3½ ins) and the centroid of the increased flange section is located 21.6 mm (0.85 inches) from the middle of the skin.

The bending of the circumferential flanges allows the points of attachment of the skin to the cross flanges to move apart, whilst the skin itself becomes slightly shorter due to the compressive force in it. Hence the whole
Fig. 26 Rubber bag strapped to a segment so that a uniform pressure could be applied to its back.
Figure 27 Effect of uniform pressure on back of skin at SG iron segment
of the centre part of the skin must move inward by an amount which can be calculated from the changes in curvature of the flanges and the distance \( h = 21.6 \text{ mm (0.85 inches)} \). A value of 2.0 mm (0.078 inches) is obtained for the values measured at 413 kN/m² (60 lb/in²) whereas the deflection measured by a dial gauge was 2.4 mm (0.094 inches). Although the twin wire gauge confirmed that little or no bending occurred at the centre, bending near the cross flanges could account for this discrepancy.

The strain gauges on both sides of the flanges also showed that in addition to the circumferential bending discussed above there was also bending in the direction at right angles, such that the centres of the flanges moved slightly away from each other.

These experiments showed that a curvature may be observed in the flange of a segment, not because local forces are changing the shape of the tunnel, but merely because a hydrostatically distributed external pressure is altered. Also not only was the modulus of flexural rigidity always less than the theoretical \( E I \), but varied inversely with the applied bending moment.

Hence it was better not to attempt to interpret a value of curvature strain in the flange as indicating that the segment as a whole was resisting a particular bending moment. Also the flanges are the most vulnerable part of a tunnel segment so the strains here are our main concern.

3.3.5 Cross joints

Two SG iron segments, bolted together at their longitudinal or cross joints, were loaded so that the behaviour of the joint in bending (but with no hoop thrust) could be investigated. A similar experiment was made with two half-
Fig. 28  Bending test on the cross joint between two half segments of 12ft. 7 in. lining
Figure 29  Stiffness of cross joints in bending, with no hoop load.

- 3 bolts in plain holes
- 2 bolts " c's'nk "
- 3 bolts " "

Ordinary iron segments

SG iron segments
segments of the ordinary flake graphite iron (see Fig 28).

Results are shown in the graph of Fig 29 in which angular movement between the segments is plotted against applied bending moment. The single steep line is for the ordinary iron segments, the group of three lines is for the SG iron segments with variations in the way the joint was made. At large deflections (1-2 degree of arc) the joint in the ordinary iron segments is seen to be about twice as stiff as that in the ordinary iron segments.

Before discussing possible application of this result it is necessary to recall the construction of the joints and clarify the geometry of their deformation. Cross sections of two joints are shown in Fig 29. The machined mating faces of the longitudinal flanges are 20 mm less wide than the whole flange because the inside edge is rebated to form a caulking groove in the completed joint. Each cross joint is fastened by a row of three bolts half-way up the inside depth of the flange.

If a pair of segments is bent in a manner tending to straighten them, hinging occurs at the joint, not because the bolts stretch but because the flange distorts. The corner where the tip of the longitudinal flange joins that of the circumferential flange rotates about the line where the joint cuts the extrados. In doing so it becomes separated from the corresponding corner on the adjacent segment. If the two segments are part of a ring subjected to hoop thrust, this thrust is carried by the line contact at the extrados. If bending is in the opposite sense, the corners come together and the thrust is taken through them. The actual points of contact are at the bottom of the caulking groove.
The deformation of the cross flanges of a pair of SG iron segments undergoing positive bending is shown in Fig 30. These flanges are 20 per cent longer and half as thick as those of the lining of ordinary flake graphite iron. Allowing for the different Young's moduli of the two materials it is calculated that the relevant stiffness of the flanges of the SG lining is 20-25 per cent of that of the ordinary iron lining. The experiment confirmed this large ratio for small angular deflections of the two kinds of joint.

Figure 30 shows the joint flexed to an angular deflection far larger than that which would be encountered in a tunnel, in order to illustrate the mechanism of the joint. It also shows that the centre bolt is unlikely to be loaded nearly as much as the outer ones. Resistance strain gauges were put on the bolts and Fig 31 was produced from the results. It shows the increase in bolt tension during a bending test plotted against the angular deflection at the joint. At zero on the tension scale the bolts had each about 30 kN (3 tons) tension due to bolt tightening and to the moment resulting from the self weight of the segments."

The rates of increase in bolt tension with angular deflection are:

<table>
<thead>
<tr>
<th>Type of joint</th>
<th>Bolt</th>
<th>kN/degree</th>
<th>ton/degree</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ordinary segments with three bolts</td>
<td>Average of two outside</td>
<td>38</td>
<td>3.8</td>
</tr>
<tr>
<td></td>
<td>Middle</td>
<td>18</td>
<td>1.8</td>
</tr>
<tr>
<td>SG iron segments with three bolts</td>
<td>Average of two outside</td>
<td>15</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>Middle</td>
<td>4.5</td>
<td>0.45</td>
</tr>
<tr>
<td>SG iron segments no middle bolt</td>
<td>Average of two (not plotted)</td>
<td>18</td>
<td>1.8</td>
</tr>
</tbody>
</table>

"Hence 'zero' for any set of strain readings is not determined with any more accuracy than any other strain readings and there is no reason for drawing the curves through the origin."
Fig. 30 Distortion of cross flanges of S.G. iron segments after a bending test on the joint
Fig 31 Bending tests of cross joints: increase in tension in bolts plotted against angle at joint.
Thus the more flexible joint between 3i iron segments, with a 3:1 ratio between the tensions in the outside bolts and that in the middle ones, exaggerates the situation in the ordinary segments where there is still a 2:1 ratio.

When the bending/angular deflection test was repeated with the centre bolt omitted from the joint between the 3i iron segments, the results (plotted with dots in Fig 29) are very little different from those with all three bolts (triangles). At first it was thought that the flexibility of the joint would be increased by the distortion of the washers between the bolt heads and the countersink which is intended to receive grommets should the bolt holes have to be water proofed (see section of joint, Fig 29). Hence the joint was tested as supplied without grommets (dots) and again with conical bushes filling the countersink so that the joint would behave as if the segment had plain holes (as did the ordinary iron segments). However, the results (squares) showed that the washers did not distort until moments above 6 kNm (24 ton/in.) were reached. The divergence of the two curves is off the scale of Fig 29.

Most of the diametrical distortion of the rings during the field experiment can be accounted for by the curvature strains recorded. Any angular movements at the joints would have been very small. The strain energy corresponding to the small distortion of the relatively flexible cross flanges would change the load in the bolts negligibly.\(^\#\) Any mechanism producing bending moments near the joints at these small deflections would not depend on the cross joint bolts.\(^*\)

\(^\#\) For angular movement about a point on the extrados the bolts are centred 50 mm from the centre of rotation in the 3i iron lining and 63 mm in the ordinary iron lining. For movement about the bottom of the caulked groove the distance is only 18 mm in both linings. Thus although in the latter case the bonding of the flanges is more complicated, and it may be stiffer, any moment transmitted via them is still negligible.

\(^*\) For bonding of the middles of the segments, this was supported experimentally by loosening cross joint bolts in the tunnel - the strain gauges on the circumferential flanges were unaffected.
The field results described later show that a tunnel lining built in London Clay rapidly develops a hoop load. This soon rises to give a compression across the joint which is greater than could be applied by bolting. However, the compressive stress is not transmitted by the whole cross joint face because this is insufficiently stiff to do so. Only those parts of the face interposed between the circumferential parts of the two segments do this. Thus the centre part of the cross flange which is not a continuation of the segment cross section plays no part in the transmission of hoop stress. The only modification from transmission of compression in the rest of the ring is that part of the cross flange tip is interrupted by the caulk ing groove. The hoop compression in the ring and virtual continuation of the cross section of the segment across the joint enable it to transmit gradients of stress, ie bending.

Should bending cause tension at one side of the joint which exceeds the hoop compression the joint will start to open there. Nevertheless, the gradient of stress is retained in the unseparated part of the joint. At a sufficiently high angular deflection a line contact would develop at one edge and there would be no gradient of strain at the joint. However, due to the eccentricity of the line of thrust of the hoop force, there would still be a gradient of strain elsewhere in the segment. This gradient would, of course, have the same sign as the gradient associated with a wholly closed or partially open joint subjected to the same kind of bending. If the only data one has are strain gradient measurements at some distance from the joint with no knowledge of the geometry of the joint, one would know that a line contact had formed. However, this is not of

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Say, by driving another tunnel near to one that not yet developed much hoop compression, or by driving it too close.
practical importance because the fact that exactly at the joint the
gradient was zero would not lessen the values in the greater part of the
segment in which the measurement was made. However, since the line of
thrust passes through the edge of a joint that has opened there is an upper
limit for the strain gradient in its vicinity. This corresponds to the
bending moment which is the product of the hoop thrust and the distance
between the neutral surface and the load bearing edge of the joint.

3.4 UNDERGROUND WORK ON THE LININGS

Work in the temporary tunnel which was located near Brixton fell into four
parts:

1. observation of the building of the trial section of lining and
   comparison with the lining used for the rest of the tunnel;

2. measurement of the structural performance of one ring of each of
   the two kinds of lining during loading;

3. similar measurements as the linings were unloaded and dismantled;

4. examination of the unloaded lining and checking zeros of gauges.

3.4.1 Site

Brixton Station, at the southern end of the Victoria Line, consists, like
most London underground stations, of two large circular bored tunnels linked
to a central concourse by cross passages. The trains leave the two
station tunnels by two running tunnels each of 3.8 m (12 ft 7 ins) internal
diameter; these converge and enter a large tunnel having a horizontal
internal diameter of 9 m (29 ft 6 ins). In this tunnel the permanent way
includes a cross-over so that trains which have arrived via the southbound
running tunnel may return via the northbound tunnel.

The method of constructing the 9 m (29 ft 6 ins) cross-over tunnel was to
Fig 32 Plan showing locations of the instrumented rings
Diameter Measurement Points

Fig 33 Cross-section of the 29ft 6in 1D cross-over tunnel showing the positions of the three pilot tunnels and order of driving.
Fig. 34  The face of the 29ft. 6in. tunnel, showing the two
12ft. 7in. tunnels and the 7ft. pilot tunnel.
Compare with Fig. 33
Fig 35  Trial length of 12 segment S.C. iron lining and its relation to the ordinary lining. Observed rings shaded.
Fig. 36 Two kinds of lining in the tunnel. Experimental S.G. iron on left.
shield-drive the two 3.8 m (12 ft 7 ins) tunnels very near to each other and later to use them as pilot tunnels when hand building the cross-over tunnel. Figure 32 is a plan of the completed work. A third, smaller, pilot tunnel was also used in the upper part of the 9 m face. The section in Fig 33 shows the relation of these tunnels and their order of driving. Figure 34 shows the work in progress.

The line was extended to Brixton between 1968 and 1970. The running tunnels were to be lined with expanded concrete rings but at this site '12 ft 7 ins' bolted iron lining was substituted because a layer of clay stones caused the top of the tunnel excavation to overbreak. Where the two 3.8 m (12 ft 7 ins) tunnels were driven close together the iron was built to break joint, but at the request of the Building Research Station the section of 12 ft 7 ins iron containing an instrumented ring was built 'centre key' to provide a better comparison with the twelve-segment lining, see Fig 35.

Observations were to be made on a section of tunnel in blue London Clay where the depth of the tunnel axis was 22 m (72 ft) below ground level. The shear strength of the clay here was about 48 kN/m² (7000 lb/in²). The face was excavated under the protection of a hooded Greathad shield and the rings were grouted one at a time immediately after erection.

3.4.2 Instruments
Changes in diameter of a ring of each kind of lining were measured with a comparator consisting of a screw micrometer set in the end of a 3.8 m ((12 ft 7 ins) long beam. Readings could be reproduced to ± 0.05 mm (0.002 in.).
Since the objectives of Stanton and Staveley and of the Station did not wholly coincide, there had to be a compromise in the choice of the position at which local strains were measured. A position near the middle of the circumferential flanges was chosen for all segments because the maximum tensile strains were expected here. This would allow close comparison between the likelihood of failure in the two kinds of segment. However, it resulted in gauges in the two rings occupying different angular positions making comparison of the ring as a whole less certain. The inset sketches on the relevant graphs (Figs 45 and 46) show the actual angular positions chosen for particular segments.

The strain gauges were of the vibrating-wire type described in Thomas (1966) and fitted to the flanges of the segments as illustrated in Figs 38 and 25. The gauges were in pairs so that the radial gradient of strain due to bending in the plane at right angles to the tunnel axis could be observed. The long segments in ordinary iron had pairs of gauges on both circumferential flanges, the short segments in SG iron only had a pair on the trailing flanges, that is on the flange at the opposite end of the ring to the shield.

Vibrating-wire gauges are particularly well suited to long-term work in tunnels because they are stable (if kept dry) and can be read remotely. If the cable to the surface is cut any alteration in the electrical properties of the circuit due to the repair is unimportant because the information is transmitted as a frequency. In order to avoid inductive coupling and interference between one pair of gauge signal wires and another, two 50-way remotely controlled switches (uniselectors) were situated in the tunnel so that only one pair of gauge signal wires ran all the way to the surface.
Fig 38 Vibrating-wire strain gauges on the flange of an S.G. iron segment
This was contained in a screened 6-core cable, the other four conductors being used to link the home and remote gauge selecting switches. The selecting mechanisms were designed by a colleague, Mr J E Cheney.

The frequencies of vibration of the gauge wires were read by matching them to a variable oscillator of high stability (Ward and Cheney, 1960). The electrical frequencies from the gauge and the oscillator are added at right angles by applying them to the two pairs of plates of a cathode ray tube. Matching is recognised by observing that an ellipse has formed* (see also section on Errors in Section 3.4.8).

3.4.3 Some observations on building the two kinds of lining

Twenty nine rings of 3.86 m (12 ft 8 ins) internal diameter of SG iron lining were built. A ring consisted of twelve segments and a key, and because it could not be properly bolted to the last ordinary ring some difficulties were experienced in erecting the first ring. Subsequent rings were built more easily. However, because of the greater number of joints and the flexibility of these joints, the rings built to a less circular shape than the six segment rings. The shape found on dismantling is illustrated in Fig 39. This shows how the lower two thirds of the ring fits the inside of the tail plate of the shield very well. The difference in curvature between the extrados of the segments and the inside of the tail plate is small. Also the gap between them increases as the square of the distance from the point of contact; so the gap at the ends of the short segments is very small (0.006 in.). The eight lower segments therefore build as 237° 3 of a ring 4.08 m (13 ft 4½ ins) diameter. The upper third

*An oscillator-comparator unit purchased from Soil Instruments Ltd, of Colindale, London N19, was used.
Fig. 39 Twelve segment ring in tail of shield showing distortion of ring by self weight
then has to be built as an arch springing from the shoulder joint faces of the segments supported by the shield. These faces are inclined outwards at $1\frac{1}{8}$ more than their correct angle and are 38 mm ($1\frac{1}{4}$ ins) further apart than in the ideal ring. Thus the arch of top segments is effectively pin jointed, the bottom of the caulked groove acting as a knife edge. In order to cover the increased span a positive curvature of $9 \times 10^{-6}$ mm$^{-1}$ ($2.3 \times 10^{-4}$ in$^{-1}$) is induced in the top third of the ring, and there is a compression of about 15 kN (1.5 tons) due to the self weight of the segments. The top joints, in positive bending and compression, can be bolted up more stiffly than the shoulder joints so the pivots here are maintained while the ring is pushed back and bolted to a similarly distorted previously built ring. The Vee at each shoulder joint becomes filled when the ring is grouted and the thin tapered layer that forms would be quite strong. It is continuous with the main collar of grout; for it to be cut off and crushed would require a sufficiently large angle between the faces that the outer corners became opposed and sheared through it.

When the rings were dismantled these wedges of grout were found adhering to the machined surfaces of the cross flanges. They had tapers of up to about $1^\circ$.

With the ordinary iron lining of six long segments the ring can be built to a much more true circle. The pair of invert segments are laid in the invert with the centres of their backs touching the tail plate, and the invert joint tending to open about $\frac{1}{4}\circ$. The axis segments are then bolted on and tend to move the point of contact of the invert-axis pair to the knee joint, so that this assumes its correct attitude of $30^\circ$ and is fully mated. The invert joint will have opened a little more. Each axis segment is then likely to be bolted up in its correct attitude and as it is standing on end can easily be adjusted.
When the top segments are added they are built as an arch from skew-backs at the correct attitude but slightly spread. However, as the bolts in this lining are 63.5 mm (2 1/2 ins) from the back of the iron, the joints in positive bending can be made strongly. Hence, when the top segments and key are bolted up they tend to pull the axis segments, which are balanced on end, together.

Thus the slight distortion which are likely to occur are opening of the invert joint at the intrados and opening of the negatively bent shoulder joint at the extrados. Failure to build a fully butt-jointed ring results in a three pinned ring which is also a stable structure. All three joints are likely to be filled with grout because grout is often spilled in the invert, where the opening is inside.

These differences between the way the twelve and six segment rings were built was reflected in the results obtained later from the strain gauges.

None of the SG iron segments were broken although breakages of the ordinary iron were quite frequent. No special care was taken with the handling of the experimental segments. Because there were twice as many cross joints the miners felt at first that they were loosing a lot of time bolting the rings together. In fact, for an equal length of tunnel, the SG iron lining had only 11 per cent more bolts, see table below.
The lightness of the SG iron segments was appreciated by the miners. The supervisory staff regarded it as a theft risk. The casting flashes do not readily break off the SG iron castings and several miners cut their hands when handling the segments.

A true comparison of the rates of tunnelling was not really possible at the site on account of the following factors. There were irregular delays in haulage of iron and spoil because the distance between the shaft and the face had reached about 610 m (2000 ft). Differences in progress of less than one ring per shaft were not revealed because the gangs were paid for completed rings only. Each gang had the experience of only 2 or 3 shifts with SG iron, which is insufficient to develop an optimum working method. Shifts of only about 6 to 6½ hours duration were worked by the miners for both kinds of lining. Four rings of SG iron were built in every shift giving an advance of 2.4 m (8 ft), whereas the rate of progress for the ordinary iron was 5 rings per shift, i.e. an advance of 2.54 m (8 ft 4 ins). One grouting operation and one timbering of the upper half of the face were saved in each shift by using 0.61 m (2 ft) long rings instead of the normal 0.51 m (20 ins) long ring.
3.4.4 The effect of the shield thrust on the segment flanges

In addition to handling and service stresses the segments have to withstand the thrust from the shield rams when the shield is advanced. In the London Clay at Brixton two methods were used to advance the shield. When the second 3.8 m (12½ ft) pilot tunnel for the cross-over tunnel was driven the hood of the shield was embedded in the face which was then excavated under its protection. Elsewhere a hole about 1 ft less in diameter than the ring was excavated and the whole cutting edge of the shield was then shoved for the length of one ring. This pared quite a large volume of clay off the sides of the excavation without the use of manpower. The total load required to shove the shield by the former method was about 2491 kN (250 tons) and by the other about 1196 kN (120 tons). This load was not spread between all twelve rams because of the necessity to steer the shield. Although the hydraulic system was capable of providing 31,000 kN/m² (4500 lb/in²) oil pressure, corresponding to 598 kN (60 tons) thrust per ram, in practice pressure gauge readings were usually limited to 15,160 kN (2200 lb/in²).

On one occasion in the southbound drive shaft north of Ferndale Road a load cell was inserted into a specially made ram shoe to obtain a record of the thrust exerted by this one ram. At this time the shield was being steered to the left and a tendency to dive was being corrected. None of the six upper rams were used; of the lower rams the right hand ones were in use continually, including that carrying the load cell, which was centred on a point 75 mm (3 ins) below the knee joint. A load v time record for the 'shove' from ring 1637 is shown in Fig 40. Pressure gauge readings were 12,440 kN/m² (1800 lb/in²), rising steadily to 13,780 kN/m² (2000 lb/in²) and peaking at 15,200 kN/m² (2200 lb/in²) as the hood hit solid ground. These values correspond to the area of the 150 mm (6 ins) diameter rams so
Fig. 40  Thrust exerted by one ram during a 'shove'
that if the pressure gauge was correctly calibrated there must have been very little friction in the rams. The saw tooth trace in Fig 40 is due to varying load as the sledge dragged by the shield alternately stuck and slipped.

It was not opportune to measure local strains in the newly built ring in this experiment, but in a laboratory experiment a uniform load was applied along the 3 mm (1/8 in.) high 'chipping face' or 'fillet' of an SG iron segment. A maximum compressive stress was measured at the centre of the skin which was 1.7 times the average stress in the skin. The circumferential flange was not significantly affected.

However, in practice flanges are highly stressed and the flanges of the 3.83 m (12 ft 7 ins) ID ordinary iron segments sometimes broke (see Fig 41) so it is worth considering if the design assumption is valid that all the shield thrust is transmitted via the fillet to the skin.

The shields used at Brixton for the running tunnels were designed for use with a concrete lining of rectangular section 0.61 m (24 ins) long by 150 mm (6 ins) radial width. The twelve shove rams are equally spaced on a pitch circle diameter of 3.77 m (12 ft 4½ ins) and they extend in a direction parallel to the axis of the shield skin plate. Each ram is 150 mm (6 ins) diameter and the end is domed to 150 mm (6 ins) spherical radius; this is seated in a socket with a concave bottom of the same spherical radius (see Fig 43). The diametrical clearance between the 44 mm (13/4 ins) deep socket and the ram is 3 mm (1/8 in.) which limits the angular movement to 5° from the concentric position. The socket is bored in a boss welded to a stiff, ribbed spreader plate, this whole assembly, which the manufacturers of the shield call a crosshead, is retained, but not
gripped on the end of the ram by a screw passing through a large clearance hole in the crosshead to a hole tapped axially in the ram end. To the 'crosshead' is bolted the ram shoe, a block of the hardwood Bkky, 100 mm (4 ins) thick; viewed from the rear of the shield it takes the form of a segment of radius 2.03 m (6 ft 8 ins) of sagitta 200 mm (8 ins) and a chord length 1.02 m (40 3/8 ins) so that it subtends an angle of nearly 30° at the shield centre. The inside diameter of the shield tail plate is 4.1 m (13 ft 4 1/2 ins) allowing a clearance between it and the cylindrical outside surface of the shoe of 6.4 mm (1/4 in.). However, the 'crosshead' may rotate about the ram axis so the shoe may turn slightly and touch the tail plate at one end.

Figure 42 shows a ram shoe bearing on a top segment of 3.83 m (12 ft 7 ins) lining. If the flat bearing face of the ram shoe were parallel to the plane of the leading edge of the ring the area of contact would be the middle 30° of the fillet of the segment. The centroid of this area is 36 mm (1 4 ins) from the extrados of the ring. If this were concentric with the tail plate of the shield the distance between this centroid and the centre line of the ram would be 120 mm (4.7 ins). Clearance between the lining and the tail plate allows this dimension to vary by ± 19 mm (± 3/4 in.). This lack of alignment between the line of thrust of the ram and the reaction from the segment results in a couple which turns the whole ram shoe radially outward when the ram is used. The flat rear face of the shoe is 311 mm (12 1/4 ins) from the centre of the spherical seating so the shoe can only turn tan⁻¹ 6.35/311 mm (1/4 in/12 1/4 ins), about 1°, before it touches the tail plate. Thus a new ram shoe does bear on the fillet and not on the flange. However, the couple has to be resisted by a force between the curved edge of the shoe and the tail plate and a sideways component added to the ram thrust. When the shoe is moving it is rubbing
Fig. 41 Instrumented 12ft. 7in. segment and a piece of the flange of a similar segment broken by a shield ram with a worn shoe
Fig. 42 Shove ram thrusting against a top segment of 12ft. 7in. lining.
Fig. 43  Relation between worn ram shoe and segment (a) view from shield showing contact at only four points; (b) section (on enlarged scale) showing how rotation causes this and how wear allows rotation.
the tail plate with a force equal to about a third of the ram thrust.

Figure 42 shows the marks made on the tail plate by this rubbing, and also shows the grout splashed on the shield providing a source of abrasive. Hence the outside of the shoe rapidly wears from the rear forward, until the harder steel crosshead is scraping the tail, which in turn causes both these components to wear. Now the crosshead-shoe assembly can turn outwards through a larger angle, \(\tan^{-1} \frac{1}{4} \text{ in}/(12\frac{1}{4}-4)\text{ in.}, \) about 2\(^o\) (see Fig 43).

The flat rear face of the ram shoe is no longer parallel to the plane of the fillet and only the ends of the shoe touch the inside edge of the fillet. This cuts into the wood. When this cut exceeds 0.8 mm (\(\frac{1}{32}\) in.), the angle adopted by the externally worn shoe is enough to bring the inner part of the flat face into contact with the tip of the flange of the segment (see Fig 43). As rotation of the shoe about the ram can allow the ends of the shoe to move radially \(\pm 6.4 \text{ mm (}\pm \frac{1}{4}\text{ in.)}\) and the lining can move \(\pm 19 \text{ mm (}\pm \frac{3}{4}\text{ in.)}\) the ends of the shoes become very worn. Contact between the shoe and the segment is now at four points. On the ordinary segment they are at the quarter points on the edge of the flange and fillet. The line of thrust of the ram is 28 mm (\(1\frac{1}{8}\) in.) \(\pm 19 \text{ mm from the flange tip according to the position of the lining in the tail plate, i.e nearly all the ram thrust can be concentrated at two points 9.5 mm (}\frac{3}{8}\text{ in.) from the tip of the flange. This can result in the whole central half of the flange between the quarter points breaking off (see Fig 41).}

Deformation of the flanges by the shield ram forces affected the strain gauge readings in the later measurements. Plastic deformation of the ordinary iron caused changes in the strain gauge zero reading from before till after the experiment (see section 3.4.8 on Errors). The changes in
the SG iron flanges were reversible and appear as spurious indication of circumferential bending.

The circumferential flanges at the trailing edge of the newly built ring, away from the shield rams, are also affected because of the wood packing. The design assumes that 6.4 mm (1/4 inch) thick soft wood packing will not bend the flanges and that the thrust will be transmitted through the fillets to the skin. In practice thick or even double packing is often used on the outside of a curve in the tunnel so that all the thrust is borne by the flanges. The condition is less severe than at the leading edge because the load is distributed more, both circumferentially and radially.

Hence the fillets are of little value in avoiding bending the trailing flanges. If the circumferential joints were all single packed they would ensure that the rings were built parallel to one another; but since a typical value of creep for a straight, shield driven, cast iron running tunnel is 3 mm (1/8 inch) per ring it is evident that corrections are required anyway and thick packing often separates the segments in the present design. If the tunnel has to be caulked the fillet is useful as a bottom stop to pack the caulking against, but, because the cross joint caulking groove is at the flange tip, requires the complication of a 'block joint' at the corner.

The joints in the SG iron ring of twelve segments occupied the same angular positions as the shove rams, so that the bearing points were in the centres of the leading circumferential flanges. To allow full use of the ram stroke with the 0.61 m (2 ft) long rings much thinner ram shoes were used. These had only a very narrow cylindrical surface to bear on the inside of
the shield tail plate, so even when they were new they could tilt as the thick ones could when they became worn. The parts of the shoe that were thrown against the tip of the segment flange were the parts near its ends where there were bolts for attaching the shoe to the 'crosshead'. The normal shoe has holes which are counter sunk to receive the bolt heads, but the thin shoes had flat domed bolt heads on their surfaces. The heads of the bolts left clear indentations near the tips of the SG iron flange. There was no sign of cracking of any of the flanges despite this evidence of severe loading at their tips.

3.4.5 Measurements of the instrumented rings in the tunnel – First Part
This section records and interprets the measurements made in the tunnel from the time when the instrumented rings were built until after the second 3.8 m (12½ ft) diameter tunnel had been driven past them. The measurements are tunnel diameter changes and readings from strain gauges; the latter are interpreted firstly as bending (curvature) strains and then as hoop strains.

3.4.5.1 Diameter changes
Diametrical deflections of the rings were measured horizontally, vertically and towards the end of the experiment, obliquely. These are plotted against time (Fig 44) and are abstracted in Table 5. The points between which the measurements were made are shown in Fig 33.
Fig. 44  Changes of Diameter of the two rings, plotted against time.
TABLE 5 - Diameter changes in inches

<table>
<thead>
<tr>
<th></th>
<th>Inches x 10^-3</th>
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<tbody>
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<td>Horizontal</td>
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<tr>
<td><strong>Six segment ring of ordinary cast iron:</strong></td>
<td></td>
</tr>
<tr>
<td>Grouting</td>
<td>20-25 June 1968</td>
</tr>
<tr>
<td>Initial squatting</td>
<td>25 June - 29 July '68</td>
</tr>
<tr>
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<td>5-29 July 1968</td>
</tr>
<tr>
<td>Other 12 ft tunnel passing</td>
<td>29 July - 6 Aug '68</td>
</tr>
<tr>
<td>Squatting of twin tunnels</td>
<td>6 Aug '68 - 20 Jan 1969</td>
</tr>
<tr>
<td>Maximum change</td>
<td>25 June '68 - 20 Jan '69</td>
</tr>
<tr>
<td>7 ft pilot tunnel passing</td>
<td>11-17 Feb 1969</td>
</tr>
<tr>
<td><strong>Twelve segment ring of steel cast iron:</strong></td>
<td></td>
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<td>Grouting</td>
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</tr>
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<td>Initial squatting</td>
<td>25 June - 29 July '68</td>
</tr>
<tr>
<td></td>
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<td>30 July - 7 Aug '68</td>
</tr>
<tr>
<td>Squatting of twin tunnels</td>
<td>7 Aug '68 - 11 Feb 1969</td>
</tr>
<tr>
<td>Maximum change</td>
<td>25 June '68 - 17 Feb '69</td>
</tr>
<tr>
<td>7 ft pilot tunnel passing</td>
<td>21-28 Feb 1969</td>
</tr>
</tbody>
</table>
TABLE 5 - Diameter changes in mm

<table>
<thead>
<tr>
<th>Six segment ring of ordinary cast iron:</th>
<th>Millimetres</th>
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</thead>
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<tr>
<td>Grouting</td>
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<td>20-25 June 1968</td>
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<td></td>
<td>25 June - 29 July '68</td>
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<td></td>
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<td>Other 12 ft tunnel passing</td>
<td>29 July - 6 Aug '68</td>
</tr>
<tr>
<td>Squatting of twin tunnels</td>
<td>6 Aug '68 - 20 Jan 1969</td>
</tr>
<tr>
<td>Maximum change</td>
<td>25 June '68 - 20 Jan '69</td>
</tr>
<tr>
<td>7 ft pilot tunnel passing</td>
<td>11-17 Feb 1969</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Twelve segment ring of S3 cast iron:</th>
<th>Millimetres</th>
</tr>
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<tbody>
<tr>
<td>Grouting</td>
<td></td>
</tr>
<tr>
<td>Initial squatting</td>
<td>21-25 June 1968</td>
</tr>
<tr>
<td></td>
<td>25 June - 29 July '68</td>
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<td>Other 12 ft tunnel passing</td>
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<tr>
<td>Squatting of twin tunnels</td>
<td>7 Aug '68 - 11 Feb 1969</td>
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<tr>
<td>Maximum change</td>
<td>25 June '68 - 17 Feb '69</td>
</tr>
<tr>
<td>7 ft pilot tunnel passing</td>
<td>21-28 Feb 1969</td>
</tr>
</tbody>
</table>
(a) Grouting

The horizontal diameters of both the six-segment ring of ordinary grey iron and the twelve segment ring of SG iron were both measured immediately after they were built and while they were still in the tail of the shield. The next measurements which were taken several days later showed the horizontal diameter of both rings to have decreased. This deflection is of the opposite sense to the usually observed distortion of tunnel linings in clay, an increase in horizontal diameter and decrease in vertical diameter commonly known as 'squatting'. The probable mechanisms of this initial contrary movement are discussed in section 3.4.7, page 192.

The reason for the delay in starting to measure horizontal diameters is that machinery towed on a 'sledge' behind the shield was in the way. The sledge itself prevented vertical diameters being measured for about two weeks.

Until the grout had hardened deformation would depend on the small stiffness given to the ring by the cross joint bolts and the extent to which support was given by earlier built rings around which the grout had already gained strength. As these are unknown variables dependent upon the tightness of the bolts and the friction in the circumferential joints, it is impossible to draw any conclusions from the magnitude of these results. It is, however, interesting to note that in the case of the ordinary iron, where the change in diameter was measured on both the leading and trailing edges of the ring, that the unsupported leading edge showed a greater change than did the trailing edge.

(b) Initial squatting

During the five weeks between grouting and the passing of the second 3.8 m
(12\(\frac{1}{2}\) ft) ID tunnel, the horizontal diameter increased and the vertical diameter decreased for each of the instrumented rings. The rate of change of diameter was greatest at the beginning of the period and became gradually less. The total changes in diameter during this period were about the same for each of the rings. The results show that the vertical diameter decreased more than the horizontal diameter increased in the case of the ordinary iron ring. However, the SG iron ring of twelve segments, with joints at axis level as well as at the crown and invert, deflected equally on the vertical and horizontal diameters.

If hinges form at the cross joints the four-fold symmetry of the ring of twelve segments would allow equal but opposite deflections along the horizontal and vertical diameters. The ring of six segments with no joint at axis level, but capable of hinging both at the points of maximum bending moment on the vertical centre line and at points between there and the likely point of inflection, must deflect more on the vertical diameter. If the segments were infinitely stiff and the linkage was distorted maintaining symmetry about the vertical and horizontal axes, the ratio of vertical to horizontal deflection would be \(\tan \frac{\pi}{6}\) or \(\sqrt{3}\).

(c) Other 3.8 m (12\(\frac{1}{2}\) ft) tunnel passing

It can be seen from Fig 44 that very large changes in horizontal and vertical diameters occurred at the end of July, when the second 3.8 m (12\(\frac{1}{2}\) ft) tunnel was driven past the instrumented rings.

The horizontal diameters increased very rapidly during the period from when the face of the passing tunnel was a tunnel diameter before being opposite each instrumented ring to the time when the tail of the shield was a tunnel.
diameter ahead of each ring. After the tail of the shield was about a diameter ahead of each instrumented ring, the horizontal diameter still increased, but at a reducing rate, for about a week. In contrast to the case of simple squatting the two diametrical changes differ considerably in this case, the horizontal change being large because support is lost from the right hand side of the tunnel locally at axis level. For the more flexible twelve segment ring the horizontal diameter change was 53 per cent greater than the vertical diameter change, but in the case of the six segment ring the horizontal change was only 28 per cent greater than the vertical change.

Comparing one ring with the other, we see that the deformations are noticeably different but were not large enough to give cause for alarm over the structural stability of either ring. The change in horizontal diameter for the twelve segment ring was 25 per cent greater than that of the six segment ring. The change in the vertical diameter for the twelve segment ring was only 16 per cent greater than that of the six segments.

(d) Squatting of the twin tunnels
About a week after the 3.8 m (12½ ft) drive had passed each instrumented ring, five months followed during which comparatively little deformation occurred. Each ring was still squatting, but with the vertical diameter decreasing a lot more than the horizontal diameter was increasing. This suggests a rounding out of an egg-shaped section acquired when the second drive was made, as a more equal pressure distribution round the tunnel was re-established. Deformation of both rings had practically ceased by the end of the period.
The diameter changes of a third ring were also measured. This was the centre ring of a row of five SG iron rings which had had the stiffness of every segment reduced (Fig 35). Each circumferential flange had a cut between the centre bolt hole and the tip of the flange. Thus, over the inch or so length of the elongated bolt hole the flexural rigidity was reduced to about one tenth of that of the full section. The diametrical distortions of this ring were hardly different from those of the other twelve segment ring. Measurements across the cut with a Demec gauge did not show any particular trend. This demonstrates, qualitatively at least, that the SG iron segment had unnecessarily high stiffness and strength.

3.4.5.2 Local bending strains

From the difference in readings of pairs of gauges the radial strain gradient were calculated. These are expressed as curvature, and for the segments near the other tunnels (see Fig 33) have been plotted against time in Figs 45 and 46. A tendency for the segment to straighten is plotted as positive curvature.

The two types of segment have approximately the same modulus of rigidity

$$E I = 8.4 \times 10^5 \text{Nm}^2 (1.3 \times 10^5 \text{tons in}^2) \pm 30\text{ per cent.}$$

The variation occurs from segment to segment because of dimensional differences and, particularly with an SG iron segment, there is variation with time due to changes in shape of its section with earth pressure and bending. Bearing these limitations in mind, a rough idea of the bending moment at any time can be obtained by multiplying the curvature by this value of $E I$.

Curvature changes of up to $10 \times 10^{-6} \text{mm}^{-1} (2.54 \times 10^{-4} \text{in}^{-1})$ were measured. Values of this order could be obtained in the laboratory when moments of
about 8 kNm (32 ton in.) were applied to an unrestrained segment. In the SG iron segments only, the same curvature could be induced by a uniform pressure of 230 kN/m² (2.25 tons/ft²) acting on the skin.

By multiplying the curvature by the product of the Young’s modulus and the distance between the centroid and the tip of the flange an approximate value of the maximum stress due to bending may be obtained. The factor for the ordinary grey iron where E is 85 to 100 MN/m² (5.5 to 6.5 tons/in²), and the stress-strain curve is far from linear, is about $6.9 \times 10^9$ N/m $\square$ $1.75 \times 10^6$ tons/in.) $\pm$ 20 per cent, and for the SG iron segments which have a more constant E, but in which the position of the neutral axis changes with curvature, is about $12.6 \times 10^9$ N/m ($3.2 \times 10^6$ tons/in) $\pm$ 20 per cent. The tension due to bending in the ordinary iron is largest on 6 August in the top segment, approximately 42 MN/m² (2.75 tons/in²).

Superimposed on this stress there is a compressive hoop stress, see below, which reduces the tension to 30.8 MN/m² (2 tons/in²).

Changes in curvature which correspond to particular periods in the life of the tunnel (eg the change between the state before the second tunnel was driven and the state after it had passed) have been plotted on polar diagrams. For each ring eight polar diagrams have been plotted and labelled with letters a to h. This section (3.4.5) is only concerned with diagrams a, b, c and d (see Fig 47).

Table 6 lists the cause of each change against the letter of the diagram and also gives the dates of the two measurements which were subtracted to give the change. The dates are indicated by darts on the curvature-time graphs for each ring (Figs 46 and 47).
TABLE 6

Changes of curvature plotted on polar diagrams comprising Fig 47:

Periods of time during which the changes occurred in each ring.

<table>
<thead>
<tr>
<th>Diagram</th>
<th>Cause of change</th>
<th>Ordinary ring of 6 segments</th>
<th>12 segment ring SO iron</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>Erection and grouting</td>
<td>20 - 21 June 1968</td>
<td>21 - 23 June 1968</td>
</tr>
<tr>
<td>b</td>
<td>Initial squatting</td>
<td>21 June 1968 to 29 July 1968</td>
<td>23 June 1968 to 30 July 1968</td>
</tr>
<tr>
<td>c</td>
<td>Second 3.8 m tunnel passing</td>
<td>29 July 1968 to 6 Aug 1968</td>
<td>30 July 1968 to 7 Aug 1968</td>
</tr>
<tr>
<td>e</td>
<td>Approach of 9 m tunnel</td>
<td>20 Jan 1969 to 18 Feb 1969</td>
<td>20 Jan 1969 to 1 Mar 1969</td>
</tr>
<tr>
<td>f</td>
<td>2.2 m pilot tunnel passing</td>
<td>11 - 17 Feb 1969</td>
<td>21-28 Feb 1969</td>
</tr>
<tr>
<td>g</td>
<td>9 m face near</td>
<td>18 Feb 1969 to 3 Mar 1969</td>
<td>1 - 13 Mar 1969</td>
</tr>
<tr>
<td>h</td>
<td>Dismantling</td>
<td>3 March 1969</td>
<td>13-14 March 1969</td>
</tr>
</tbody>
</table>

On the polar diagrams curvature corresponding to a tendency for the segment to straighten (positive curvature) is plotted towards the centre. The angular positions of the points correspond to the positions of the pairs of strain gauges round each ring. The small plans in the figure show that state of the tunnel work at the end of one period and the beginning of the next (ie on the dates listed in Table 6).

The points in the polar diagrams have been joined with a smooth dashed line in order to educe the bending pattern. Once the hoop stress has built up,
Fig. 45 (i) Bending (expressed as curvature) in the ordinary segments, plotted against time. The darts correspond to the dates in Table 6.
Fig. 45 (ii)  
Bending (expressed as curvature) in the ordinary segments, plotted against time. The darts correspond to the dates in Table 6.
Bending (expressed as curvature) in the experimental S.G. iron segments, plotted against time.

The dates correspond to the dates in Table 6.

Fig. 46 (b)
Fig. 46 (ii)  Bending (expressed as curvature) in the experimental S.G. iron segments, plotted against time. The darts correspond to the dates in Table 6.
Fig. 46 (iii) Bending (expressed as curvature) in the experimental S.G. iron segments, plotted against time. The darts correspond to the dates in Table 6.
Fig. 46 (iv)  Bending (expressed as curvature) in the experimental S.G. iron segments, plotted against time. The darts correspond to the dates in Table 6.
Fig. 47  Bending distribution around the rings. Positive changes of curvature (tend the segment) are plotted radially inwards. The letters refer to the periods changes occurred: these are listed in Table 6. The plans indicate the state the dates given in Table 6 and shown on the curvature v time graphs.
radial gradients of strain can occur anywhere in the segment even near the cross joints. Hence the dashed line gives a fair approximation to the distribution of curvature change around the ring, except actually at the joints. The curves make it possible to compare patterns from the two types of ring although the angular positions of the gauges were different.

(a) Erection and grouting
During and soon after the erection of the instrumented rings large changes were recorded from the gauges on nearly all of the segments. Each ring was built by laying the invert segments in the lower part of the shield tail and then building the upper part of the ring on them as an arch, and then bolting the whole ring to the previous ring. Thus the strains in the erection stage depend very largely on the attitude of the shield at the time and the manner in which the shield rams were used. After a ring has been built the shield is advanced and steered by an uneven distribution of loads from the shield rams. As we have seen these loads may be high enough to induce plastic yield or even failure of the leading flanges. The bolting stresses in the trailing flange are released and perhaps reversed as the wood packing is compressed. Only after the ring has been grouted into the ground and a second above has been completed on the next ring does it behave in a consistent fashion. Therefore, the change from laboratory zero until after a few rings had been built has been considered together (period a) and next period (b) started from the latter time.

*No line has been drawn in diagram 'a' because they do not result from true circumferential bending, see (a) above and page 175. In the diagrams for the ordinary iron ring, which has gauges on both circumferential flanges, an average value of curvature has been plotted, except were the values differed markedly.*
In the ordinary iron ring only the pair of gauges on the trailing or rearmost circumferential flange of the right hand axis level segment showed negative bending (point marked 'T' on the polar diagram 'a' of Fig 47). All other pairs showed positive bending and in every case this was somewhat greater for the leading flange than the trailing flange. In the ordinary segments there was a shield ram shoe pressing on the centre quarter of each leading flange. Thus it is fairly certain that the strain gauge changes here were not caused by external forces bending the segments circumferentially, but were due to bending the flanges about their roots. The tip of the flange is pushed towards the other circumferential flange at its middle but not at its ends which are supported by the cross flanges. The vibrating-wire strain gauge on the inside surface of the flange near the tip thus records a tension whereas the strain gauge near the root is little affected. Therefore no line has been drawn through the points in polar diagram 'a' of Fig 47.

In the 5G iron ring there were no gauges on the leading edges of the segments, but double packing was used which could cause apparent positive bending of the instrumented trailing edges, this occurred in nine of the twelve segments.

(b) Initial squatting

For the next five weeks each ring was "squatting" as is usual with single, deep tunnels in London Clay. However, diagrams 'b' of Fig 47 show that in neither ring was the pattern of bending in the segments bilaterally symmetrical. However, if a smooth line is drawn through the points in

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The gauge cable to the right hand segment was cut at this time and was inaccessible so this point is omitted.
the left hand diagram it takes the form of an ellipse, the minor axis of which is tilted 15° to the right of vertical. Its axes intersect at a point equivalent to 38 mm (1.5 ins) in full-scale above the centre of the tunnel. If an attempt is made to construct a similar tilted ellipse through the points in the right hand diagram the resulting figure appears to be distorted in a way that at first sight suggests that the six-segment rings (1\(^{1}/3\) diameter away) are influencing the twelve-segment rings. However, the pattern is also consistent with the disposition of those cross joints which, due to the method of erection of the ring, act as hinges during the very early life of the tunnel, see section 3.4.3, page 160 and Fig 39. During this period the curvatures developed in the short 3G iron segments were larger than those in the long ordinary iron ones. The segment above left hand axis level developed a curvature of \(1.5 \times 10^{-6} \text{ mm}^{-1}\) \((0.36 \times 10^{-6} \text{ in}^{-1})\); half of this change occurred between 17 and 19 July. Interpretation of the strain gauge readings as hoop load (see later) shows that, during the five weeks period represented by diagrams 'b', the soil pressure had increased by 160.95 kN/m\(^2\) (1.5 tons/ft\(^2\)). This would itself have produced additional negative bending in the 3G iron segments (as described in section 3.3.4). Because the edges of the segments are in contact with other segments, this kind of change could occur suddenly – in the manner of the 'oil-can effect'.

Although the bedding of the London Clay was quite horizontal it is possible that a stress field existed in the clay before which intersected the tunnel to give a distribution of pressures on the ring compatible with the bending pattern shown in the left hand diagram 'b' of Fig 47.

From Figs 45 and 46 it can be seen that in general the increase in curvature with time over the five weeks was linear. The only segment to
change in a curve like the diameter vs time curve is the right hand axis level segment of the six-segment ring of ordinary iron (Fig 45, ii). The sudden displacement from the curve on 8 to 10 July may be due to the installation of struts in nearby rings as a precaution against iron breaking as the other tunnel was driven past.

(c) Other 3.8 m (12½ ft) tunnel passing

The very large changes to both rings at the end of July (Figs 45 and 46) were caused when the clay at the right hand side of the tunnel moved towards the excavation that was being made for the other tunnel. Most of the movement occurred over two days in each case. Strain gauge changes (to ± 1 microstrain) were first observed on the right hand axis level segment of the ordinary iron ring, 7.1 m (20 ft) from the approaching face; but horizontal diameter measurements to 0.051 mm (0.002 in.) were not recorded until the face was 4 m (13 ft) away. Radial movements in the clay were first detectable 6 m (16 ft) ahead of a similar approaching face (see Chapter 4). Thus the deformation was carried forward three rings by the friction between their circumferential joints and the wood packing compressed between them.

In both cases the rate of bending increased until the cutting edge of the shield was opposite the instrumented ring. The changes in the gauges on the right hand side of the rings, next to the other tunnel, were far greater than on the other side, reaching a maximum rate as the tail of the shield passed the instrumented ring. By the time the tail was a tunnel diameter away changes were no longer taking only hours and each ring appeared to have settled down although the six-segment ring took about six hours longer to do so than the other one. However, as Figs 45 and 46 show bending of
the same sense continued at a reducing rate for at least another five days. Diagrams 'c' of Fig 47 therefore cover the change in curvature of each segment over 8 days. The bending patterns in both rings are approximately symmetrical about a horizontal plane through the axes of the tunnels. The greatest change of curvature occurred at the point which is nearest to the other tunnel. There were two pairs of strain gauges here on the six-segment ring and both of them indicated a negative change in curvature of $8.8 \times 10^{-6}$ mm$^{-1}$ ($2.23 \times 10^{-4}$ in$^{-1}$). Subsequent change was in the same sense so that by 6 August a total change had occurred of $9.2 \times 10^{-6}$ mm$^{-1}$ ($2.40 \times 10^{-4}$ in$^{-1}$); thereafter the curvature reduced here. There was no gauge on the twelve-segment ring at this point of maximum curvature, but the gauge pairs 10° above and 20° below the axis level showed quite large negative changes of curvature: 8.1 and $6.8 \times 10^{-6}$ mm$^{-1}$ ($2.05$ and $1.73 \times 10^{-4}$ in$^{-1}$) respectively.

The bending that is shown by diagram 'c' near the right hand axis level joint does not mean that no hinge formed here. (See section 3.3.5, page 152.)

The positive changes of curvature in diagram 'c' of Fig 47 are $4.8 \times 10^{-6}$ mm$^{-1}$ ($1.2 \times 10^{-4}$ in$^{-1}$) for the top segment and $3.5 \times 10^{-6}$ mm$^{-1}$ ($0.9 \times 10^{-4}$ in$^{-1}$) for the invert segment of the right hand side of the ordinary iron ring of six segments. As in the case of the axis segment these changes are in the same sense as the initial squat and the tensile stresses at the tips of the flanges due to bending, reached on the 6 August, were maxima. However, in each case they only total about a quarter of the ultimate strength, and they are probably nullified by the locked up casting stress and the hoop stress. The values for the nearest corresponding gauge pairs in the SG iron ring are little different, $4.4$ and $3.2 \times 10^{-6}$ mm$^{-1}$ ($1.1$ and $0.8 \times 10^{-4}$ in$^{-1}$). Thus as in the case of the initial squatting the two
kinds of segment which have similar stiffnesses, but a two to one length ratio, were bent about the same amount at their centres.

The curvature measured at this time were the peak values obtained from the gauges on the ordinary segments whereas in the short SG iron segments larger values were recorded later when the 2.2 m pilot tunnel was driven (see Figs 45 and 46). A rough calculation can be made of stress at the extremity of the section. Allowing for the hoop stress, the tensile stress at the tip of the worst stressed flanges was about 13 per cent of the ultimate tensile strength in the ordinary iron, and about 16 per cent of the ultimate tensile strength in the SG iron.

The main difference between the patterns of bending in the two diagrams 'c' of Fig 47 is that the six-segment ring has been affected on its left hand side as well as on its right hand side adjacent to the other tunnel. It appears that the longer segments spread the effects round the ring more. The twelve-segment ring had developed negative curvature of $0.8 \times 10^{-6}$ in$^{-1}$ ($0.2 \times 10^{-4}$ in$^{-1}$) on its left hand side. However, the average earth pressure changed by 24 kN/m$^2$ ($3.5$ lb/in$^2$) between 29 July and 6 August. These changes accord fairly well with the relation between pressure and bending found in the laboratory and shown in Fig 27. Hence it is deduced that the bending of the segments on the left hand side of the twelve-segment ring was not caused by moments transmitted from other segments or by a change in the distribution of earth pressure on the ring, but merely by the change in value of a fairly uniform pressure.

(d) Squatting of the pair of twin tunnels

The change during the period after the 3.8 m ($12\frac{1}{2}$ ft) drive had passed the
instrumented rings and before the 2.2 m (7 ft) tunnel approached them is due not only to the squatting of the observed tunnel but also to the presence of the other one; this transmits the earth pressure on most of its circumference to the central spandril of clay between them. This in turn restores the more even distribution of pressure originally acting on the first tunnel.

One would expect this process to start immediately the grout is as strong as the clay—say after one day; but Figs 45 and 46 show that deformation of the first tunnel is not reversed until a week after the second drive has passed. A likely explanation is that the second tunnel, like the first, deformed by increasing its vertical diameter and decreasing its vertical diameter ('peaking') during the first week.

When the second tunnel does start to squat, pressure is exerted which tends to bend the segments of the right hand side of the first tunnel back to the curvature they had before the second drive. About a third of the curvature imparted to the segments when the second tunnel was driven is lost in this way. There was much less recovery in the lower quadrants of the tunnel. Perhaps this is because the invert of the tunnel usually contained water which allowed the lower part of the disturbed clay to swell readily as and after the other tunnel was passing.

3.4.5.3 Hoop stress

Hoop stress v time curves have been prepared from the strain gauge data and are shown in Fig 48. The average stress across the section has been calculated from each pair of gauges on a segment flange, and these results have been averaged to give the average hoop stress values used. The
Fig. 48 Hoop thrusts in the two rings v. time
results for individual flanges varied considerably, sometimes being com-
pressive in one flange of a segment and tensile in the other. Some points
represent the average of less than the full number of gauge pair results
due to temporary failure of some gauges. Because of this, and the fact
that the 32 iron had flange gauges on the trailing edge only, and also
because of structural instability of the 32 iron segments, the curve for
the ordinary iron ring is likely to give a truer picture.

Figure 48 shows that the curve for the two instrumented rings are similar
in shape but since the ordinary iron lining is about twice as thick as the
32 iron lining the stress is about half as much at the corresponding times.
The lengths of the rings are also different so that in order to make a
direct comparison it is necessary to express the results in pressure. This
has been done in Table 7. The values for the dismantled ring do not quite
agree with the initial zeros. This error has been considered as due to a
constant drift in each case and distributed proportionally to the time
since installation to give the 'corrected pressures' in columns IV and VII.
The percentage by which the results from 32 iron ring exceed those from
the ordinary ring has then been obtained from these figures (column VIII).
Mostly the pressures thus obtained from the two rings agree quite well;
the largest difference is ~4.5 per cent at the end of July and this is
associated with gauge failures in the invert of the ordinary iron ring.

With Table 7 and the limitations of the instrumentation in mind it is now
possible to summarise the behaviour of the two rings as shown in Fig 48.
The first parts of the curve would be smooth had not some gauges temporarily
failed; so it appears that, up to the time the second tunnel was driven,
both rings behaved normally. However, the thrust indicated for the
### TABLE 7a  Comparison of overburden pressures deduced from the strain gauge readings of the two rings

<table>
<thead>
<tr>
<th>Episodes</th>
<th>Six segments - ordinary iron</th>
<th>Twelve segments - SG iron</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(ii) Time</td>
<td>(iii) Pressure</td>
</tr>
<tr>
<td></td>
<td>Days after erection</td>
<td>kN/m²</td>
</tr>
<tr>
<td>Tunnel 4 days old</td>
<td>4</td>
<td>136.3</td>
</tr>
<tr>
<td>Before 2nd 3.8m (12ft) drive</td>
<td>38</td>
<td>168.0</td>
</tr>
<tr>
<td>Imm after 2nd drive</td>
<td>42</td>
<td>191.0</td>
</tr>
<tr>
<td>5 days after 2nd drive</td>
<td>47</td>
<td>193.0</td>
</tr>
<tr>
<td>3 weeks &quot; &quot; &quot;</td>
<td>62</td>
<td>191.0</td>
</tr>
<tr>
<td>Before approach of 2.2m (7ft) pilot</td>
<td>215</td>
<td>193.0</td>
</tr>
<tr>
<td>Next max on curve</td>
<td>233</td>
<td>209.2</td>
</tr>
<tr>
<td>Next min &quot; &quot; &quot;</td>
<td>240</td>
<td>187.7</td>
</tr>
<tr>
<td>Final max on curve</td>
<td>247</td>
<td>212.4</td>
</tr>
<tr>
<td>Zero after dismantling</td>
<td>256</td>
<td>- 5.41</td>
</tr>
<tr>
<td>Ring</td>
<td>Six segments - ordinary iron</td>
<td>Twelve segments - SG iron</td>
</tr>
<tr>
<td>---------------------------</td>
<td>------------------------------</td>
<td>---------------------------</td>
</tr>
</tbody>
</table>
|                           | (i) Time (ii) Time (iii) Pressure (iv) Corrected pressure | (v) Time (vi) Pressure (vii) Corrected pressure | (viii) (vii)-(iv) x 100/
|                           | (i) Time (ii) Time (iii) Pressure (iv) Corrected pressure | (v) Time (vi) Pressure (vii) Corrected pressure | (viii) (vii)-(iv) x 100/
|                           | Days after erection | tons/ft² | Tons/ft² | Days after erection | Tons/ft² | Tons/ft² | (iv) |
| Tunnel 4 days old         | 4                | 1.27    | 1.27    | 4                | 1.28    | 1.28    | + 0.8 |
| Before 2nd 3.8m (12ft) drive | 38              | 1.57    | 1.56    | 36               | 1.50    | 1.49    | - 4.5 |
| Imm after 2nd drive       | 42              | 1.78    | 1.79    | 40               | 1.83    | 1.82    | + 1.7 |
| 5 days after 2nd drive    | 47              | 1.80    | 1.81    | 44               | 1.83    | 1.82    | - 0.5 |
| 3 weeks " " "             | 62              | 1.78    | 1.79    | 59               | 1.86    | 1.84    | + 2.8 |
| Before approach of 2.2m (7ft) pilot | 215             | 1.80    | 1.84    | 213              | 1.97    | 1.90    | + 3.2 |
| Next max on curve         | 233             | 1.95    | 2.00    | 230              | 2.12    | 2.04    | + 2.0 |
| Next min " "              | 240             | 1.75    | 1.80    | 237              | 1.89    | 1.81    | + 0.5 |
| Final max on curve        | 247             | 1.95    | 2.03    | 244              | 2.14    | 2.06    | - 1.5 |
| Zero after dismantling    | 256             | -0.051  | 0       | 264              | 0.09    | 0       | 0     |
ordinary iron ring appears somewhat high and is probably due to gauge failures between 16 and 27 July. The change that occurred when the second 3.8 m (12½ ft) ID tunnel was driven past the instrumented rings was about a third less in the ordinary iron ring than in the SG iron one. By 21 August the changes have balanced out and both rings were indicating the same pressure. The small bumps in the curves close to the time the second drive passed are real measurements since all the gauges were working at this time.

During the 150 days before January 20th when the first effects of the 9.0 m (29 ft 6 ins) ID tunnel occur there is an increase in 'corrected pressure' of 5.4 kN/m² (0.05 tons/ft²) for the ordinary iron ring and 6.43 kN/m² (0.06 tons/ft²) for the SG iron ring. These figures represent a very small rate of increase in pressure, bearing in mind that at the start of the period the tunnel was only two months old and that only 45 per cent of the overburden value had been reached.

3.4.5.4 Conclusions drawn from first measurements of instrumented rings At this stage it was already possible to conclude the following from the experimental work:

A tunnel of 3.8 m (12½ ft) internal diameter had been driven in London Clay at a depth of 22 m (72 ft) with its edge one sixth of its diameter from another tunnel of similar size and at the same level without damage to the first tunnel.

It was not necessary to 'break joint' the lining of the first tunnel. In this tunnel lining of six segments and a key and of twelve segments and a key, both with continuous longitudinal joints,
behaved in a substantially similar manner.

Distortion of the experimental twelve-segment, SG iron ring was about 40 per cent greater than that of the normal six-segment ring or ordinary grey iron. This is quite acceptable and safe.

Nearly all the disturbances of the six week old first tunnel occurred over that length which was between one tunnel diameter ahead and one diameter behind the face of the second tunnel. The hoop thrust in the first tunnel was increased by about 10 per cent of the full overburden acting hydrostatically.

3.4.6 Measurement of instrumented rings in the tunnel - last part

Some months after the pair of twin 3.8 m (12½ ft) tunnels had been completed a 'break up' was made from them; that is segments were removed from one of the small tunnels and a cavity constructed outside them by hand mining and timbering. When the cavity was large enough two rings of 9 m (29 ft 6 ins) bolted iron lining were erected in it and grouted. This was a long way from the instrumented rings and did not affect them. A face was driven towards the instrumented rings as shown in Fig 34. A 2.2 (7 ft) ID pilot tunnel was driven ahead of the main face for a distance of 11.3 m (36 ft 9 ins) or 22 rings. It occupied the upper part of the main face and made it easier to control and also spread out the work force so that it could work more efficiently.

The instrumented rings were by now in a state of near equilibrium with the clay surrounding them, and the strain gauges were giving virtually unchanging readings. Perhaps they could be regarded as load cells.
situatc in the ground ahead of the advancing 9 m (29 ft 6 ins) face - a very desirable piece of instrumentation. If the effects of the 2.2 m (7 ft) pilot tunnel could be separated from those of the main face something new might be learned.

The 9 m (29 ft 6 ins) tunnel was built by hand, without a shield or segment erecting gear (see Fig 34). Consequently progress was much slower than with the 3.8 m (12½ ft) tunnels and the effects of excavation on the instrumented rings are spread over quite a large part of the deformation v time graphs (Figs 44, 45 and 46). Although the 2.2 m (7 ft) face was advanced at the same speed, it only caused marked changes for a few metres, and the time in which these happened was only a few days.

3.4.6.1 Diameter measurement

In order to measure the maximum deformation due to the passing of the 2.2 m (7 ft) ID tunnel and to isolate its influence from that of the 9 m (29½ ft) cross-over tunnel, we also measured the oblique diameters which, when produced, passed through the centre of the 2.2 m (7 ft) ID tunnel (see Fig 33).

The 2.2 m (7 ft) ID tunnel, located 30° to the right of the crown of the 3.8 m (12½ ft) tunnel, stands in the same angular relation to the right top ordinary iron segment as the second 3.8 m (12½ ft) ID tunnel stood with respect to the right hand axis level segment. There is a similar correspondence with the positions of the joints in the SG iron ring.

Due to the small tunnel passing 0.9 m (3 ft) away the increase in the oblique diameter of the ordinary iron ring was 2.1 m (0.085 in.) and that
of the SG iron ring was 2.5 mm (0.100 in.); in each case this was about a quarter of the amount by which the horizontal diameter increased when the second 3.8 m (12½ ft) ID tunnel was driven (Table 5, page 166/167).

The deformation was local and had little effect on the vertical diameters.

The crown of the ordinary iron ring might have been expected to move obliquely with the centre of the top segment, but in fact most of the deformation must have occurred as bending of the top segment because the vertical diameter of the ordinary iron ring only increased 0.1 mm (0.004 in.). The decrease in horizontal diameter was 0.5 mm (0.019 in.). The SG iron ring with more freedom due to its twelve joints decreased in both orthogonal diameters: 0.3 mm (0.012 in.) vertically and 0.7 mm (0.030 in.) horizontally.

Almost all the changes in the oblique diameter of each observed ring occurred when the 2.2 m (7 ft) diameter face was very close to the ring, between 2.2 m (7 ft) before it and 2.2 m (7 ft) after it. However, the first movements due to the main face had occurred much earlier. In the case of the ordinary iron ring the vertical diameter began to increase soon after 20 January. At this date the horizontal diameter was not changing.

The main face was about 27 m (90 ft) away, equal to the distance from the main tunnel invert to the surface. A similar reversal of squatting type of deformation occurred in the SG iron ring but the change from decrease to increase in vertical diameter is less definite though it probably occurred when the face was further away. Figure 44 also shows only very flat peaks in the curves for the horizontal diameters of both rings.

For each ring the horizontal diameter vs time curve steepens markedly when the main face was one and a half diameters away on 7 and 21 February and
the 2.2 m (7 ft) ID pilot tunnel had not yet arrived. However, the vertical diameter curves steepen on 17 and 28 February when the main face was just one diameter away and the 2.2 m (7 ft) face was about 2.2 m (7 ft) past each observed ring. From the above dates until the last reading was taken each diameter changes about 2.5 mm (0.1 in.). The last reading was taken with the previous 3.8 m (12½ ft) ID ring still erected and the working platform protruding into it. Since different gangs worked in different ways the readings are not strictly comparable. The unmeasured movement at the very last stage may have been considerable because the muck was usually removed from above the 3.8 m (12½ ft) ID lining before it was taken from the sides and underneath.

3.4.6.2 Strain gauge measurements

(i) As the 9 m (29 ft 6 ins) tunnel was approaching

Three polar diagrams e, f and g have been drawn in Fig 47 for each ring to illustrate changes of curvature that occurred whilst the 9 m (29 ft 6 ins) tunnel was being built. The dates of the readings chosen to give the differences plotted for 'e' and 'g' are somewhat arbitrary; they are marked on the time graphs in Figs 45, 46 and 48 for the sake of completeness. However, the dates marking period 'f' (included in the time covered by 'e') delimit the period when very definite changes due to the 2.2 m (7 ft) tunnel were superimposed on those due to the approaching 9 m (29 ft 6 ins) face. The most striking features of the data collected at this time are that the hoop stress v time curves (Fig 48), both have double peaks; also the two increases in hoop stress were not accompanied by squatting of the tunnel. Indeed the reverse movement (peaking) had started in both rings before the final increase in hoop stress began.
This suggests that the horizontal stress in the ground ahead of the 9 m (29 ft 6 ins) face was increasing for a while before the final reduction in vertical stress at the very end of the measurements. The other data are consistent with this hypothesis.

The first increase in hoop stress began shortly after 20 January when the 9 m (29 ft 6 ins) face was about 27 m (90 ft) away and was the first indication of its approach. The trough between the two peaks in each of the hoop stress v time curves represents a temporary relief of ground pressure due to the excavation of the 2.2 m (7 ft) tunnel. This began when its face was 2.2 m (7 ft), or one pilot tunnel diameter, away from each of the observed rings. The main face was 13.5 m (1 1/3 main tunnel diameters) away. In the six-segment ring, the top segment, which was adjacent to the 2.2 m tunnel, bent negatively in its centre moved outwards as the ground yielded towards the approaching 2.2 m (7 ft) face.

After the face had passed the six-segment ring, and a few 2.2 m (7 ft) rings had been built and grouted, the instrumented top segment was bent the opposite way even though the tunnel was peaking, not squatting. Also the hoop stress was increasing again. The top segment was now no longer opposite yielding ground but was being pressed towards a rigid tunnel. Meanwhile the axis level segment of the same six-segment ring had been bending more positively as it was pushed towards the second-built 3.8 m (12 1/2 ft) tunnel, see Figs 45i and 45 ii.

For comparison of the effects of the 2.2 (7 ft) face on the two different kinds of 3.8 m (12 1/3 ft) ring, the two polar diagrams 'f' of Fig 47 have been plotted. These show the changes in curvature of all the segments.
Fig 49  Behaviour of the top quadrant, adjacent to the twin tunnel, in the twelve segment ring
from the time when the 2.2 m (7 ft) face was 2 m (6.2 ft) away until it was 2.5 m past. The arc where significant changes occurred is, of course, occupied by four segments in the twelve-segment ring. The curvature vs time graphs for these four segments (Figs 46, i–iv) contain fluctuations which do not occur in Figs 45, i and ii for the six-segment ring. This is because the short segments had more freedom for angular movement in the plane of the ring. If hinging occurred at a longitudinal joint which had formed a line contact at its intrados, the contact could suddenly move to the extrados and vice versa. In this event the line of thrust would move by a large amount, and a large change of strain gradient would occur.

Detailed examination of the strain gauge results enabled Fig 49 to be drawn. The segment with its lower joint at axis level is of particular interest. The axis level joint would have opened at the extrados when support was lost as the second 3.8 m (12.5 ft) tunnel was driven and the horizontal diameter increased by 12 mm (0.466 in.). At the other end of segment the shoulder joint was similarly open because of the way the twelve-segment ring is built (Fig 39) with the upper joint open and filled with grout. This grout maintained the line contact so that both joints now had line contacts at the intrados* accounting for the particularly large negative bending change that accompanied the driving of the second 3.8 m (12.5 ft) tunnel. With both joints open this segment would have been in a state of limited instability like that of a voussoir in a deformed arch (Pippard et al, 1936). A slight redistribution of pressure on the outside of the tunnel would cause one joint to close and precipitate the opening of

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*Actually at the bottom of the caulking groove. Also, due to the flexibility of the longitudinal flanges only the parts near the corners would have transmitted thrust (see p 152).
another. Hence the line of thrust would change resulting in the sudden bending changes that were observed. Bearing this mechanism in mind it may be seen that the opening and closing of joints can account for all the changes in bending strains that were observed. The moment at the joint resulting from the tension in the bolts would have had little additional effect (see section 3.3.5 on joints).

The peak hoop stresses correspond in both cases to 50 per cent of the pressure that would be exerted by the full overburden acting hydrostatically, 430 kN/m² (4 tons/ft²). About half of this was lost during the remaining 4.5 m of the main 9 m (29 ft 6 ins) tunnel drive (see Fig 48).

(ii) Strain gauge measurements as the rings were dismantled
Although half of the hoop stress was lost before the ring was uncovered and dismantled, the greatest part of the curvature caused by the second 3.8 m (12½ ft) tunnel passing (diagram 'c') was only released at the last moment. The polar diagram 'h' for the ordinary grey iron shows this quite clearly. Diagrams 'g' and 'h' for the SG iron ring show irregularities though. It should be remembered that the release of earth pressure from the backs of these segments would cause additional positive bending whereas spurious negative bending would occur when their thin springy flanges recovered from the effects of the shield thrust.

3.4.6.3 Conclusions drawn from the later measurements of instrumented rings
Before examining these data further, the more obvious results will be listed:

A large tunnel (9 m, 29 ft 6 ins, ID) at shallow depth (three
diameter to invert) affected the ground ahead of the face at a
distance equal to the depth of the invert.

The hoop stresses in the two instrumented rings increased and
reached a peak when the large face was at a distance from them
equal to half its diameter.

The 2.2 m (7 ft) ID pilot tunnel (No 3 tunnel Fig 33) affected the
instrumented rings from 2.2 m (7 ft) before to 2.2 m (7 ft) after
it was opposite them.

This tunnel (No 3, Fig 33) caused a temporary reduction in hoop
stress when passing whereas the other 3.8 m (12½ ft) ID tunnel
(No 2) did not.

The strain gauges for the ordinary iron ring of six segments show
smooth curves as No 3 tunnel passed, but those for the SG iron
ring of twelve segments do not.

3.4.7 Discussion of the results obtained from the instrumented rings
in the tunnel

3.4.7.1 Aspects common to both tunnels
The two observed rings behaved in similar ways. The rapid development of
hoop stress and the 'squatting' agreed with other observations on tunnels
in London Clay. Observations of diametrical changes very early in the life
of a tunnel are not easily made so there are few records for comparison.
The author has measured early changes in a 6.5 m (21 ft 2½ ins) bolted and
grouted cast iron tunnel. The horizontal diameter decreased by 1.2 mm
(0.050 in.) during the first three days; the tunnel then began to squat
normally. This initial movement (called 'peaking' by tunnellers) has not
been seen in expanded linings despite careful measurements by ourselves and
others (K Smyth-Osborne, H Parker, personal communications). Possibly the
self-weight of the beds above the excavation causes them to separate along
the bedding planes. In the London Clay these are horizontal and commonly
have micaceous partings. When the grout has hardened and the lining starts
to accept the pressure of the ground this would be much more compressible
above the tunnel than elsewhere, and until the beds had closed again the
crown of the tunnel would move upwards and allow the sides to come in.
Separation along bedding planes would be much more reversible than
separation along 'fissures' which are irregular and tend to interlock. The
process would be less likely with expanded linings because the ground is
unsupported for a shorter time and the very operation of expanding the
rings may reclose any openings at the bedding planes. The possible role
of the grout itself is discussed later.

Hoop stresses and changes in diameter occurred at a very much smaller rate
after the twin tunnel had been driven than before. This contrasts with
observations on a pair of tunnels half a diameter apart, in London Clay
(Ward and Thomas, 1965). Then the first tunnel resumed deformation at a
very similar rate to that observed before the second drive. The increase
in hoop stress in the rings corresponded to about 10 per cent of the full
overburden pressure, as in the 1965 example. In both examples the
second tunnel was driven about six weeks after the first one. The
fairly sudden increase in hoop stress associated with a passing twin
tunnel is probably due to a dilation mechanism (discussed on pages 212/213)
together with swelling of the clay on the surfaces of fissures that have opened. Pressure could be transmitted through the blocks of the dilated clay mass immediately after disturbance; but after a while the boundaries of the blocks would soften and the capacity to transmit stress would be impaired. Eventually the voids would close and the clay would become somewhat more homogeneous and once again the field stress in the more distant undisturbed clay could start to be transferred to the tunnels. The interval before this happened would probably be longer in the more disturbed ground associated with the more closely spaced pairs of tunnels at Brixton.

The 2.2 (7 ft) tunnel was driven above the two 3.8 m (12½ ft) tunnels through ground which had not only been disturbed by them but was also being disturbed by the forward-reaching effects of the main 9 m (29 ft 6 ins) face. There was similarity between its effects and those of the twin 3.8 m (12½ ft) tunnel in that they began one diameter (2.2 m tunnel) ahead, and finished one diameter after, the face had passed. The differences were that the small tunnel temporarily reduced the hoop stress in the instrumented ring and that the bending was more localised.

The first increases in hoop stress in the instrumented rings due to the main face were seen when it was about 27 m (90 ft) away, a distance equal to the depth of the main tunnel invert. Surface subsidence is usually found to begin one tunnel depth ahead of a face in London Clay. The consultants state that from their settlement observations at Brixton it appears that the first effects on the surface occurred at a similar horizontal distance ahead of the tunnel face. * The same observation is

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* Personal communication from Mr T O'Donnel, of Messrs Mott, Hay & Anderson.
reported for the plastic clay at San Francisco (Peck, 1969). It is quite likely that the tension zone at the lip of the subsidence trough would be underlain by a compression zone.

At half this distance, or one and a half times the diameter of the main 9 m tunnel the horizontal diameters of both rings began to decrease quite suddenly. The same rate of change was maintained. The first observation suggesting movement had been one of bending strains in the twelve-segment ring somewhat earlier.

It has been stated (p. 189) that the later observations ahead of the 9 m (29 ft 6 ins) diameter face are compatible with the mobilisation of a horizontal pressure in the clay. Mechanisms which might produce this could depend on the following possibilities:

(i) the stress field already existing in the ground may be anisotropic;
(ii) the ground itself is anisotropic, particularly in its elasticity and permeability;
(iii) the collar of grout has different properties in different places around the ring.

On (i) most workers believe that in London Clay the horizontal stress exceeds the vertical stress; but this is derived from laboratory observations (Skempton, 1961; Bishop et al, 1965) and has not been measured in-situ.

On (ii) the bedding of sediments is the most obvious feature. The mechanism postulated on p 193 may have been operating around and ahead of the 9 m (29 ft 6 ins) face. Increase in hoop stress in the 3.8 m (12 1/3 ft) rings
accompanied both their own building and the approach of the 9 m (29 ft 6 ins) face.

A likely mechanism due to (ii) and (ii) above is that excess water from the grout migrates into the clay more quickly horizontally than vertically because of the horizontal silty partings. This would allow more rapid swelling of the clay and hardening of the grout at axis level than at invert and crown. When all the excess water from the grout had been used up this process would gradually reverse, the clay near axis level tending to consolidate by migration of water upwards and downwards. This would accord with the normal 'squatting' that follows.

A secondary effect of this process is that the water/cement ratio of the grout at the sides of the tunnel would decrease. Since there is excess water in the grout this would hasten the setting here and allow the pressure of the ground to be transmitted to the sides of the tunnel earlier. The process would have been exaggerated if the cement of the grout settled out. However, examination of the grout surrounding the 3.8 m (12½ ft) rings, page 205, showed little evidence of this. This does not mean that an effect due to (i) was not operating. The horizontal movements 1½ diameters ahead of the 9 m (29 ft 6 ins) face would seem more likely to be due to (i) while those within half a diameter of it are more likely to be due to (ii) and (iii).

3.4.7.2 Aspects concerning lining design

The most noticeable difference between the behaviour of the ordinary ring of six segments and the twelve segment SG iron rings is the greater diametrical deflection of the latter, both over the whole period and during the
two days when the other 3.8 m (12½ ft) tunnel was driven past. This has not, however, caused large differences in the bending strains measured in the two kinds of segment although they are of approximately similar stiffness. The implications of this depends on the mechanism of the interaction between the clay and the lining, which is not clearly understood.

If the lining is the dominant factor controlling the deformation then the inequalities of pressure distribution around the ring will cause the segments to bend until a particular bending resistance is reached which is in equilibrium with the forces applied by the clay. Approximately the same bending resistance is mobilised in the short segments as the long ones. However, the SG iron ring is more likely to form partial hinges because the cross joint flanges are less stiff and the cross joint faces are narrower; there are also twice as many of these joints. Thus a larger total deflection of the ring accompanies the equilibrium state.

On the other hand the clay may be dominant. The interface between it and the lining would tend to form a particular shape as a new equilibrium was established in the clay after it had been disturbed by excavating the tunnel. In this case a stiff ring would modify the new shape somewhat, and in doing so, develop bending moments. A flexible ring should conform to the new shape without developing bending moments.

Although articulation of the SG iron ring allowed it to follow the overall shape taken by the clay, the individual segments had either to conform to changes in curvature or to develop line contact at their butt joints. Either would have resulted in the unexpected strain gradients that were observed.
Experimental verification that the SG iron ring was flexible enough to allow the clay surface to adopt its natural shape is that a nearly identical history of diametrical distortion was observed in the even more flexible ring with cuts in the circumferential flanges.

If this second explanation is appropriate then there is no objection to making the segments themselves more flexible since little additional squat would be likely to occur. The stiffness of the experimental segment is almost wholly derived from its deep flanges; hence these could be much reduced. Since for a given curvature the strains at the edge of the section are directly proportional to the distance between the edge and the neutral axis, this would result in lower stress at the flange tips. Also the resulting section would be more symmetrical and so more suitable for a component which may, in service, be bent in either direction.

From a geotechnical viewpoint the second hypothesis seems more likely, but a full-scale trial of a lining composed of segments with little or no flanges is required to demonstrate this.

The values of hoop stress in the lining which correspond to the full overburden acting hydrostatically are 25.3 MN/m² (1.64 tons/in²) for the ordinary iron ring and 48.5 MN/m² (3.14 tons/in²) for the SG iron ring; both values are small compared with the compressive strengths of the respective materials. The overburden value may be regarded as a maximum value of the average compressive stress in the lining of a properly designed and executed tunnel which is not involved in a failure of the ground. Hence the cross sectional area used in any future design could be considerably reduced provided that consideration had been given to its bending, buckling and handling aspects.
The cross joints did not all behave in the way that a bolted butt joint should behave. When the distortions were small and the hoop load enough (eg the ordinary six-segment ring during period b) it appears that the joints stayed closed and transmitted bending moments. However, at other times hinges formed and the line contacts made at the edges of the joints caused the bending to be exaggerated. The polar diagrams of Fig 47 suggest that, particularly in the twelve-segment ring, strain gradients near the joints were noticeably larger than those recorded by the gauges near the middles of the circumferential flanges. The strain gradients produced in this way could have been avoided by using knuckled longitudinal joints. The positions of these joints should be on the horizontal and vertical diameters of the ring, hence 12 or 8 segments is a satisfactory number.

A knuckle jointed ring of more than three segments would form a more flexible ring than one with butt joints that actually functioned in the manner expected of them. However, on the same argument as that presented above for less stiff segments, it is not likely that in London Clay the use of fully flexible joints would result in unacceptably large diametrical distortions when the ring squatted. Nor is it likely that the bending moments at the centre of the segments would be particularly high.

3.4.8 Errors in strain measurement

3.4.8.1 Causes

Errors may have arisen in the strain gauge readings from causes in either (i) the iron; (ii) the gauges; (iii) the reading equipment.

(i) The iron

Possibilities of dimensional changes in the iron casting may arise from
(a) dissipation of locked up stresses; (b) stressing beyond the yield point in handling or service; (c) elastic distortion giving strains similar to those under investigation but actually of a different nature.

It was found that:

(a) readings taken in the laboratory showed that the castings were stable;
(b) such over stressing was confined to the twisting by the shield rams of the flanges of the ordinary flake graphite iron segments;
(c) the shield rams have similarly affected the thin springy flanges of the Si iron segments - but reversibly. Also pressure on the back affected the curvature in the flanges.

(ii) The gauges
The vibrating-wire element of the strain gauges may corrode. If it does its mass and hence its natural frequency of vibration changes and a false reading is obtained. After the experiment the wires were inspected for such corrosion and in the case of a few invert gauges it was found. Creep of the wire and slipping at the anchorages can be, and was, avoided (Thomas, 1966).

(iii) The reading equipment
Every time the frequency of the gauges was read reference was made to a frequency standard. This standard is an electrically excited tuning fork which, of course, vibrates most strongly at a single frequency - its fundamental frequency of natural vibration. However by comparing this frequency with fractions or multiples of it a large number of reference frequencies is available. The comparison is easily seen by adding the two
frequencies at right angles using a cathode ray tube to observe appropriate
Lissajous figures. The oscillator used to measure the gauge frequencies
was not found to drift provided that it was switched on for about an hour
before it was used.

3.4.8.2 Errors found
As a check on drift or damage in the vibrating-wire strain gauges, or
dimensional instability or damage in the segments, readings were taken in
the tunnel before the rings were built and after they were dismantled.
Zero readings were also taken at the Building Research Station before the
experiment, after return from the tunnel and three months after that.
Change during transport was nil, drift after three months was small and
only occurred in some invert segments.

However, zero changes in the tunnel were quite large, they are shown in
Table 8 for the ordinary iron ring in which the largest strain measured was
541 microstrain, and in Table 9 for the SG iron ring in which the largest
strain was 432 microstrain. In order to see to what extent these changes
produce errors in the results, they have been plotted in terms of curva-
ture (Figs 37 a and b) as polar diagrams of the same form used for the
results in Fig 47.

Comparing the left hand column of Table 8 with the left hand side of
Fig 37b, it can be seen that, despite the strain errors, the bending errors
are small. Similar strain errors have occurred in both gauges of most
pairs and, since bending is proportional to the difference between the
strains, the strain errors, whatever their cause, are largely eliminated.
Grout which escaped from the pan flooded the invert during building. This made it difficult to dismantle the invert SG iron segments so they were probably damaged. Also, on dismantling the gauges, their vibrating-wire element were found to be corroded. The wires of the other gauges did not corrode. The segments in the upper right third of the SG iron ring were severely bent while the earth pressure was acting on their backs. Bending errors occur here because of instability of the SG iron segments in this condition. When the SG iron part of the tunnel was dismantled, turning gear was used to cut away the bolts. The thermal stresses this caused would impair the value of the data used to make diagrams 'g' and 'h' of Fig 47 for the instrumented SG iron ring.

In Fig 37a the positive bending errors are permanent spurious bending due to overstressing the ordinary iron segment flanges by the shield rams. Its mechanism is dealt with in detail under section 3.4.4: Effect of shield thrust on the segment flanges, also see page 175. However, some of the positive bending caused in this manner and shown in diagram 'a' for the ordinary iron ring was released slowly as is shown by the small amount of negative bending shown on the left hand side of the subsequent polar diagrams for this ring. The negative bending left in the right hand axis segment of the ordinary iron ring (Fig 37a) would be the result of overstretching in negative bending at the time of the second 3.8 m (12½ ft) drive.

Thus the changes in curvature for the particular stages in the life of the tunnel, after erection and grouting and before relief of load, shown in Figs 47 'b' to 'g' can be relied upon with a lot more certainty than Tables 8 and 9 at first suggest.
### TABLE 8 - Vibrating-wire gauges on ring of ordinary iron segments
Changes of zero readings from before until after field experiment

<table>
<thead>
<tr>
<th>Segment and gauge position</th>
<th>Segments on the left hand side</th>
<th>Segments on the right hand side</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Zero difference</td>
<td>Zero difference</td>
</tr>
<tr>
<td></td>
<td>microstrain</td>
<td>microstrain</td>
</tr>
<tr>
<td>Top</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Trailing</td>
<td>Tip</td>
<td>+ 16.2</td>
</tr>
<tr>
<td></td>
<td>Root</td>
<td>+ 10.2</td>
</tr>
<tr>
<td>Leading</td>
<td>Tip</td>
<td>+ 1.8</td>
</tr>
<tr>
<td></td>
<td>Root</td>
<td>+ 3.0</td>
</tr>
<tr>
<td>Axis</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Trailing</td>
<td>Tip</td>
<td>- 22.8</td>
</tr>
<tr>
<td></td>
<td>Root</td>
<td>- 42.6</td>
</tr>
<tr>
<td>Leading</td>
<td>Tip</td>
<td>+ 24.0</td>
</tr>
<tr>
<td></td>
<td>Root</td>
<td>+ 6.0</td>
</tr>
<tr>
<td>Invert</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Trailing</td>
<td>Tip</td>
<td>- 53.4</td>
</tr>
<tr>
<td></td>
<td>Root</td>
<td>- 13.6</td>
</tr>
<tr>
<td>Leading</td>
<td>Tip</td>
<td>- 64.2</td>
</tr>
<tr>
<td></td>
<td>Root</td>
<td>- 28.8</td>
</tr>
</tbody>
</table>

### TABLE 9 - Vibrating-wire gauges on ring of SG iron segments
Changes of zero readings from before until after field experiment

<table>
<thead>
<tr>
<th>Segment and gauge position</th>
<th>Segments on the left hand side</th>
<th>Segments on the right hand side</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Zero difference</td>
<td>Zero difference</td>
</tr>
<tr>
<td></td>
<td>microstrain</td>
<td>microstrain</td>
</tr>
<tr>
<td>Top</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tip</td>
<td>+ 6.0</td>
<td>- 9.6</td>
</tr>
<tr>
<td>Root</td>
<td>+ 1.8</td>
<td>+ 6.0</td>
</tr>
<tr>
<td>Shoulder</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tip</td>
<td>+ 10.8</td>
<td>- 82.0</td>
</tr>
<tr>
<td>Root</td>
<td>+ 16.2</td>
<td>- 1.8</td>
</tr>
<tr>
<td>Upper axis</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tip</td>
<td>- 7.8</td>
<td>+ 55.0</td>
</tr>
<tr>
<td>Root</td>
<td>- 8.4</td>
<td>+ 10.0</td>
</tr>
<tr>
<td>Lower axis</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tip</td>
<td>- 18.0</td>
<td>- 53.0</td>
</tr>
<tr>
<td>Root</td>
<td>- 11.4</td>
<td>- 50.0</td>
</tr>
<tr>
<td>Knee</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tip</td>
<td>+ 6.0</td>
<td>- 13.0</td>
</tr>
<tr>
<td>Root</td>
<td>- 3.6</td>
<td>- 9.0</td>
</tr>
<tr>
<td>Invert</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tip</td>
<td>+ 25.8</td>
<td>+ 54.0</td>
</tr>
<tr>
<td>Root</td>
<td>- 10.8</td>
<td>+ 9.0</td>
</tr>
</tbody>
</table>
Figure 37  Zero readings of the vibrating-wire strain gauges: change from before to after field experiment, expressed as bending (c.f. fig 47)
Calculation of the stress at the neutral axis in individual segments however includes the errors shown in Tables 8 and 9; but, since they are nearly random in sign and magnitude, averaging the stress round each ring gives consistent and useful results. See Table 7 in the section 3.4.5.3 on 'Hoop stress' which shows that the results from the two rings give similar values of hoop stress at corresponding times in the histories of the two rings.

3.4.8.3 Conclusions about errors
Hence it is concluded that the strain gauge results though not free of errors may be used:

1. for calculating the average hoop stress in the ring by averaging the values calculated for the stress at the neutral surfaces of all the segments in a ring;

2. for calculating the local bending strains because this depends on the difference in strain between adjacent gauges and these had similar errors. However, care must be taken to see that strain differences due to twisting of the flanges by the shield rams are not mistaken for circumferential bending of the whole segment; also that circumferential bending of the flanges by movement of the skin of the segment is not interpreted as bending applied from outside the segment, when merely due to a change in uniform pressure.

3.4.9 Tunnel grouting and its effect on the instrumented rings
The tunnel at Brixton was grouted in the usual way with neat cement grout pumped at low pressure after each shove.

When the 4.04 m (13 ft 3 ins) OD pilot tunnels were dismantled to build the
Fig 50a Thicknesses of grout layers around rings.
9 m (29 ft) ID main tunnel the instrumented rings were each carefully examined at the stage when their trailing edges were exposed and most of the muck had been removed from the top segments. Figure 50a shows the dimensions of the grout collars that were seen. A fairly even thickness of 38 mm (1 1/2 in.) had generally been maintained. The ground near the top of the tunnel had contained a stratum of clay stones and occasionally, when one of these had dropped out, grout had filled the space so that a bump was seen on the grout collar.

In some places at the top of the grout collar, when broken, showed internal voids, but no spaces were seen between the outside surface of the grout and the clay; nor did the clay appear to have moved into any such space. The voids seen within the grout layer appeared to have been formed by bubbles of air in the wet grout. However, they may originally have been 'bubbles' of water from which the cement particles had settled out. If so they were the only evidence seen of such settling. The rest of the grout collar had the same texture and density all round the ring.

A search was made for cracks in the grout collar, especially by the cross joints of the lining, but none were seen. There was no opportunity to see if cracks extended from the circumferential joint. Some of the cross joints had opened about 1.6 mm (1/16 in.) at the extrados and a thin wedge of grout had penetrated the cavity.

The grout typically broke away with 25 to 38 mm (1 to 1 1/2 in.) of clay adhering to the convex surfaces of the segments. The exposed surface of this clay showed signs of movement; it was slickensided in the direction
Polished and slickensided clay adhering to grout layer taken from the back of lining after its dismantling.
parallel to the axis of the tunnel and if some of the clay were removed it usually revealed other slickensided surfaces beneath. At the time the tunnel was built it was possible to see a length of six or seven rings contracting and expanding axially (in the manner of a concertina) as the shield rams were pressurised or exhausted, and the wood packing squeezed or expanded. Close to the shield where the grout was fluid the movement could occur freely but once the grout had hardened shear forces would be thrown onto the clay. Again when the tunnel was unloaded it expanded and the clay would have been sheared.

The instrumented segments had been painted with matt red paint instead of being dipped in bitumen. When the grout was broken off these segments an intact paint layer was left on both the outsides of the segments and the concave surfaces of the grout fragments. There was no evidence of differential movement between the lining and the grout collar.

Although the grout layer itself is not stiff in comparison to the flanged segment, it would, if perfectly bonded to the back of the flanged segment, produce a composite section of considerably enhanced stiffness. The second moment of area for the 3.83 m (12 ft 7 ins) ID ordinary cast iron segment is increased by one third, for the 3.86 m (12 ft 8 ins) ID SG iron segment it is increased by a half.

Similarly a well bonded layer of grout would also very considerably stiffen the thin skin of the SG iron segments in the longitudinal direction and so lessen the extent to which the skin fails to act with the flanges. The

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* A piece of grout with the polished and slickensided clay adhering to it was brought back to the laboratory and photographed. Figure 50a was taken some six hours after removal from the tunnel (the photographer does not work nights) by which time the clay had deteriorated somewhat.
grout could provide the equivalent of cross webs across the back of the segment.

It is now necessary to make some assessment of the adhesion between the grout and the segment and of the shearing force in their composite section tending to cause slippage between them. Normally cast iron segments are coated with a layer of bitumen about 0.5 mm (0.02 in.) thick and so are effectively lubricated for slowly applied shear. The paint on the instrumented segments was a thin layer and had been applied to a surface which was rough, both because of casting irregularities and sand inclusions. Pull out tests on smooth steel bar cast into concrete show initial bond strengths of around 1380 kN/m² (200 lb/in²) and preliminary tests show that painted steel bars adhere even better. Tensile tests on paint films have given strengths of the order of 6.89 x 10³ kN/m² (10³ lb/in²) for metal surfaces, 6.89 x 10² kN/m² (10² lb/in²) for concrete surfaces but rather less for plaster surfaces. There is some suggestion that very wet mixes such as plasters and grouts may allow a porous paint to soak up alkaline waters which would cause it to deteriorate.

If the lining was a continuous cylinder the slight distortion from a circular cross section to an elliptical one would cause bending that was so close to simple circular bending that shear forces would be negligible. In the segmental lining, although the absolute values of bending moment are small, there are changes of curvature in the iron-grout composite ring at the cross joints and hence increased local shear stresses here. Between the cross joints, however, the rate of change of bending moment around the segment and hence the shear forces, is probably small. Along the tunnel, between the circumferential flanges, a similar situation occurs: the thin
skin of the 3G iron will dish giving a parabolic distribution of bending moment causing no shear force at the centre but maximum shear force at the edges. These maximum values at the edges are of the same order as the adhesion between the iron and the grout so it seems likely that adhesion over most of the area of the segment will be maintained hence the instrumented segments will have been somewhat stiffened by the grout.

The above argument does suggest that should a stiff tunnel be required for a particular application the segments should not be bitumen coated but be provided with shear connections to the grout.
REFERENCES TO CHAPTER THREE


CHAPTER FOUR :

MOVEMENT OF THE GROUND NEAR THE FACE :

ITS AMOUNT, CAUSES AND CONTROL

The shield used for driving both the running tunnels was of the Greathead type with a Dalrymple-Hay hood. The overall length was 3.17 m (10 ft 6\(\frac{1}{2}\) in) and the outside diameter of the skin plate was 4.11 m (13 ft 6 in). The cutting edge was fitted with a bead 10 mm (\(\frac{3}{8}\) inch) wide so that the largest dimension of the hole it cut was 4.13 m (13 ft 6\(\frac{3}{4}\) in). As is usual in order to prevent the shield dropping as it is driven, the bead was omitted from the bottom 60° of cutting edge. The shape of the hole it cut was fairly regular, see Fig 50a showing the shape of the grout layers around two of the rings. The shield had twelve shove rams and four face rams. Its progress was usually five 0.51 m (20 in) long rings per eight hour shift and three shifts were worked on the five working days of a week. The second tunnel was driven about two months after its twin, as is the usual and prudent practice.

Beyond the length where the twin 3.8 m tunnels were driven close together in order to act as pilot tunnels for the 9 m (29 ft 6 in) cross-over tunnel they diverged until they reached the next working shaft at Ferndale Road, Brixton. Here the shields were each driven into concrete eyes constructed in the ends of lengths of 5.05 m (16 ft 6 in) tunnel. These had been built by hand on the line of the running tunnels and were connected to the shaft by cross passages. In order to accommodate these various tunnels around the shaft, the two running tunnels had to be over 7.6 m (25 ft) apart at the shaft. With this spacing it was possible to
use the first driven tunnel as an observation position and reference point to observe the movement of the ground near the second driven tunnel. Normally twin tunnels are too close.

The 5.05 m (16 ft 6 ins) shield chambers made suitable observation points for observations of movement of the ground in front of an approaching tunnel face. The eye itself had been close timbered and grouted at the time of construction. A 32 mm (1 1/4 in.) diameter hole was drilled for 6.1 m (20 ft) on the line of the axis of the approaching tunnel. Into this was inserted a steel rod 8 m (26 ft) long and it was driven 15 cm (6 ins) into the ground at the far end of the hole in order to anchor it there. The end which protruded into the chamber lay in a bush mounted on a stout steel beam which was bolted to the 16 ft 6 ins cast iron lining. The movement between the bush and the end of the rod was measured with a depth micrometer to an accuracy of 0.025 mm (0.001 in.). At the same level two other such rods were installed, one towards the side of the chamber so that it was in line with the cutting edge of the approaching shield and one on the other side angled so that it was 15 cm (6 ins) outside the cutting edge but still on the horizontal diameter.

A colleague observed the progress of the shield in the approaching tunnel and noted the time of each shove. These times were later correlated with the times of the micrometer readings to produce a scale of distance rather than time in Fig 51.

---

*When the shield arrived it was 57 mm (2 1/4 ins) off line so that this point was 209 mm (8 1/4 ins) outside the cutting edge and the other point a little inside it.*
Fig. 51 Axial movement of the ground ahead of an advancing face
Figure 51 shows that the first observation was that the central measuring point (A) was moving away from the advancing face. When the distance between this measuring point and the face was reduced to 4.6 m (15 ft) its direction of movement changed. Thereafter its movement towards the face accelerated as the face came nearer; half the total movement of 17.5 mm (0.7 in.) occurred in the last metre of driving.

The movements at the edge of the excavation (points B and C) followed the same pattern but were very much less. The face bulged at the centre but was almost held stationary 20 cm (8 ins) beyond its edge. The volume of this bulge, assuming it to be a segment of a sphere is $0.116 \text{ m}^3 (4.1 \text{ ft}^3)$. Thus for every 4.6 m (15 ft) of drive (the distance over which the bulge is developed) $0.116 \text{ m}^3 (4.1 \text{ ft}^3)$ more ground is mined than the volume of the hole. This gain is due to:

1. elastic expansion of the ground as the overburden pressure is relieved (ie elastic movements not due to the weight of the ground);

2. formation of a depression at the surface (ie elastic movement due to the self-weight of the ground);

3. dilation of the ground behind the face by the formation of voids.

The bulge forms quite quickly (about 16 hours) so it is clear that (3) is not 'swelling' in the geotechnical sense of a volume change due to more water moving into the interstices between the mineral particle. The clue is in the big difference between the displacement at the centre of the face and that at the edge; this implies considerable shearing of the clay and a tendency for tension at the actual surface. Because the fissures are not plane surfaces, movement along them causes them to be opened, that is for air voids to occur where previously there were only planes of
weakness. Now that the fissure planes no longer interlock, stresses more easily cause movement along them. Referring to the mass of the material, one may say that once the clay is disturbed the modulus of elasticity is reduced. This in itself facilitates disturbance causing a further reduction in the modulus. Thus the reduction in modulus is time-dependent.

Returning to the initial shearing causing the first dilation of the clay, if the clay is sheared in a confined condition the average pressure between adjacent blocks will increase, that is a compressive stress will develop in the whole clay mass. When in the field experiment the first movement of point 'A' was observed to be away from the shield, it was thought that this was due to pressure from the cutting edge; but that this was not so, was soon indicated by the very small movement at point 'B' immediately in front of the cutting edge. It now seems possible that shearing by the shield gave rise to a compressive zone in the ground between the shield and point 'A' which deflected it away from the shield.

Although it is obvious that shield cutting edge cases severe shearing of the clay another mechanism is postulated which might account for the dilation of the clay around the skin of the shield (the observations of which are to be described later). Since the clay is a sedimentary material it is orthotropic in its elastic properties, that is to say the modulus of elasticity normal to the bedding planes is different from (probably less than) that in any direction parallel to the bedding (i.e. in this plane it is elastically isotropic). Therefore, if a piece of the clay were found

*In deep tunnels, when a break occurs in the work, the smooth surfaces left by the clay spade can be seen to develop fissures after about 45 minutes.*
in a condition of hydrostatic loading and then were released from this, it would strain more in the vertical direction than in the horizontal one. Hence in any direction other than these it would suffer shear strains. So if the London Clay at depth were in a state of hydrostatic stress under the action of the overburden, \((K_o = 1)\) merely releasing it from confinement would cause it to develop shear strains, and because of the uneven fissure planes, to dilate.

Some difficulty arises because some of the stress in the London Clay has already been released by geological removal of the overburden and \(K_o\) is probably greater than 1. This would mean that there would be a greater stress relief in the direction of the probable greater stiffness, horizontally, which would tend to equalise the horizontal and vertical strains, and reduce the shearing and consequent dilatance.

However, it may be that the reduction of the overburden load in the Pleistocene period itself initiated small movements along planes of weakness leaving the final unloading to continue the same trend and to actually open the fissure planes. That \(K_o\) is now greater than 1 would merely be concomitant and mean, not that shearing will be reduced, but that some shearing has already occurred. The decrease in fissure spacing with depth and the discovery that \(K_o\) becomes closer to 1 with depth (Skempton, 1961) supports this.

Unlike the modulus of elasticity, the shear strength of the clay mass is at first little changed because the fissure surfaces are hard and rough so that although the disturbance reduces the area of contact between blocks the pressure between them is increased. However, after the fissures have
been open for a day or two the clay starts to take up water (the pore water pressure had become negative when the overburden pressure was released) the effective pressure at the block-contacts falls and the whole mass becomes weaker. If free water is available, of course, the process is sped up and, also if there is enough softened clay to act as a lubricant between the blocks the strength of the mass is reduced even more quickly.

The method of working the face at Brixton at the time these measurements were made was firstly to erect a stage across the horizontal diameter of the face and from this drive the semi-circular top half of the face forward for a distance equal to the length of one ring 0.51 m (20 ins). This half was then open poled, the poling boards being supported by walings pressed forward by some of the face rams of the shield. The stage was then removed and the bottom half of the face advanced. The shield was then shoved to trim the excavation to its final shape.

This procedure relates to the mode of movement of the clay, discussed above, as follows: In a face as small as 4 m it is not hoped that the use of face rams will obviate subsidence at the surface. On the other hand it was not necessary to safeguard against plastic failure. This does not occur until the ratio of the overburden pressure, \( \gamma_h \), to the compressive strength of the clay, \( c \), is as high as about 6 (Broms and Bromanmark, 1967). At the place chosen at Brixton, \( c \) was about 320 kN/m² (3 tons/ft²) and \( \gamma_h \) was 430 kN/m² (4 tons/ft²) making \( \gamma_h/c \) only about 1.3. What is necessary with a clay face which is changing from an intact mass of clay to a wall of loosely connected blocks, with a stressed elastic material
Fig. 52 Radial convergence of the ground at the side of a passing tunnel
behind it, is to avoid having a ton or so of material thrown onto the
minors. This could happen with an unsupported clay face if some delay
occurred. Instances are known of large blocks bounded by joint surfaces
being ejected from the face. The possibility of failure due to water
trapped in sandy lenses or in gravel filled channels is discussed elsewhere.

Ground also enters the excavation by radial convergence mainly due to the
shield skin being of smaller diameter than the cutting edge and also because
after a shove there is a gap between the clay and the newly built ring of
lining.

Measurements of radial convergence of the approaching second tunnel were
made from the first tunnel at a distance suitably remote from the shaft to
avoid disturbed ground; at this point the 3.8 m (12 ft) running tunnels
were on the same level and the edge to edge distance between them was 7.6 m
(25 ft). Four holes 32 mm (1 1/4 in.) diameter were drilled from this
tunnel. Three were towards the line of the as yet undriven second tunnel.
They terminated at distances 0.45 m (1 ft 6 ins), 2.0 m (6 ft 6 ins) and
3.5 m (11 ft 6 ins) from the expected position of the edge of the new
tunnel. The holes were drilled at tunnel axis level each through the grout
hole of a side segment; a brass tube screwed into the grout hole
facilitated starting the hole in the correct attitude. It was later found
that the worst error in level at the other end was 6 inches.

The fourth hole was driven from the other side of the tunnel for a distance
of 6.1 m (20 ft). The bottom of this hole was to be a primary reference
point. Again steel rods were used to transmit the movements to the
secondary reference points which were collars attached to the grout holes
in the lining. The diameter of the tunnel was also measured with a micrometer rod so that any movement between the primary reference point and the secondary ones at the grout holes on the other side of the tunnel could be noted. Throughout the three days of the experiment there was no movement between the primary reference point and that side of the tunnel. However, the diameter of the tunnel did change a little — 0.6 mm (0.024 in.) over the three days. Appropriate corrections have been made for the small movements of the reference surfaces on the moving side of the tunnel before plotting Fig 52. The relative positions of the shield and measuring points were established when a cross passage had been made between the two tunnels as part of the civil engineering work.

The total displacements measured were 12.4 mm (0.489 in.) for the point 0.46 m (1 ft 6 ins) from the new tunnel, 7.1 mm (0.280 in.) for the point 2.0 m (6 ft 6 ins) and 2.8 mm (0.112 in.) for the one 3.51 m (11 ft 6 ins) away. Attempting to use those horizontal displacements, and the radii at which they were measured, to integrate around the tunnel axis to make comparisons of volume changes gives unsatisfactory results. For the volume changes near the tunnel to be larger than (or even equal to) those further away needs larger values of average displacement near the tunnel than those measured. This suggests that the deflections in the vertical plane containing the tunnel axis were considerably larger than those in the horizontal direction. This is to be expected from the fact that 6 to 10 mm (1/4 to 3/8 in.) surface subsidence is added to the vertical displacement which is due to elastic relief of stress and opening of the ground along bedding planes. Also, by the nature of a sedimentary material, these latter quantities are greater in the vertical direction than the horizontal one.
The volume of a subsidence trough 6 mm (1/4 in.) deep spreading one tunnel depth either side of the line of the tunnel and triangular in section would be 0.15 m³ per metre run (1.56 ft³ per foot run). If the average radial convergence of the hole were only as much as that measured for the horizontal direction and 0.46 m (18 ins) away, the volume gained (0.16 m³ per metre run or 1.7 ft³ per foot run) would exceed that lost at the surface. Bulking obviously occurs. As it was not possible to make vertical displacement measurements at Brixton, or measure the volume of clay extracted, one can but guess at the volume gained due to bulking. However, having seen the insignificant effect caused at the surface by really large voids left by bad tunnelling practice, the author's guess is that the volume due to bulking is about equal to that due to subsidence, i.e. half the convergence will appear as subsidence. A rough calculation of the elastic convergence, due only to the overburden pressure and neglecting self-weight, suggests this is a small proportion of the whole, 2.5 mm (0.1 in.) for the Brixton tunnel.

If the undoubtedly low figure of 0.16 m³ per metre run is multiplied by the length over which the bulge at the face was fully developed, the volume obtained (0.72 m³ or 25.44 ft³) exceeds the ingress at the face by a factor of 6. This suggests that prevention of convergence (about half the volume of which may manifest itself as subsidence at the surface) is much better achieved by attention to what happens around the shield than what happens at the face.

---

8 This estimate was made using thick ring theory (Timoshenko 'Strength of materials', vol II, p 241) and calculating the convergence of the inside of a ring of clay of ID that of the excavation and ID five tunnel diameters. Plane strain was assumed, Poisson's ratio taken as 1/3, E as 100 m/ft² (1000 ton/ft²) and the pressure inside the ring as zero and outside it as 430 kN/m² (4 tons/ft²) (equivalent to γh acting hydrostatically).
The main value of the measurements made in the horizontal plane containing the axis of the advancing tunnel which are depicted in Fig 52 lies then not in their absolute values but in the shapes of the graphs. Curve 'a' for the point nearest the line of the new tunnel shows that when the hood of the shield was 4.6 m (15 ft) away very little was happening. Soon afterwards when it was about 4.1 m (13\(\frac{5}{16}\) ft) away, the point began to move towards the projected axis of the approaching tunnel. The rate of displacement then increased until the cutting edge was opposite the point of measurement. (The zero mark on the 'feet ahead of hood' scale.)

It will be noticed that the next part of the displacement - shield distance curve is not smooth. This, of course, is because the shield was moving intermittently. The less steep parts of the curve represent the moves or advancement of the shield, whereas the steep parts represent the 'mucking' or excavation of the face.

After the bead had passed point 'a' the ground here began to lose support from the ground at the face. After about 1.2 m (4 ft) it was unaffected by what was happening at the face and was unsupported by the bead. It then moved towards the skin of the shield, quickly at first and then more slowly. After the tail of the shield (zero on the 'feet behind tail' scale) had passed the rate of displacement again increased. This suggests that the earlier decrease in the rate of displacement indicates that the tail plate was supporting the clay to some extent; the convergence while the shield was passing was 4.5 mm (0.18 in.), about half the thickness of the bead. (Often clearance can be seen between the clay and the tail plate at axis level while it is tight at the crown and invert.)
It appears from these curves that the clearance provided by the bead was appropriate to the speed of tunnelling. If it had been smaller the tail would have been trapped. It is also clear that the only way the convergence could have been lessened would have been to increase the speed of working or shorten the shield. The length of the shield is determined by:

(i) the working space ahead of the diaphragm
(ii) the length of the shield rams
(iii) the length of the ring.

(i) is roughly constant; (ii) and (iii) are related and larger rings are generally shorter, (0.45 m (18 ins.) for rings over 6.46 m (21 ft 2½ ins), compared to 0.6 m (2 ft) for the Fleet line running tunnels of 4.0 m (13 ft 2½ ins) diameter. Hence, in proportion to the total volume excavated, the loss of ground associated with a large shield driven tunnel is likely to be less.

With hand mining, the early radial convergence may be lessened by 'burying the hood'. With a tunnelling machine it is possible to have the excavating part set back into the shield and of course the speed of machines is potentially high.

These measurements confirm that the early practice of driving ahead of the face could cause large settlements - it effectively doubled the time for which the clay was moving.

The best available combination of these convergence reducing features for 4 m (12 ft) tunnels is the expanded lining used with a tunnelling machine. The shield length is reduced by omission of the tail plate and the lining
is in position and supporting the ground in a very short time.

Summarising this Chapter:
The ground converges axially and radially onto the cylindrical space cut by a shield in London Clay before it is lined.

Measurements of the movement in good ground have been made in the horizontal plane containing the axis of an advancing tunnel.

While the shield was passing a point in the clay near the edge of the advancing tunnel the ground started moving when the front of the shield was one tunnel diameter away and had virtually ceased when its tail was one diameter past.

The extra ground mined in the tunnel is more than (say twice) that lost by subsidence at the surface.

Considerably more (say six times) as much ground enters the excavation radially as enters from the face.

The mechanism causing the swelling of the ground (in the miners' use of the term) is probably dilatancy of the clay mass caused by movements along the fissure surfaces as the mass is sheared.

At depth it is likely that merely unloading the clay, as at the boundary of the excavation, causes shear strains, apart from the alteration of stress conditions around the excavation.
REFERENCES TO CHAPTER FOUR


CHAPTER FIVE:

RÉSUMÉ AND CONCLUSIONS

The London Clay and the London Basin have been described in terms of the geological processes which created them. This gives a very broad picture and later when describing how earlier engineers tunnelled through it, it has not been necessary to be presumptuous about the extent of their appreciation of the problems. The results they achieved, as far as they are known, speak for themselves.

The picture is not one of clay in the sense of a homogeneous water-bound aggregate of particles of less than 2 µm diameter, but of an environment changing in its physical properties with position and with direction and even time, containing inclusions of, or liable to invasion by, very different materials.

Miners of experience develop a feel for, and more importantly, a respect for, this material which leads the inexperienced to question, say, the necessity of close poling and grouting a twelve foot face which is only going to be left for a weekend. Similarly the engineer may be accused of overdesigning linings and head walls.

It is, therefore, not inappropriate to describe the practice of tunnelling as an art and to indicate its progress by describing its evolution over 150 years or so that it has been extant.

Evolution in engineering has many of the attributes of Darwinian evolution
such as parallel evolution, as evinced by the independent invention of the circular tunnelling shield by Barlow in England and Beach in America, the continuation of early simple forms whilst later more specialised forms died out, or Cope's rule that forms once developed tend to evolve from smaller to larger physical dimensions.

Fortunately it does not preclude cross fertilisation between different species of engineering and again unlike biological evolution, an aberrant trend once started does not have to be followed beyond its useful stages.

The survival of a particular tunnelling method is partly due to whether the use for that kind of tunnel continues. A break in this continuity may cause the problem to be re-thought or a solution which has been thought out, but not applied, may be revived. Thus brick tunnels were used continuously for main line railways between 1837 and 1913, but the break between then and 1953 caused the bolted iron stage to be left out and concrete linings were adopted. During the intervening period the social and economic conditions had changed. We might say that the evolutionary driving force was socio-economic.

To an old-fashioned economist this term would suggest the sort of feed-back process in which investment results in increased prosperity which in turn gives rise to more investment. This process tends to stabilise; it is the one operating during periods of little engineering change. Better words for our purpose would be 'socio-ecological' (Lampard, 1973) or 'anthro-po-ecological' for it is the effect that the engineering works have on the surroundings in which people live determine their real value. The 'tube' tunnels and aqueducts did not provide fortunes for their promoters,
but the social and environmental benefits guaranteed their continued demand, or, to resume our biological analogy, their phyogenetic persistence.

The London to which the railways were built was topographically Georgian London (Summerson, 1973) but with its population swollen by countrymen displaced by the Enclosure Acts and by immigrants driven from Europe by the wars. The latter earned their livings producing luxuries of the day: furniture, lace, watches etc in small workshops in the East End and near the West End shops; the others were employed in more everyday trades: carpentry, saddlery, coach building, brewing and many on the river. The surplus labour drifted to shanty towns on the outskirts of the town, living by casual jobs or thieving. The railways were built across these slums such as Agar Town and Somers Town and terminated on the edges of the estates of the Dukes of Portland, Bedford and Devonshire.

By Victorian times the more successful arsians and the middle class who had the leisure to walk or the wealth to ride had moved out to the surrounding villages. The poor remained trapped in the centre and still the density of population was increasing. The diseases such as cholera, typhus, typhoid and phthisis due to overcrowding, provided the only control. A new pattern of city life had developed: an annulus of prosperity with poverty, squalor and disease inside and outside.

Official reaction to this was included in the 1844 Railway Act which introduced 'Parliamentary trains' for which the fares charged were to be

*Their Graces' wishes were backed up by a Royal Commission (The Commissioners on Railway Termini within or in the immediate vicinity of the Metropolis, 1846) which defined an area from which the railways should be excluded. Using modern names this extended from Euston Road in the north to the Lambeth Road in the south, and from Park Lane in the west to Bishopsgate in the east.
low enough to allow the poor man also to commute. Acts for later railways were to include provision for 'workmen's trains'. The main line railways obeyed the spirit rather than the letter of such legislation and were helped by the remoteness of the termini from the City (Dyos, 1957). However, the Great Eastern in 1874, in order to obtain Parliamentary approval for its Liverpool Street terminus which entailed clearing a large area of working class housing, reluctantly agreed to provide workmen's trains at the low rate of 2d for a return journey of up to ten miles. This resulted in the development of Ilford, Walthamstow, Leytonstone and Tottenham (Perkin, 1970).

By 1855 Parliament* was recommending that the termini should be connected with each other, the City, the Post Office and the docks. The Metropolitan and the Metropolitan District railways started in West London in 1868, but it was not until 1884+ that, largely due to the campaigning of the City Solicitor Sir Charles Pearson, the Inner Circle was completed and the railways began to lessen rather than add to the overcrowding of London.

The City and South London Railway was the first real commuter's railway and, having to derive revenue from peak hour passengers and with no goods traffic, was not the commercial success that many of the surface railways had been. The fare from Stockwell to the City (3 1/2 miles) was 2d, before

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*1855 Select Committee of the House of Commons on Metropolitan Communication. The Joint Committee on Railway Schemes (Metropolis) of 1864 endorsed the proposal. The alternative idea of a large, shared, central terminus was finally killed by the 1863 Select Committee of the House of Lords on Metropolitan Railway Communications.

+Part of this delay was in raising money for the more expensive engineering in the City. This included the Ray St and Widened Lines tunnels.
8 a.m.; the journey could be made from the Oval for 1d (Jenkins, 1892). This was not only much cheaper than the horse omnibuses (outside fare about 1d per mile) (Baedeker, 1895), but about three times as quick—an important consideration for the artisan working very long hours. Hitherto trams (horse-drawn until 1900), not buses, had been the working man's transport. Tramways had a statutory obligation to offer workmen's fares at 3d per mile (Rowbotham, 1872). Sharing the same method of traction and the same overcrowded streets, these too were slow. The electric underground railways, and later the electric trams, were as important as Basalgette's sewers or Koch's formulation of the aetiology of tuberculosis and typhoid, in rehabilitating central London and making it able to support a (daytime) working population of increasing density. Parliamentary insistence on still cheaper fares for workmen's trains on the Central London Railway and later lines in exchange for freedom to tunnel beneath the London streets (Joint Select Committees of 1892 and 1901) shows that there was official recognition of the socio-ecological relation of the 'tubes' to Metropolis.

Later, under the influence of Yerkes, the 'Underground' was actually able to initiate the expansion of London suburbs, while at the same time relieving the congestion that the long distance commuter traffic would cause as it converged on inner London.

Having studied fairly comprehensively the development of tunnelling in London Clay it seems with hindsight that each succeeding step followed fairly naturally from the previous one. We have already seen that successful techniques were continued unchanged. We are reminded that this implies that new ideas tend to remain dormant unless some crisis forces their use and that imperfect ideas, if tried at all, survive because they
are modified during the progress of the work.

The question now arises as to whether it is justifiable to interfere with this natural drift of the technology. 'Yes if it will make things cheaper' is the reply from traditional economic/engineering/accounting quarters. We now see that this means 'Yes, if it anticipates the next crisis'.

How can the scientific method assist the development of a backward technology? Although technology is supposed to be applied science any archeologist knows the two need not be interdependent and that, for instance, blast furnaces and tanneries were working long before chemists postulated the process of reduction or the existence of collagen fibrils.

So if the scientific method is to be applied to the relatively backward tunnelling industry, a starting point is to make some quantifiable assessments of the most commonly used current methods, and, while the unbiased eye is available, observational data can be obtained as well. The next stage is to repeat these with some innovative methods. An opportunity arose to make all four kinds of field investigation.

At the time the work was planned the fastest methods of excavation were not being hampered by the way the clay was cut, but by the logistics of the spoil disposal. This was not our problem.

That tunnels squat had been observed as long ago as 1923 (Jones and Curry, 1926). That the hoop stress increases monotonically to a value corresponding to the overburden acting hydrostatically had been established by workers at the Building Research Station (Ward and Chaplin, 1957) (Ward
and Thomas, 1965) and measurements had been made of the hoop stress and
deflections of rings of concrete tunnels by these and other workers eg
Tattersall, Wakeling and Ward (1955); Wood and Gibb (1971); Smyth-
Osbourne (1971). Bending of traditional iron linings had been largely
neglected, but cracked flanges had been found although the findings do not
appear in the literature.

Measurements of settlement of the ground above tunnels are made regularly
but are not published. These are made in towns as a precaution against
legal claims from injurious effects due to the tunnelling. Because in
towns there are movements due to heavy traffic (including lorries moving
spoil from the tunnelling works) moisture movements due to roadside and
garden trees, thermal and frost movements etc they do not however have much
scientific value.

The field of research therefore was the distortion of iron linings and, if
possible, the movements in the clay which caused them. Unfortunately our
site too was built-up, but it was possible to place oneself, in a sense,
in the ground ahead of an advancing 9 m (29 ft 6 ins) diameter tunnel and
alongside a 3.8 m (12 ft 7 ins) diameter tunnel.

The crisis envisaged was a future shortage of iron and metallurgical coke
and the innovation was saving iron through increasing the strength and
impact resistance of castings, by altering their metallurgical structure.
The resulting 'spheroidal graphite cast iron' had already been used in
commercially available pipes. Its use could make tunnel segments two or
three times lighter than previously. To do this by structural design
would require a confidence about the conditions in the ground which would
be unjustifiable bearing in mind the linear extent of a tunnel and the lateral variation in the properties of London Clay.

The alteration to the iron is achieved by adding magnesium or cerium to the melt; it also requires careful metallurgical control. The present-day price is about twice that of ordinary flake graphite cast-iron so the experimental rings were no cheaper than the old ones, but since the richer iron ores are being rapidly depleted whereas magnesium is available from seawater, the situation will change.

Given the opportunity to install a novel lining it seemed, from the engineering point of view, sensible to embody in the design any implications of the initial change from flake graphite to spheroidal graphite iron. From the scientific viewpoint, if there were more than one change, each one should be given a separate trial, so that the effects of each change could be separated. From the administrators' point of view, it was better since the tunnel formed part of the work of extending the railway, a public asset, at public expense, to have the trial length in one kind of lining and for that lining to differ as little as possible from the one that was familiar to the miners.

The compromise adopted, involved two changes, thinning the section and doubling the number of cross joints; both were included in the same design and this was used for one length of tunnel.

When the resulting pan-shaped segments were tested in the laboratory their flexural stiffness was seen to be due mostly to their flanges because the skin was not constrained to act with them. Besides providing an understanding
of the relation between applied bending and the results from the strain
gauges on the flanges of the experimental segments, this mechanism is of
interest because, to a lesser degree, it operates in conventional segments.
The pan-shaped experimental segments provided a model of the way ordinary
segments behave.

Other tests (Fig 30) showed that the cross flanges of the SG iron segments
similarly exaggerated the behaviour of the flanges forming the cross joint
between a pair of ordinary segments subjected to bending. When, as in
service, the lining ring carries a hoop load, the cross joint flange has
but little function. Those parts of the cross flange which are backed-up
by the circumferential flanges carry the thrust; the skin behaves as it
does when the joint is subjected to positive bending, only the part close
to the circumferential flange acts with it. If circumferential bending is
superimposed on the hoop load the thrust becomes concentrated towards the
tip or root of the circumferential flange and, if bending is severe enough
to form a hinge, only the corners of the cross flange are severely stressed.
(In contrast to the line loading that would occur if the segments were
voussoirs.)

The local flexibility of the thin SG iron segments allowed the rigidity of
a whole segment to change according to the way its back was constrained.
Also a uniform pressure on the back of one would cause its flanges to bend
even though no external couple had been applied. Also, because of the
torsional flexibility of the circumferential flanges, care had to be taken
in recognising circumferential bending. The measurements of bending were
left as curvature so as not to beg the questions of how much of the segment
was involved and what its stiffness was. The strain gauges behaved in a
generally satisfactory manner but in neither the twelve nor the six segment rings could they be used to give values of local hoop stress. The average hoop stress round the whole of either ring was reliably measured, however.

Allowing for these effects due to local flexibility and recalling that the moduli of flexural rigidity of the two kinds of segment were approximately equal, the first part of the field experiment may be regarded, in its broadest aspect, as a comparison of a length of tunnel built with rings of six segments and a key with one built with rings of twelve segments and a key.

This first part showed that both lengths of tunnels were quite safe up to and including the time when the twin tunnel was driven alongside. Twin tunnels are normally driven at 6.4 m (21 ft) centres. In this size of lining, this is an edge to edge spacing of $\frac{4}{7}$ of a diameter (OD). At Brixton the edge to edge spacing was $\frac{1}{6}$ of a diameter.

Cast iron tunnel linings have sometimes failed when other tunnels have been driven close to them. The damage observed has been tensile failure of the flanges above and below the point at which the other tunnel was closest. At this latter point the flanges are in compression and the back of the segment is in tension. It is unlikely that unseen tensile failures occur in the back of the segment because it is close to the neutral axis of the section of the segment (see Table 3). The tips of the flanges are much more vulnerable, especially near bolt holes. However, the author had on an earlier occasion measured the distribution of residual stresses in a tunnel segment and found that there were locked-up compressive stresses in the
flanges of around 30-45 ksi/in$^2$ (2-3 tons/in$^2$) which reduce the danger. A visit to the foundry confirms that segments are still cast in the manner which causes this stress distribution on cooling.

The failures occurred in tunnels which had much smaller tunnels driven close to them. The resulting bending would have been localised. In pairs of equally sized tunnels (as at Brixton) not only is the bending more spread but it is possible that the whole of the first driven tunnel may move towards the second one so that, although the absolute displacements may be quite large, relative to one another they are not large enough to cause undue distortion of the lining.

Distortion of the experimental twelve-segment SG iron ring was about 40 per cent greater than that of the normal six-segment ring of ordinary iron. This distortion was acceptable. It was due to hinging at the butt-joints of the ring, because the ring was under hoop stress this hinging caused the line of thrust to move, that is, it caused bending stresses, which were higher in the twelve-segment ring than in the six-segment ring. If knuckled cross joints had been used this bending would have been reduced, probably to a value which was less than that measured in the six-segment ring.

In neither length containing instrumented ring were the rings built to 'break joint'. The lengths (in ordinary six-segment 12 ft 7 in. iron) up to and after the trials lengths did break joint. The object was, of course, to stiffen the tunnel and it probably did so. It also probably increased the bending in some of the rings but none of the iron failed so it is concluded that in this case it was neither necessary nor harmful.

Nearly all the distortion of the six-week old first tunnel occurred while
the face of the passing tunnel was between one diameter before and one
diameter after the point of observation. The hoop thrust in the first
tunnel was increased by about 10 per cent of the full overburden acting
hydrostatically. The results accord with previous work on twin tunnels
with wider spacings.

Measurements were continued after the second tunnel was completed and
continued until these two 3.8 m (12½ ft) diameter tunnels were surrounded
by a 9 m (29 ft 6 ins) diameter iron lining and dismantled. It will be
recalled that a 2.2 m (7 ft) diameter pilot tunnel was driven centrally
above the pair of 3.8 m (12½ ft) diameter tunnels to facilitate this (Fig 33).

The hoop stresses measured in the two instrumented rings which had been
increasing before and while the second tunnel was driven past, remained
nearly constant for some months. This is attributed to the clay being
more than usually disturbed by the very close twin tunnel. As the pressure
built up on the outside of the very disturbed zone immediately around the
twin tunnel there would be a period during which there was some re-closing
of the fissures before the increasing pressure was transmitted to the
linings.

When the 2.2 m (7 ft) pilot tunnel was driven it caused a temporary reduction
in hoop stress in the instrumented rings which was not observed when the
second 3.8 m tunnel passed. Whether this is due to the more disturbed
state of the ground, or whether it is due to the slow speed at which the
2.2 m (7 ft) tunnel advanced, or to both factors, is not known.

The peak hoop stresses in both observed rings were reached when the main
9 m (29 ft 6 ins) face was at a distance from them equal to half its diameter. In each ring the value of this stress was equal to half that which would be caused by the full overburden acting hydrostatically. Indeed, throughout the period of observations the pressures deduced from the strain gauge readings from the two rings agreed with each other.

The first indication that the 9 m (29 ft 6 ins) face was approaching was the small increase in hoop stress when it was about 27 m (90 ft) away. This distance is equal to the depth of the main tunnel invert. Surface subsidence is usually found to begin on tunnel depth ahead of the face in London Clay. Perhaps if more accurate measurements could be made, the distance would be found to be greater. On the other hand the shear zone on the lip of the subsidence trough may cause dilation of the soil and uplift which perhaps delays the appearance of surface subsidence. Uplift preceding the subsidence has been reported (Morgan and Bartlett, 1969; Babbers, 1971). The relation between shearing, dilation and compression, no doubt arises from the fact that in general the fissure surfaces are not plane. Movement along a fissure between two blocks therefore causes them to move apart. Shearing of a mass of clay results in many such sliding movements along many fissures each of which produces an associated opening of the fissure so that the whole mass expands. If it is not free to expand the average pressure between adjacent blocks of clay will increase, that is a compressive stress will develop in the clay mass. This is the first time that an opportunity has occurred for making a measurement of, or at least related to, such a compression ahead of an advancing face.

The observations of local bending and of diametrical distortion for this latter period do show some difference between the behaviour of the twelve-
segment ring and that of the six-segment ring. The diametrical distortion
due to the 2.2 m (7 ft) tunnel being driven past was (as with the second
3.8 m tunnel) greater in the twelve-segment ring than in the six-segment
ring. In the twelve-segment ring there were violent fluctuations in the
local bending strains whereas those measured in the six-segment ring changed
fairly smoothly. This is due to the greater rotation of the short segments
in the plane of the ring whilst the pressure behind them was fluctuating.
They could rock far enough for the line of thrust to be suddenly transferred
to the edge of the section of the ring. This tendency for rotation
provides a warning that if a ring were made with a larger number of
flexible joints there would be a danger that it could buckle is used unwisely.

However, the conclusions, drawn from the evidence provided by these measure-
ments, are that for normal use in London Clay a lining should be designed
with knuckle joints and segments of sufficient flexibility that it can
conform to the shape taken by the twelve-segment ring. This probably
differs little from the shape that would be taken by a much more flexible
ring. In practice the flexibility of the segments would be determined by
their behaviour when transported to the face and whether they could hold
their shape while the ring is built and before it is grouted.

The number of joints should be a multiple of four if there is to be a
knuckle at each of the four points of maximum change of curvature developed
when a ring squats normally. If the ring is to be built in the tail of
a shield, then the greater the number of segments the more the lower half
of the ring will follow the slightly larger radius of the tail plate (see
Fig 39). The crown of the arch, from one segment above axis level, will
fall away from the tail plate however. An arris will tend to form at
these two positions if the usual clearance (36 mm over a 4 m OD lining) is provided between the outside of the lining and the inside of the tail plate. This tendency will be greater as the number of segments is increased. The clearance is mainly provided against the possibility of the tail plate getting out of round and fouling a six-segment ring that is expected to be built truly circularly. A more articulate ring should therefore be given less clearance to improve the shape of its upper part; any distortion of the tail plate will be transferred to the lower part of the ring in any case.

An example of a design of lining embodying the principles arising out of these studies, and using SG iron in an economical way, is given in the last Chapter. The solution adopted for the shape problem discussed immediately above is to reduce the number of joints.

Knowledge relating to how a tunnel should be driven in London Clay was provided by the measurements of deformation of the ground near the advancing tunnel face. These quantitative data agree with the practice that was shown to be developing in the latter part of Chapter 2 although the reasons for this are not confined to the application of the principles of soil mechanics. The recent advances in the speed of tunnelling have, one suspects, been made for their own sake although economic arguments for it are advanced. One is that an expensive machine should not remain on one job for longer than necessary; if it is not re-used when the drive is finished, however, the argument fails. Another is that the sooner the tunnel is finished the sooner it will pay for itself and for the machine. It seems that the work should be planned to allow enough time so that the use of a slower, but less expensive and more easily assembled, machine could be considered.
The aspect of time that must concern us is the time taken between cutting the hole and inserting the lining. During this time, as shown in Chapter 4, the ground converges towards the hole, both around the shield and ahead of the face. This allows settlement at the surface; but before this happens the fissures near the tunnel open and if there is trapped water nearby or water-bearing gravel overlying the clay, this water will flow towards the tunnel and lubricate the surfaces of the opened fissures. The bulk strength of the clay is then much reduced. If the clay cover is very small and the water pressure gradient large, a failure may occur as a face 'run', or a 'back run' if there is no tail plate.

Collapse of the roof and sides of the tunnel will be prevented by the skin plate of the shield and, working within it, the miners may be able to secure the face. However, if the shield contains a tunnelling machine it is unlikely that they will have enough room to work quickly and effectively enough to do this.

In good ground the shield is not used to prevent all ground movement (and consequent subsidence) because the cutting edge must provide a hole sufficiently large that even after convergence has occurred it is still possible to move the rear end of the shield laterally and vertically to steer it. The convergence is inevitable, its amount depends on the time over which it is allowed to continue.

The time taken for the whole shield to pass through the hole made by the cutting edge in a particular vertical plane in the clay depends on the length of the shield and the speed of advance. Measures to shorten the shield have been described and once they have been taken their advantage
cannot be lost. Speed, of course, is very easily lost and occasionally the shield must be stopped. A prolonged delay will result in convergence equal to the whole cutting clearance provided. The solution at present adopted with fast machines is to use a cutting edge bead to provide a clearance which is small but adequate at the speed normally maintained, and also to provide very powerful shové rams so that after the machine has been stopped it may be advanced (albeit with little directional control) until the bead again cuts an oversized hole. This is satisfactory for water or sewer tunnels which are driven in straight lines with arbitrary changes in direction, but not for railway tunnels which must follow pre-determined curves to very close tolerances. A shield skin of variable circumference seems to be indicated. A recent partial solution which at least allows a fixed size of expanded lining to be placed when the ground has converged is a second cutting edge at the rear of the shield.

In the observations made at Brixton the clay was seen to be just coming onto the tail of the shield (Fig 52) from which one may conclude that the correct size of bead had been chosen for this speed of drive in this clay.

It is also concluded that movements of the ground towards the face and towards the shield and their relation to surface subsidence constitute the area of study in the behaviour of the London Clay that is most likely to provide information and theories which can be applied to advancing the art of tunnelling in this formation. The measurements at Brixton were only made in the horizontal plane but they show:

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*With very powerful rams, especially if they are angled outwards from the axis of the tunnel, it is possible to induce small 'bearing capacity failures' in the clay with the tail of the shield, and so to steer it.*
(i) that much more ground was lost by radial convergence towards the shield skin behind the cutting edge than was lost by movements towards the face;

(ii) that there was considerable bulking of the clay near the excavation.

Measurements in the vertical direction are required to give a complete picture. They are likely to differ from the horizontal ones because:

(i) the initial stresses in the ground in the horizontal and vertical directions differ in this heavily overconsolidated clay;

(ii) self-weight stresses are added to the vertical stresses;

(iii) the stress-strain properties (and many other properties) of the clay in the hand specimen differ in the horizontal and vertical directions;

(iv) the bulk properties of the clay similarly differ in the two directions. In the horizontal direction they are controlled by the fissures and by the jointing. (The joint pattern at Brixton was insufficiently developed to be discernible.) In the vertical direction the bulk properties will be dominated by the bedding planes and silt lenses (at Brixton there was also a claystone layer) which can be expected to provide 'no tension' planes.

Accurate measurements of surface subsidence made on land which is not liable to moisture movements due to vegetation or drainage, and which is free of traffic and vandals, are also required. An airfield would be an ideal site. Subsidence profiles drawn from these measurements would ideally be related to the measurements near the face. Nevertheless, they would be of considerable value if the only other information obtainable was the method of tunnelling used and a description of the ground. The
curvature and gradients of the profile could be compared with the growing collection of settlement observations on buildings, both those which had shown distress and those that had not.

The observations of soil movement as well as helping us decide if present methods of tunnelling are acceptable will also be useful in studying the soil-lining interaction. Before this interesting subject is pursued further, however, the present work should be completed by a trial of the extra flexible lining advocated above and detailed in the next Chapter. This trial should include a ring which is instrumented. The success of the instrumentation at Brixton indicates that similar methods would be satisfactory. In addition to measuring circumferential bending of the parts corresponding to the flanges it would be useful to measure bending in the direction at right angles. This would be done by using gauge posts carrying two vibrating wires. Another desirable measurement would be the angular deflection between adjacent segments, measured at the cross joints.

Should these angles not be small, the author would regard his segments as being too stiff. If he is told that most designers make their segments far more stiff, and should it be suggested that the proposed segments are undesirably flexible, he would ask to measure the distribution of earth pressure on the back of a stiff segment with a view to demonstrate that it attracts increased pressure to its ends, and hence is bound to develop bending moments if it is at all long.
REFERENCES TO CHAPTER FIVE


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JONES, I J and CURRY, G (1926): see p 135, references to Chapter 2.


ROWBOTHAM, T K (1872): Evidence to the Joint Select Committee on Tramways (Metropolis) 1872. Paper 252, H C Sessional papers 1872, vol XII, qq 299, 303.


Measurements of the structural behaviour of the linings used at Brixton (Chapter 3), observation of the difficulties encountered by the miners there, and a study of tunnel linings (Chapter 2) indicate that a new sort of tunnel lining is needed.

Although in recent years a number of variations have appeared, bolting together flanged iron segments to form rings which are grouted in to the ground, and expanding rings of concrete lining directly against the hole in the clay, remain the two basic methods used for lining tunnels in London Clay. The merits and de-merits of the two methods may be summarised as follows:

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<th>BOLTED IRON</th>
<th>EXPANDED CONCRETE</th>
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<tbody>
<tr>
<td>Cost</td>
<td>Expensive (about half cost of tunnel)</td>
<td>Cheaper (about 1/6 cost of tunnel)</td>
</tr>
<tr>
<td>Weight</td>
<td>Heavy (3.5 tonnes per m. of 4 m tunnel)</td>
<td>Also heavy</td>
</tr>
<tr>
<td>Damage in handling</td>
<td>Flanges crack</td>
<td>Corners and edges spall</td>
</tr>
<tr>
<td>Effect of overbreak</td>
<td>Merely use more grout</td>
<td>Makes building very difficult</td>
</tr>
<tr>
<td>Caulking</td>
<td>Fairly easy (easy, but more expensive if specially machined segments used)</td>
<td>Difficult</td>
</tr>
<tr>
<td>Speed</td>
<td>Slow</td>
<td>Rapid</td>
</tr>
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</table>

However, the most serious drawback of the expansion method is the difficulty of changing to the bolted lining if unsuitable ground is encountered.
Knowledge of running ground such as gravel will probably be available from borehole information, but a propensity to overbreak or local softening may well be met without warning. As we have seen, with the London Clay delay is dangerous. It may well be possible to carry on without a tail plate on the shield, but it is unlikely that iron segments will be available at the face. If the drive is stopped the chances are that troubles will increase and, if no lining can be placed, the ground behind the shield may fail. It is more difficult to secure the ground here than it is to make the face secure.

Hence a lining has been designed which, while it may not be suitable for the very worst type of ground, can in London Clay replace either bolted iron or expanded concrete and obviates the disadvantages of both.

6.1 DESCRIPTION OF THE NEW LINING

A segment of the new lining consists in part of a spheroidal-graphite cast iron frame. The frame has curved sides about 50 mm (2 ins) deep which taper towards the extrados. These curved sides, which correspond to the circumferential flanges of an ordinary iron segment, are joined by two ends and two ribs. These ends and ribs do not themselves taper but, because of their radial disposition, are angled towards each other.

The three compartments of the frame contain concrete slabs. These are each curved to form part of a cylinder about 600 mm (24 ins) long and subtend an angle of about 15° at the centre. The 50 mm x 600 mm (2 ins x 24 ins) faces are radial plane surfaces and hence slope towards each other at 15° too. These surfaces, and sections of the slab which are also radial to the tunnel axis, are trapezoidal, the short sides being inclined towards each other at 15° (see Fig 53).
Fig. 53 Exploded view of proposed tunnel lining, showing expanding key segment and also small solid one for use when lining is grouted
Hence a uniform pressure acting on the extrados of a slab produces compressive forces in the circumferential direction and also at right angles in the longitudinal direction. Both sets of forces are balanced by tensile forces in the frame.

Stiffness in bending is provided mainly by the frame, but this is not considered an important function of the frame, except insofar as it enables the tunnel to be built with longer segments. Longer segments can be built to a better shape (pp 107-8, 156-7 and 236-7) and can be built faster (see lower half of p 100) than short ones. Should large curvatures develop in the steel frame it will deform plastically without detriment.

The straight ends of the frame are made in the form of knuckle joints, hence a small change in angle formed at a cross joint by two segments does not affect the stress distribution near the joint.

The ring may be built in three ways:

1. As a true circle in a hole 4.114 m diameter and expanded against this hole by means of the folding wedges which constitute key 2 (Fig 53).

2. As a polygon of somewhat smaller size using key 1. In this case the ring is built inside the tail plate of a shield and later grouted into the ground. In this method of building, each segment, (starting with an invert segment) is attached in turn to the corresponding segment of the previously grouted ring. Attachment is at two points on the circumferential joint by slotted pins and wedges. One set only is shown in Fig 53 in which the left hand
side is in the direction of drive. The pin fits loosely into recesses in the leading and trailing faces of successive segments. Wedges pass through slots in the segments and also through slots in the pins. When a wedge is driven the pin is moved with respect to the segment. Driving the wedge which is near the leading edge of the previously built segment tightens the circumferential joint. Driving the other wedge, which is arranged at right angles, adjusts the newly built segment in roll.

The next segment in the ring is erected in a similar way. The foremost corners of the two segments being connected by a dog (one of which is shown in Fig 53). With this, and the circumferentially acting wedges, the longitudinal or cross joint can be closed.

Finally, the narrow key 1 is inserted between the two top segments which are clamped to either side of it by a special dog which reaches across the key, via a slot in its end. Thus the key is retained positively as well as by the friction at its tightened cross joints.

3. As in method 2 but using frames without concrete cast into them. Instead sheets of steel can be temporarily attached to the intrados. In this form they can be used in a hard rock tunnel as ribs with or without lagging. The steel sheets can then be attached as shuttering and the concrete placed in-situ.

6.2 ADVANTAGE IN USE

The following advantages are claimed for the lining:
The new lining saves material. The weights of this lining, traditional cast iron, expanded concrete and the Groves reinforced concrete lining are respectively 1500, 3200, 7400 and 10,000 kg per metre run of 4 m tunnel (1015, 2150, 5000 and 6750 lbs/ft run). Nine per cent of its volume is iron; in Groves' reinforced concrete about 2 per cent of the volume was steel. Bearing in mind the greater mass of the Groves lining, the SG iron frame is a more economical use of ferrous metal for taking the tensile stresses in a composite component.

Identical segments are used all round the ring (except for the small key segment) so that loss or damage of a segment in transit to the face does not upset the cycle of work there.

The same segments are used for lining by the expansion method and by the grouted method. Hence should an expanded type of tunnel encounter bad ground, the method of building can be rapidly changed.

Because the concrete is compressed in three directions, a thin section may be used without the overburden pressure causing failure.

Circumferential bending of the segments does not give rise to tensions in the concrete since this is prestressed in compression. The thin section causes the bending stress to be lower for a given curvature strain.

Bending stresses in the concrete in the longitudinal direction are low because the centre of the segment does not bend as a diaphragm
but is free to move inwards as a whole.

7. Because the section of the segment is fairly symmetrical about the neutral axis for circumferential bending, and the flanges are not deep, bending strains in the SG iron frame are minimised. Also differences in strain in the concrete and iron are minimised, so that large contact stresses do not develop at the longitudinal edges of the concrete slabs.

8. The SG cast-iron frame is not vulnerable to impact stresses and protects the concrete when the segment is roughly handled. Spalling which occurs if ordinary concrete segments are mishandled, is avoided.

9. The lining is lighter than either the conventional bolted iron lining or the newer expanded concrete linings.

10. The thin lining gives a larger inside diameter of tunnel for a given volume of ground excavated.

11. Joints between the SG cast-iron frames may be caulked by the traditional method used for iron segments.

12. Tunnel furnishings (cable brackets, lighting etc) may readily be attached to tapped holes in the cast iron cross ribs.

13. Speed of erection: lightness and ability to withstand accidental impact allow the ring to be built rapidly by hand. If the ring is
to be grouted, attachment by means of the pins and wedges provided is quicker than by using nuts, bolts and washers.

14. A smooth interior surface is produced even if the segments have to be attached to one another for grouting; the wedges may later be removed or cut off.

15. For a short length of smooth bore, in-situ concrete tunnel in hard rock, the expense of special shuttering may be avoided by using the unfilled frames as ribs and to support simple recoverable shuttering.

6.3 MANUFACTURE

A full set of working drawings has been prepared from which a 4 m ID lining could be manufactured. Such manufacture would be the subject of a licence agreement based on a patent which is pending (Thomas, 1971).

The SГ iron frame is a rather more complicated casting than the traditional iron tunnel segment. Nevertheless, SГ iron lends itself to the casting of thin complicated shapes. The frame has a thick coating of the bituminous compound normally used to protect cast iron segments. This not only provides protection against corrosion but acts as a water seal between the concrete infilling and the iron. In the long-term it acts as a lubricant between them, facilitating the radial inward movement of the concrete parts.

For infilling with concrete the frames are laid on tables having a cylindrical surface matching the internal radius of the segments. This should be a compliant surface so that the self-weight of the frame will provide a seal between the frame and the table. A stiff mix is poured into
the frame and the top or outside surface finished with a trowel. Unless the water content of the mix can be kept very low it would be advisable to cast only the least tilted third of a frame at one time. The mix may be dewatered to some extent after pouring by applying a suction via a porous surface in the table top.

At the concrete casting stage, this way of making segments has a number of advantages over conventional pre-cast concrete manufacture:

1. Expensive moulds are not required because the frame acts as part of the mould.
2. No oiling or other pre-treatment of this is required since bitumen dipping is a normal foundry process.
3. Demoulding time is eliminated.
4. The SG cast-iron frame is light and resistant to impact so that if necessary it may be easily transported and the concrete infilling made elsewhere, perhaps at the tunnel site.

6.4 COST

6.4.1 Labour

It follows from 13 of 6.2 that the use of expensive labour at the face is minimised. From 6.3 the cost of demoulding is saved. These savings are off-set by the rather elaborate coring called for in manufacturing the SG iron castings. Nevertheless, factory labour is cheaper than underground labour and the factory process is amenable to automation.

6.4.2 Materials

From 1 of 6.2 it is evident that less concrete is used. Concrete has a high energy content so that its price will rise. At present SG iron castings
are more expensive than those of ordinary flake graphite iron; but as iron becomes more scarce this difference will diminish. It is perhaps more appropriate to compare the cost of iron in this lining with the cost of steel, since its function is one for which steel is often chosen.
REFERENCE TO CHAPTER SIX

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