SITE INVESTIGATION AND CONSTRUCTION OF THE CARDIFF CABLE TUNNEL

by

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THE CARDIFF CABLE TUNNEL

ABSTRACT

The site investigation and construction records for the 1.5 km long Cardiff cable tunnel have been examined to see whether they can give lessons of good practice for tunnel site investigation. The tunnels were constructed wholly within the Keuper Marl; excavation was mainly by full-face tunnelling machine and lining was with pre-cast concrete bolted segments.

The Report describes the site investigation and construction of the works in some detail with special emphasis on ground conditions. On this site preliminary information proved most useful. The site investigation gave a generally satisfactory account of the site geology. However, points to emerge from the study were that if mechanised tunnel excavation is a possibility and the ground is predominately soft, then bands of harder material at tunnel depth take on extra significance and must be carefully recorded and tested. Rock strength test results in the site investigation report should be carefully considered when making a choice of method. If the operating conditions call for a watertight tunnel close attention should be given to permeability results in the site investigation report which show the possibility of high ground water flows.

1. INTRODUCTION

The Laboratory is making a series of case history studies of tunnel projects1-6 with special emphasis on the role played by the site investigation; the tunnels chosen for study were of varying complexity and were constructed in different ground conditions. The aims of the studies are to bring out the essentials of good site investigation practice, show where improvements in technique are required, and put on record the experience gained in constructing tunnels in the particular conditions of each project.

1.1 The Cardiff cable tunnel

This Report describes the site investigation and construction of the Cardiff cable tunnel. It has been prepared with the generous co-operation of the promoter, the Post Office, and their consultants, Sir William Halcrow and Partners.

The scheme involved the construction of 1.5 km of tunnel of 2.4m internal diameter, and included two spurs 100m and 300m in length (Figure 1). The system was provided to bring cables from three parts of the city to shafts already constructed under the new telephone exchange near Cardiff Central Station. The works also included the sinking of three additional shafts up to 3.7m diameter and the construction of 15 enlargements or chambers varying from 3 to 5.8m in diameter. The main length of tunnel ran on a curved line with a minimum radius of about 150m.
The principal participants in the Cardiff cable tunnel scheme are given in the Appendix. Tenders were called for in February 1975, the contract was let in August 1976 and work commenced in September 1976; construction time was scheduled to be 85 weeks. Construction was virtually complete by January 1979 although work on making the tunnel watertight and finishing continued throughout the rest of the year. A short account of the scheme has been given by Gavin and Bocock.

The tunnel is to carry multiple telephone cables, supported in racks on either side of a central walkway, and because of this the Consultants sought to achieve a minimal leakage of water into the tunnel, the specification being aimed at perfect watertightness.

### 2. SITE INVESTIGATION

#### 2.1 Preliminary information

The Promoter provided the Consultants with site investigation reports for seven sites in the vicinity of the line of the tunnel. The borehole logs showed similar types of material at all the sites. A typical profile consisted of made ground about 2m thick overlying a 1.5m thick layer of firm to soft silty clay overlying a 3 to 5.5m thickness of compact gravel containing variable proportions of sand and cobbles. Keuper Marl was found below the gravel and was usually weathered to a soft clay at the top of the deposit becoming progressively harder with depth. The level at which the top of the Keuper Marl was encountered at the different sites generally varied between OD +2m to OD −3.5m. Records of the standing water level in the borehole logs indicated that the water table usually remained within the upper reaches of the gravel layer.

Standard field and laboratory test results were also reported, but with the exception of one site, only a small range of tests was carried out and at all the sites only a small number of samples was taken for each type of test. Grading tests and observations during borings suggested that the flow of water in the gravel layer could be fairly high but that pumping from open sumps should be adequate to keep excavations dry. The gradings also showed that cement grouting or other chemical injection methods could be employed to assist in dewatering by reducing the permeability of the ground. Analyses of the soluble sulphate content of samples of ground water gave results which indicated that the use of sulphate resisting cement for concrete structures was not considered essential. At two of the sites where standard penetration tests were made the strength of the Keuper Marl below the weathered zone was assessed as being between medium to hard. The compressive strength of cores taken from the Keuper Marl at one site varied between 2 and 33 MN/m². The condition of the Keuper Marl was not described in detail but at one site the presence of decomposed bands was noted and the borings also revealed the existence of a 2m thick cavity filled with loose clayey sand. The comment was also made that such cavities were not unknown in the region.

The sites of six of the previous investigations were about 50 to 100m off the proposed tunnel line with one site practically on the line. The borehole logs confirmed the general geology of the area as being made ground, alluvium and river gravels overlying Keuper Marl. It can be seen that this preliminary information provided a valuable guide to the conditions on the tunnel line. It also assisted preparation of the Specification for the main site investigation contract.

#### 2.1.1 Feasibility report

Following their study of the preliminary information together with an inspection of cores and samples, the Consultants made a feasibility report to the Promoter in January 1973. The report concluded that on the information available it seemed likely that the tunnel and chambers...
could be constructed entirely in the Keuper Marl and in free air. Methods of excavating and lining the tunnels and shafts were also discussed and it was considered likely that a pre-cast bolted concrete lining would be suitable for the tunnels and chambers and for the lower sections of the shafts in Keuper Marl, but that cast-iron lining was recommended for the upper sections of shafts in made ground or gravel to avoid the difficulties of achieving watertightness with concrete lining. It was also recommended that a main site investigation be carried out before the tunnels were driven and it was envisaged that this would comprise the sinking of boreholes at shaft sites and additionally at 100m intervals along the tunnel line.

On the basis of the preliminary information and the feasibility report, the Consultants prepared a Specification and Bills of Quantities for the main site investigation in July 1973.

### 2.2 Main site investigation

The Specification stated that the purpose of the main site investigation was to obtain the following information:

1. The geology of the area, including the position, throw and hade of significant faults, and the dip of the bedding planes in the various strata.

2. The physical characteristics of the ground, with particular reference to the design and construction of manholes and culverts near ground level, sinking of shafts, and tunnel excavation.

3. The position of the water table.

The exact depth to which borings were to be made was to be dependent upon the nature of the strata revealed. However it was anticipated that the depth would not exceed 25m and the average depth of the boreholes was expected to be about 20m.

In soft strata disturbed samples were to be taken at 1.5m intervals except where changes in strata necessitated more frequent sampling. The samples obtained were to be made available for inspection by the Consultants. For hard strata a complete core at each boring was to be obtained and retained for inspection by the Consultants. Both disturbed samples and cores were later offered for inspection to tenderers.

As directed by the Consultants, the following tests were to be carried out on selected samples:

- Plastic and liquid limits
- Specific gravity
- Bulk density
- Particle size distribution
- Strength in triaxial compression
- Sulphate content
- Moisture content.

The site investigation contractor was also required to take other samples and carry out additional tests as necessary to enable recommendations to be made regarding soil stabilisation in water-bearing ground when sinking shafts.
Particular attention was to be paid to the presence of water revealed by the borings. In the event of water entering a borehole the contractor was required to inform the engineer immediately. The level at which the water entered and the quantity encountered were to be accurately defined in the log. At the completion of the boring a record of the level of the standing water was also required. Provision was also made for installation of standpipes.

2.3 Field work

The field work, which commenced in August 1973, involved the sinking of ten boreholes using a combination of percussive boring with a shell-and-auger rig and rotary drilling. Normally rotary drilling was used to obtain cores of the bedrock from the bottom of the lined shell-and-auger boreholes. Eight of the boreholes penetrated to depths of between 16 to 24m, one borehole was terminated at a depth of 7.4m and one borehole was abandoned at a depth of 3.65m after an unsuccessful attempt to penetrate granular material using rotary open-hole methods. The sides of the augered boreholes were supported by 200 mm diameter casing and rotary drilling was carried out using 75 mm or 55 mm diameter core barrels with air and water flushing respectively.

Prior to the commencement of boring a starter pit one metre square and about one metre deep was excavated at each location to permit the boring equipment to be positioned without interfering with buried services. The locations of the boreholes are shown in Figure 1.

Standpipe piezometers were installed in six boreholes to enable the Consultants to make long-term observations of ground water levels. The piezometer tips, each positioned at the centre of a sand cell 2 to 3m in length, were all located within the Keuper Marl usually at the bottom of the boreholes, which varied in depth between 17 and 24m. In one of the boreholes the piezometer was installed at a higher position, at a depth of 12m.

In situ falling-head permeability tests were made in several of the boreholes during the course of boring through the Keuper Marl formation.

Standard penetration tests were made in the coarse granular stratum at six of the borehole locations. Penetration resistances were measured at about three different levels in each borehole to provide an assessment of the relative density of the granular material.

2.4 Ground conditions

In general the strata revealed at the site comprised a 5 to 8m thickness of fill and drift deposits overlying Keuper Marl. The level of the top of the Keuper Marl varied between -0.35m OD (Borehole 8) and +2.70m OD (Borehole 7). At each end of the main route the Keuper Marl was present at about +2m OD but was generally lower, approaching Ordnance datum, along the central section between Boreholes 3 and 5.

The drift deposits consisted of layers of variable soft to firm silty or sandy clays overlying a coarse stratum of gravel and cobbles containing a little sand with slight clay binding. Along the main route the coarse stratum was found to vary in thickness from 2 to 4.25m, the variation occurring in a random fashion. Made ground was present up to a maximum thickness of 2.75m at the north and south ends of the route but no significant deposits of made ground were met along the central section between Boreholes 1 and 5.
The Keuper Marl consisted predominantly of red-brown mudstone with occasional blue reduction spots and frequent iron-stained fracture surfaces. Some finely disseminated gypsum was also noted within the mudstone and thin gypsum veins were also recorded in one of the boreholes. The upper levels typically comprised a red-brown silty or very silty clay, or clayey silt, in some cases containing mudstone fragments or mudstone bands.

Bands of sandstone and siltstone, of variable coloration, typically grey-green, were present within the mudstone and, in comparison with the other areas, a greater proportion of siltstone was observed in Borehole 8. Significant thicknesses of silty clay were found interbedded with the sandstones and siltstones at various levels. Evidence of thin bands of silty clay was also seen in the cores obtained from most of the boreholes.

Although a generally high core recovery, normally close to 100 per cent, was obtained in the Keuper Marl, much of the core was fragmented and it was difficult to ascertain to what extent this may be attributed to the drilling operations in such a friable material. In many cases, however, the surface of relatively small fragments were noted to be heavily iron-stained, indicating natural fracture surfaces.

Six unconfined compression tests made on core samples of Keuper Marl gave results which, for five of them, ranged between 10 and 40 MN/m², but which included one value of 195 MN/m² for a sample from Borehole 1A. With the exception of this last result the Keuper Marl at this site was therefore classified as being 'moderately weak' to 'moderately strong' rock using the Geological Society Engineering Group Working Party's classification system. The last result, however, falls into the 'very strong' rock category. Much later, in the course of investigation of claims concerning the strength of rock met in the excavation, the site investigation contractor suggested that this high value may have been an error and that the probable true result should have been 44 MN/m².

A longitudinal section along the proposed tunnel line showing graphical borehole columns is given in Figure 2. Superimposed on the drawing is a longitudinal section of the tunnel as constructed although this is on a slightly different line as shown in Figure 1. It should be noted that the level of the tunnel was not decided upon before the borehole survey was carried out.

On the borehole logs themselves, the driller had sometimes recorded the condition of the Keuper Marl as being soft, stiff or hard and the Consultants made some use of these observations in their assessment of ground conditions at proposed tunnel level.

2.5 Ground water conditions

Water strikes occurred in each of the boreholes at depths ranging between 2m in Borehole 1A and 7m in Borehole 7. The rate of water inflow was generally medium to fast. In Borehole 5 seepage was observed at 4m below ground level and a second strike, with medium inflow, occurred at a depth of 7m. Overnight standing water levels were also observed in the boreholes. In some cases, successive readings showed considerable fluctuation, and equilibrium conditions may not have been attained. Moreover, water levels were likely to have been affected by the proximity of the tidal River Taff.

Long-term water level observations were made by the Consultants using the standpipes and piezometers installed for this purpose. The readings taken in Boreholes 2 and 4, at the northern end of the site, were very similar to the maximum overnight standing water levels recorded during the field work.
However, readings in Boreholes 5, 6 and 7 indicated an equilibrium level several metres lower than that suggested by the observations made during boring. However, it is clear that the water table was within the drift deposits, in most cases close to the upper surface of the coarse granular stratum.

The standing water levels and the levels at which water was first struck are shown on Figure 2.

_in situ_ falling-head permeability tests were made in several of the boreholes during the course of boring; all of the tests were carried out in the Keuper Marl. In some cases the drop in head was unusually rapid. The results showed that the coefficient of permeability ranged from $5 \times 10^{-8}$ to $8 \times 10^{-5}$ m/s. The highest permeability was obtained from tests made in Borehole 1A and examination of the cores showed that the mudstone was interbedded with bands of siltstone near test level. Moreover, the Keuper Marl in Borehole 8 was also noted to comprise largely siltstone with some mudstone and a permeability of $2 \times 10^{-5}$ m/s was obtained from tests performed in this hole. The lowest permeability was observed for the mudstone in Boreholes 5 and 7.

If the Keuper Marl is considered as a jointed rock, the above range of values indicates that it would be 'moderately' to 'slightly' permeable on the Geological Society Engineering Working Party's classification. However, if the Keuper Marl is considered as a soil, this range of values indicates that it would have 'good' to 'poor' drainage on the British Standard classification. The upper limit of the observed permeability range thus indicates a relatively high permeability, although it must be noted that only seven tests were made, two in Boreholes 1A and 8 and one in Boreholes 4, 5 and 7.

2.6 Engineering appreciation in site investigation report

The site investigation report, submitted to the Consultants by the site investigation contractor in March 1974, contained a discussion of ground conditions in relation to the proposed works as well as the usual factual information.

Advice was first given on the problems that would be encountered in driving a high level tunnel through the drift deposits, but because, in the event the tunnel was driven at a lower level through the Keuper Marl, this discussion will be omitted here.

Advice was then given on tunnel driving through the Keuper Marl. With regard to ground water, it was pointed out that relatively high values for the coefficient of permeability had been observed and that because it had not been possible to seal the ground water from the borehole in the Keuper Marl it was to be expected that a considerable inflow of water would occur into a heading through this formation.

It was considered that full scale de-watering using conventional methods was almost certainly impracticable. Alternative solutions suggested included localised grouting of an annulus surrounding the proposed tunnel followed by de-watering within this zone, or the use of compressed air within the heading to exclude ground water; the latter suggestion having the added advantage of also reducing any tendency for the ground to squeeze. It was therefore recommended that strong consideration be given to the use of air pressure to exclude water, to help maintain a stable face, to enable back-packing to be carried out satisfactorily and to minimise surface settlement.

It was advised that a further series of shallow boreholes be sunk, particularly along the central part of the route between Boreholes 4 and 5 where the top of the Keuper Marl is low. Such boreholes would
need only penetrate the top of the Keuper Marl in order to prove the minimum cover. The reason given for this was that it was important to establish that there is adequate cover to the roof of the tunnel, allowing for the anticipated overbreak, if compressed air was to be used. If a drop in the level of the top of the Keuper Marl occurs in any area sufficient to cause the coarse granular material to be met in the heading, a rapid loss of air pressure could result, leading to possible inflow of water.

Turning to construction of the tunnel, it was considered that excavation of the Keuper Marl by hand-held pneumatic tools would not present any particular difficulties. If machine excavation was considered, it was pointed out that the strata conditions may change rapidly both vertically and horizontally and that the materials vary from strong siltstone and mudstone to silt and fine to soft silty clay.

Although the mudstone was noted to be friable and closely fractured, often with iron-stained surfaces, it was considered probable that this material would, in the absence of seepage forces, stand unsupported for a satisfactory period without significant loss of ground. Ravelling would be small and, for this relatively small diameter tunnel, squeezing of the mudstone before installation of the permanent lining would be negligible. However, a shield was considered necessary, even if the ground water was controlled, for protection of the personnel and mining equipment. A cylindrical tailpiece in which to erect the permanent lining segments was also recommended.

A stability calculation was made for the tunnel face assuming that silty clays are present over the full face area and that surface settlements due to squeezing are likely to be small provided that the ratio of overburden pressure to shear strength is less than 4. For a depth of 10m, the total overburden pressure was approximately 200 kN/m$^2$ and hence the required shear strength to satisfy this criterion would need to be greater than 50 kN/m$^2$. It was considered probable that this would be the case for relatively thick layers of the silty clay.

Some advice on shaft construction was given. It was noted that the made ground and the underlying drift deposits must be continuously supported with close sheeting in order to maintain their stability. Some difficulty could be experienced in driving sheeting through the coarse granular stratum due to the presence of cobbles and boulders. This could lead to significant deflections of individual sheets and lack of continuity of support.

A warning was made that a seal of ground water could not be obtained by driving sheeting into the Keuper Marl. As discussed above in relation to the tunnel, a combination of cylindrical curtain grouting and de-watering within the zone of the shaft could be employed. However, the method of shaft sinking would need to be closely linked with the chosen method of tunnel excavation.

Finally it was recommended that in the zone of influence of the tunnel and shafts, tolerable surface displacements in the area of existing structures should be decided in relation to the depth and type of their foundations, and the possible effects of the proposed works on existing sub-surface installations should be given careful consideration.

Some of the advice given in the engineering appreciation section of the site investigation report did not conform with the experience of the Consultants in tunnelling and shaft sinking operations.
3. CONSTRUCTION

The route of the cable tunnels lay mostly beneath the centre of Cardiff and ran close to several large buildings. To avoid the possibility of causing structural damage to these buildings it was important to ensure that settlements arising from tunnel construction were minimal. The Consultants decided to locate the tunnel within the Keuper Marl which was expected to be largely self-supporting and therefore a more stable tunnelling medium than the overlying water-bearing gravel. Also it was thought that exclusion of water would be much simpler in the Keuper Marl than in the gravel. The method of driving the tunnels was specified at the tendering stage to be by hand shield but the contractor was also given the option of offering alternative approved methods. Provision for the supply of compressed air was also made but the Consultants did not anticipate that this would be necessary for the main drive, but would be advisable for the spur tunnel under the River Taff. Construction commenced in September 1976 with the sinking of the access shaft at the main work site. Throughout the construction period the Contractor's Agent made a photographic record of the works, from which Plates 1 to 4 are taken.

3.1 Shaft construction

The three central shafts connecting with the main enlargement (see Figure 1) were sunk in advance of the main contract just prior to the construction of a new telephone exchange building when suitable access to the site was possible. The main contract included the sinking of three additional shafts at the locations shown in Figure 1. All the shafts were between 15 to 16m in depth and were constructed ring by ring using hand mining and under-pinning methods. The shaft at the main work site, at the eastern extremity of the tunnel, which provided access for the start of the tunnel drive, was 3.66m in diameter. At the north and west sites the shafts were 3.05m and 2.44m in diameter respectively. The Promoter required the Contractor to use an existing stock of cast-iron lining in the works and so all three shafts were lined with bolted cast-iron lining (Plate 3) down into the Keuper Marl and bolted pre-cast concrete lining was used thereafter. Two temporary guard rings of concrete were placed at the top of each shaft. Allowance was made in the contract for the control of water during shaft sinking by chemical treatment or other means and the pumping of large quantities of water was not permitted. There were no problems during sinking the north and west shafts and it was found that the excavations could be maintained in a satisfactory dry condition by pumping from a sump.

At the main work site construction of the shaft proceeded without difficulty until the excavation had penetrated some depth into soft clayey Keuper Marl. A serious problem then occurred, caused by the sudden influx of a considerable volume of water which burst through the floor of the excavation and rose in the shaft to an equilibrium level of 1.7m OD. This could have been due to the presence of numerous water-filled interconnecting fissures in the hard Keuper Marl below the excavation floor tapping a large source of water contained in the surrounding rock. It was not possible to control the water by pumping and it was therefore decided to engage a specialist contractor to carry out ground treatment. At the same time it was also decided to alleviate the problem by reducing the planned depth of the shaft; the tunnel level at the shaft was raised by 3.5 metres and the direction of the gradient of the tunnel was thus reversed. The ground treatment consisted of injecting cement grout by *tube à manchette* from vertical boreholes drilled from the ground surface. An array of holes was used to place a horizontal grout curtain over 3 metres thick at about tunnel level. This curtain was successful in sealing off the inflow of water in the immediate area and excavation of a 17m long chamber adjoining the bottom of the shaft was able to proceed without further difficulty. From this chamber horizontal and inclined boreholes were drilled in the direction of the tunnel line and the cement grout treatment was used to stabilise and seal the ground ahead of the first section of tunnel.
3.2 Tunnel construction

The Contractor opted to use a full-face tunnelling machine to excavate the main drive from the work site to the north shaft. At a point about 100m from the north shaft the machine was turned in an enlarged chamber 4.6m in diameter and the drive was continued to link up with the north shaft which had already been constructed. A hand shield was used for the short and partly subaqueous west drive from the main enlargement. It was originally intended that the main drive would proceed uphill at a gradient of about 1 in 250 with drainage to a sump in the bottom of the shaft at the main work site. This was later amended when, because of the water problem encountered when sinking the shaft, the gradient of the tunnel drive was changed to run downhill at a gradient of about 1 in 400. A sump for receiving drainage from the whole tunnel system was subsequently located in a shaft adjacent to the central enlargement. The entire tunnel was constructed with a pre-cast bolted concrete segmental lining having an internal diameter of 2.4m; this also served as a pilot tunnel from which enlargements were made to form chambers varying between 3.7 to 5.8m in diameter at the locations indicated in Figure 1. Installation of air locks and compressor capacity for compressed air working were provided for under the contract. As a result of the unexpected flooding of the shaft at the main work site during construction, and the resulting experience of sealing the ground by grouting, a decision was made by the Engineer to employ compressed air working from the onset of machine tunnelling. An air lock was therefore constructed at the commencement of the drive in the stabilised section of ground.

3.2.1 Excavation. The McAlpine full-face tunnelling machine employed for most of the excavation was 2.8m in diameter and consisted of a cutting head mounted in a shield (Plate 1) which was coupled in train with five sledge-mounted units for transporting the main transformer and switch house, grouting equipment and belt conveyors. Power for driving the cutting head was provided by two motors capable of developing 187 kW (250 hp). The maximum head speed was 3.5 rpm and the direction of rotation was reversible in order to be able to correct for any roll of the machine.

The machine was equipped with twelve 150m diameter thrust rams having a maximum stroke of about 850 mm; each ram was capable of exerting a force of 400 kN. Spoil removed from the face excavation was transported by a series of three belt conveyors and deposited into skips. The conveyors were 450 mm wide, were run at a speed of 70 m/minute and were capable of transporting 260 tonnes of material per hour (approximately 2.7m$^3$ of loose spoil per minute). A train of four skips could be accommodated in the space behind the rear conveyor. Each skip had a capacity of about 1m$^3$ and it was estimated that two train loads would be required to remove the material excavated for each one ring advance of the tunnel.

At first the cutting head was fitted with flat toothed rippers which the contractor had found to be satisfactory for excavating stiff London Clay at a site where the machine had previously been working. After a short period during which several teeth were broken the Contractor decided that the Keuper Marl was much stronger than had been anticipated and the drive was halted whilst new mountings were designed and manufactured so that an array of Big-A tools could be fitted on the cutting head (Plate 2). These tools were fitted with tungsten carbide tips and performed well in cutting through the ground containing bands of hard siltstone and occasional massive rock. A smooth periphery without overbreak was also obtained. For the main drive the progress of the machine gave an overall mean rate of advance of 5.4 rings per ten hour shift with a maximum of just under 9 rings per shift being achieved (Figure 3), each ring representing an advance of 0.6m. The need to change the tools resulted in 1½ weeks delay.
On completion of the main drive difficulty was experienced in pulling back the shield from the headwall at the turning enlargement so that the tunnelling machine could be turned for the drive to the north shaft. The reason for this was that grout had built up on the outside of the tailskin of the machine. The problem was solved by cutting out around the tailskin which enabled the machine to be pulled back into the centre of the chamber. It was not considered to be worthwhile for the remaining short length of drive to expend any more effort in attempting to turn the train of sledge-drawn units and instead extended control lines were fitted between the switch house and the digger shield.

A hand shield was delivered to the site and assembled in the main enlargement in November 1977 for excavation of the 300m long spur tunnel to the west. Face excavation was accomplished using hand-held pneumatically powered clay spades. Apart from teething troubles at the start of the drive when a top plate on the shield buckled and a weld was broken on the thrust ring, a steady rate of progress was achieved. The overall average rate of advance was 3.2 rings per ten hour shift with a maximum rate of 4 rings per shift, see Figure 3.

3.2.2 Lining. A pre-cast bolted concrete segmental lining (Plate 4) was selected by the Consultants as the most economic means of satisfying the Promoter’s requirements. Standard sizes of circular linings were used for the tunnels and also for forming the various enlarged chambers; they were supplied by Charcon Tunnels Ltd. The tunnel rings were 2.44m inside diameter, 2.74m outside diameter and 0.61m wide; each ring consisted of five segments together with a key segment. The different sizes of chambers were constructed in progressive stages of enlargement from the 2.44m diameter pilot tunnel. A temporary concrete lining was assembled at each stage of enlargement which proceeded in increments to a maximum diameter of 5.79m or until the required sized chamber was obtained. The lining for the cable tunnels was erected in the tailskin of the shield and the excavation was advanced one ring at a time; grouting of the annulus behind the lining was generally carried out after two rings had been placed. The grout which contained a fast gelling additive was injected to refusal at a pressure of 400 kN/m$^2$. Caulking of circumferential and cross joints was done some time later. During the machine drive the caulking grooves in the invert of the lining were sometimes damaged by the sledges transporting the switch house and conveyor units and this temporarily prevented an adequate seal being achieved. Where the invert segments were damaged they were subsequently broken out and replaced. The accumulation of sludge in the invert on both machine and hand shield drives also caused caulking problems.

The chambers were formed from the pilot tunnel by replacing one ring at a time. It was specified that at all times the face of the pilot tunnel was to be not less than 10m ahead of the start of the enlargement. Each lining ring was grouted immediately after it was erected and on completion of the chamber the permanent lining was back grouted and caulked.

Assessment of the suitability of standard pre-cast bolted segmental concrete linings for the construction of the cable tunnels and chambers was based upon experience and the Consultant’s knowledge that this form of construction was more than adequate for tunnels driven at the design depth. It was also considered that the lining would be capable of being made acceptably watertight under a hydrostatic head of 15m without having to use special caulking compounds.

3.2.3 Compressed air working. The Consultants anticipated that it might be necessary to use compressed air for construction of the tunnel, and particularly the section of the spur tunnel under the River Taff. However, compressed air working proved to be necessary for the whole of the tunnelling works.

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in order to reduce the inflow of water to a level which allowed excavation to proceed in acceptable conditions and to prevent loss of ground occurring at the face. The required working pressure was normally around 100 kN/m² (approximately 1 atmosphere), corresponding with the full hydrostatic head at the level of the tunnel axis (approximately 10m depth of water). At this pressure water was still able to enter the tunnel invert and it occasionally turned the excavated spoil into a sludge, clogging up the conveyors and causing a mucking-out problem. It was also difficult to caulk the tunnel lining below knee level during construction and this may have been a source of air leakage. To maintain the air pressure in the tunnel and to compensate for air losses, at times air production was required at the rate of about 150m³/minute. The large air losses which occurred were an indication of the very fissured and hence permeable condition of the Keuper Marl along the route of the tunnels and also suggested that satisfactory sealing of the lining during construction was not being achieved.

3.3 Ground observations during construction

Ground observations could not be made on the main drive where the full-face machine was operating as the excavation was screened by the head and the shield. However, on a few occasions when the Contractor claimed that unexpected hard ground was encountered a record in the form of a face section was prepared and lump samples of the rock were broken out for subsequent strength testing. The Keuper Marl was usually described as being firm to strong and generally fragmented, with bands of hard siltstone and sandstone.

Ground observations were made during hand shield driving of the spur tunnel to the west, and typical examples of the face sections prepared are shown in Figure 4. Here the Keuper Marl was described as deep reddish brown with pale green reduction spots and having a lithology described as a fresh, very thinly bedded, arenaceous or argillaceous, possibly dolomitic, calcareous siltstone with scattered gypsum crystals. The variable nature of the ground is illustrated by Figure 4, the two face sections shown being only 28m apart. It can be noted that a cavity is shown in one of the face sections; the presence of cavities in the Keuper Marl having been referred to in one of the records studied during the preliminary investigation (see Section 2.1). Point load strength tests on the lumps of rock sampled from both the main and spur tunnels gave results which, when converted into equivalent unconfined compressive strength values, showed that the harder arenaceous rock varied in strength between the 'strong' and 'moderately strong' ranges of the Geological Society Engineering Group Working Party's classification of rock strengths. The strength of the argillaceous rock varied between the 'moderately weak' and 'moderately strong' ranges. Full details of the strength test results are given in Table 1.

3.4 Settlement and vibration measurements

To monitor the magnitude of any surface settlement caused by the tunnelling operation the Consultants recorded building levels along the tunnel line. Levels were recorded again at intervals as the tunnel progressed and after completion. The results of these measurements showed that virtually no settlement had occurred during construction of the tunnel. In general the levels recorded at each point varied only by ± 1 mm of the initial value which corresponds to the accuracy of the optical surveying procedure used. Additional measurements of surface settlement were made by the Laboratory on two 15m long lines along and at right angles to the tunnel centreline which were set out in a car park to the north-west of the Telephone Exchange. The maximum settlement observed was 1 mm and occurred over the tunnel centreline as the tunnel face
### TABLE 1
Summary of results of rock strength tests made during construction

<table>
<thead>
<tr>
<th>Date</th>
<th>Location</th>
<th>Rock description</th>
<th>Unconfined compressive strength from point load tests(^{(1)})</th>
<th>Rock strength classification(^{(2)})</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Specimen</td>
<td>Number of tests</td>
</tr>
<tr>
<td>June 1977</td>
<td>Main Drive</td>
<td>Purple-red indurated siltstone</td>
<td>lump</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>Upper 1/3 of face</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Lower 2/3 of face</td>
<td>Reddish-brown silty mudstone</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>S.I. 1973</td>
<td>Greyish-green indurated siltstone</td>
<td>core</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>(3) B.H. 8</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>B.Hs 1 &amp; 8</td>
<td>Red silty mudstone</td>
<td>core</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>July 1977</td>
<td>Main Drive</td>
<td>Purple-red indurated siltstone</td>
<td>lump</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td>Face</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>October 1977</td>
<td>West Shaft</td>
<td>Deep reddish-brown sandy siltstone</td>
<td>lump</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>O.D. – 3.5m</td>
<td>Reddish-brown clayey siltstone</td>
<td>lump</td>
<td>10</td>
</tr>
<tr>
<td>March 1978</td>
<td>Spur Drive</td>
<td>Greenish-grey indurated siltstone</td>
<td>lump</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>Top 1/8 of face</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Upper 1/2 of face</td>
<td>Reddish-brown clayey siltstone</td>
<td>lump</td>
<td>16</td>
</tr>
</tbody>
</table>

Notes:
1. Unconfined compressive strength = 16 x \(I_p\) \((50)\) where \(I_p\) \((50)\) is the corrected point load strength index\(^{11}\).
2. Classification based on Geological Society Engineering Group Working Party Report\(^8\).
3. Core specimens from main site investigation (1973) which had been stored for later testing.
passed beneath the measurement site. This settlement is negligible as regards damage to structures above the tunnel. Observations were continued until the tunnel was completed and the removal of compressed air produced no further settlement.

The Laboratory also made measurements of the vibration at the ground surface caused by the tunnelling machine some 13m below. A triaxial geophone array was installed directly over the tunnel centreline on the surface of an open stretch of ground north of the Telephone Exchange. Vibration measurements from the three geophones were recorded on magnetic tape and an ultraviolet chart recorder as the tunnel face passed beneath. Analysis of the results showed that the maximum resultant peak particle velocity observed was 1.2 mm/s. This level is well below the established thresholds for either architectural or structural damage to buildings. The vibrations were occasionally ‘just perceptible’ by an observer standing directly above the tunnel machine during periods of vigorous rock cutting.

3.5 Waterproofing

The tunnels had been completely caulked, grummeted and treated with surface joint coatings before the compressed air was taken off. Air pressure was taken off in December 1978 and immediately there was a substantial inflow of water and a total inflow in excess of 2000 litre/minute was recorded. Stemming the leakages by back grouting, re-caulking, re-grummeting of bolt holes and applying proprietary surface coatings was begun. The Consultants considered that the annulus behind the tunnel lining was well grouted but suspected that the air pressure had forced pathways in the grout even though a fast gelling additive was used. It was also known that the use of compressed air produces an increased temperature in the tunnel and that the resulting cooling when the air pressure was taken off may have caused shrinkage cracks to be formed in the cement grout which then became a source of leakage. The re-caulking of the major leaks was hampered when the compressed air was removed because of the high water flows through the joints of the lining. These remedial measures to the waterproofing of the tunnels were continued for a period of about twelve months during which the total measured inflow into the tunnel was reduced from 1600 litre/minute to under 40 litre/minute, see Table 2. The total inflow has also been expressed per unit internal surface area of the tunnel lining including shafts and chambers as recommended by the Construction Industry Research and Information Association; by April 1980 the measured inflow expressed in this manner was down to 3.9 litre/day/m². In order to control and dispose of any minor seepage of water into the tunnel, drainage channels have been provided in the tunnel floor which lead to a sump at the bottom of a lift shaft in the central enlargement area where the water is pumped to the surface and discharged into the existing sewerage system. Any such leakage occurring between the shoulder and crown of the lining in the form of ‘drippers’ which could cause problems with the power, lighting and alarm systems which are located above tunnel axis, is unacceptable to the Promoter and attempts to deal with these were still in progress in April 1980. The identification of minor leaks was made difficult by the condensation of water vapour on the surface of the lining in some places. The waterproofing work called for a high standard of workmanship.

Joints in the cast-iron sections of lining in the shafts were made waterproof by caulking them with lead followed by a mixture of iron filings and sal-ammoniac, applied twenty-four hours after preparation, (lead and rust).

Waterproofing work in the tunnel was finally completed in June 1980, the works having been handed over to the Promoter in May 1980.
TABLE 2

Total inflow of water measured during re-sealing of tunnels after removal of compressed air

<table>
<thead>
<tr>
<th>Date</th>
<th>Inflow</th>
<th>Date</th>
<th>Inflow</th>
<th>Date</th>
<th>Inflow</th>
<th>Date</th>
<th>Inflow</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>litre/minute</td>
<td></td>
<td>litre/day/m²</td>
<td></td>
<td></td>
<td>litre/minute</td>
<td></td>
</tr>
</tbody>
</table>

3.6 Costs

The tender price for the main contract was £1.9 million (February 1975) and that for the site investigation contract was £6,350 (July 1973). Making no allowance for inflation, the site investigation contract price is 0.33 per cent of the main contract price for the scheme. The final cost of the project is estimated to be £2.32 million at original rates.

4. DISCUSSION

4.1 Comparison of predicted and as-found conditions

The general geology of the site as encountered during shaft and tunnel construction agreed with the geology established during the site investigation as described in Section 2.4. In particular the whole of the tunnel drive was found to lie entirely within the Keuper Marl even though the extra boreholes suggested (see Section 2.6) were not sunk.

The site investigation gave a reasonably accurate assessment of the strength and fractured nature of the Keuper Marl. In general the strength of the rock as found during construction fell into the 'moderately strong' category but some 'strong' rock was also encountered (see Table 1).

Very high rates of inflow of water were experienced as had been suggested in the site investigation report. The Consultants were obliged by the extent and magnitude of the water inflows to order the employment of compressed air working for the whole of the tunnel construction.

The engineering appreciation of the site investigation report also considered there was a possibility that seepage forces could lead to local collapse of the heading when bands of silty clay within the Keuper Marl were exposed in the face. During construction it was seldom possible to observe the condition of the ground through which the tunnel was being driven (see Section 3.3) and therefore the extent and frequency of bands of silty clay could not be noted. However, the use of compressed air and the erection of the lining close behind the shield maintained a stable face at all times. These measures were also successful in preventing surface settlement and avoiding damage to buildings (see Section 3.4).

4.1.1 Strength of the Keuper Marl. The Keuper Marl was described in the site investigation report as being a friable and closely fractured mudstone but the presence of siltstone and sandstone bands was noted. It will be recalled from Section 3.2.1 that at the commencement of tunnel driving, the head of the full-face tunnelling machine was fitted with clay teeth suitable for excavating London Clay. The Contractor's decision to employ clay teeth relied upon any harder material encountered being closely
jointed and hence capable of being broken out by the teeth. These proved inadequate to cope with the
harder material encountered in the Keuper Marl and were replaced with Big-A tools (see Plate 2) which
performed satisfactorily. The presence of hard material in the Keuper Marl, however, was indicated in the
site investigation, the unconfined compressive strengths ranging up to 'moderately strong' rock (see
Section 2.4). Taking the range of unconfined compressive strength values of the Keuper Marl as found
during the site investigation, 10 to 40 MN/m², and comparing this with a high value for the strength of
London Clay, 1 MN/m², it can be seen that the Keuper Marl is at least ten times as strong as the London
Clay. This should have indicated its possible unsuitability for excavation with clay teeth.

4.1.2 Ground water. The site investigation report drew attention to the potentially adverse ground
water conditions, the relatively high permeability of the Keuper Marl and the possibilities of high rates of
inflow being remarked upon in the engineering appreciation (Section 2.6). In the event compressed air
was used during the whole of the tunnel drive in order to exclude water from the works, but extensive
ground treatment was carried out at the bottom of the main work site shaft to reduce the permeability
of the ground to enable the chamber to be constructed there in free air. The compressed air was completely
successful as a construction expedient, but in January 1979 when the tunnel was completed and the
compressed air was taken off, flow of ground water into the tunnel at a rate exceeding 220 litre/day/m²
immediately commenced. Remedial work to the waterproofing measures, described in Section 3.5,
brought the inflow down by August 1979 to 2.5 litre/day/m². In view of the ground water conditions as
suggested by the site investigation, and the difficulty encountered in waterproofing the tunnel after
construction, it would seem that an intrinsically more waterproof type of tunnel lining 14,15 might well
have been chosen for the tunnel than the one in fact used. However the Consultants considered that a
more sophisticated waterproof lining would have been so much more costly that it would have precluded
a tunnel solution for the routing of the cables. In the event, the Promoter has a cable tunnel, which
although not perfectly dry throughout, will be adequate for its purpose.

The site investigation report attributed the high permeability of the Keuper Marl to the presence of
siltstone (see Section 2.5), but it is just as likely to have been due to the presence of joints, there being
many references to the fractured or blocky nature of the Keuper Marl.

The Engineer considered that there were two separate water tables on site, an upper water table in
the gravel and a lower water table in the Keuper Marl, sealed from each other by the silty clay which formed
the top of the Keuper Marl. This was indicated by the sudden influx of water that occurred during
excavation of the shaft at the main work site (see Section 3.1). However, the dual water table theory
remains unconfirmed in the absence of independent piezometric observations in each stratum.

4.2 Other comments on the site investigation

A very good understanding of the geology of the site and of the geotechnical and ground water
conditions was obtained from the preliminary information relating to other previous site investigations
nearby (see Section 2.1). This project therefore demonstrates the large return available at little cost that
preliminary information can provide; all that is required is the time to seek out and acquire copies of the
records. The comments on geotechnical and ground water conditions made in the engineering appreciation
of the site investigation report may have been in some measure based on previous experience or knowledge
of the Keuper Marl and the extent to which this was confirmed by the borehole survey at the Cardiff site.
If this is so it further reinforces the value of existing information.
Because of the change in tunnel route (see Figure 1), the main work site shaft was located some distance from the nearest site investigation borehole, Borehole 7. With hindsight it can be seen that it may have been worthwhile putting down an extra borehole near the main work site shaft as soon as its position had been finalised. If this had been done it may have given forewarning of the ground water conditions at depth in the Keuper Marl which produced difficulties during the sinking of the main work site shaft (see Section 3.1). And piezometers could have been installed to see if there were two water tables present (see Section 4.1.2).

5. ACKNOWLEDGEMENTS

This Report was prepared in the Tunnels and Underground Pipes Division (Division Head: Mr M P O'Reilly) of the Structures Department of TRRL. The following acknowledgements are gratefully made.

Mr C J Kirkland, Mr P Bradbeer and Mr R Ferguson, all of Sir William Halcrow and Partners provided information and records on the site investigation and construction of the project. Sections 2.2 to 2.6 are based on the unpublished site investigation report prepared by Dr S Dyson and Mr J Daley of Cementation Ground Engineering Limited. Figures 1 to 4 are adapted from the site investigation and construction records. Mr R L Moorby of Sir Robert McAlpine and Sons supplied Plates 1 to 4. Mr G H Alderman and Mr B M New provided the information on TRRL's settlement and vibration measurements described in Section 3.4.

Mr N A James of the Wales and the Marches Telecommunications Board, Mr Kirkland of Halcrows and Mr Daley of Cementation are thanked for making useful comments on the draft report.

6. REFERENCES


Fig. 1 Plan of Cardiff cable tunnel showing location of boreholes
Fig. 2 Longitudinal section along tunnel line
Fig.3 Progress of tunnel construction in main drive and spur drive using full-face machine and hand shield excavation respectively
Fig. 4 Face sections recorded on West drive
Plate 1  Front view of tunnelling machine
Cutting tools have not yet been
fitted to cruciform head

Plate 2  View of cutting head during drive, showing final arrangement of
Big-A tools fitted
Plate 3 Main work site shaft, showing bolted cast-iron lining

Plate 4 Cardiff cable tunnel, showing bolted pre-cast concrete lining
7. APPENDIX

PRINCIPAL PARTICIPANTS IN THE CARDIFF CABLE TUNNEL SCHEME

Promoter: The Post Office (Wales and the Marches Telecommunications Board).
Consultants: Sir William Halcrow and Partners.
Contractor: Sir Robert McAlpine and Sons (South Wales) Ltd.
Site investigation contractor: Cementation Ground Engineering Limited.
(Soil Mechanics Department)
ABSTRACT

Site investigation and construction of the Cardiff cable tunnel: G WEST BA MPhil DIC FGS and D McLaren: Department of the Environment Department of Transport, TRRL Laboratory Report 1012: Crowthorne, 1981 (Transport and Road Research Laboratory).

The site investigation and construction records for the 1.5 km long Cardiff cable tunnel have been examined to see whether they can give lessons of good practice for tunnel site investigation. The tunnels were constructed wholly within the Keuper Marl; excavation was mainly by full-face tunnelling machine and lining was with pre-cast concrete bolted segments.

The Report describes the site investigation and construction of the works in some detail with special emphasis on ground conditions. On this site preliminary information proved most useful. The site investigation gave a generally satisfactory account of the site geology. However, points to emerge from the study were that if mechanised tunnel excavation is a possibility and the ground is predominately soft, then bands of harder material at tunnel depth take on extra significance and must be carefully recorded and tested. Rock strength test results in the site investigation report should be carefully considered when making a choice of method. If the operating conditions call for a watertight tunnel close attention should be given to permeability results in the site investigation report which show the possibility of high ground water flows.

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