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JACKED BOX TUNNELLING

Using the Ropkins System™, a non-intrusive tunnelling technique for constructing new underbridges beneath existing traffic arteries

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Jacked Box Tunnelling using the Ropkins System^{TM}, a non-intrusive tunnelling technique for constructing new underbridges beneath existing traffic arteries

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The Ropkins System^{TM} is a non-intrusive tunnelling technique that enables engineers to construct underbridges beneath existing traffic arteries in a manner that avoids the cost and inconvenience of traffic disruption associated with traditional construction techniques.

The paper outlines a tunnelling system designed to install large open ended rectangular reinforced concrete box structures at shallow depth beneath existing railway and highway infrastructure. The system provides measures to control ground movements around the advancing box so that movements of overlying and adjacent infrastructure are minimised and maintained within acceptable limits.

Application of the technique to three projects in the UK is described at Silver Street railway station, Edmonton, London, under the London to Bristol railway at Dorney, Berkshire, and under the M1 Motorway at Junction 15A, Northampton. The influence of site specific conditions and hazards on the design of the tunnelling system adopted is noted together with the performance achieved in practice.

Consideration is given to the manner in which the Ropkins System^{TM} should be selected, developed and executed. The importance of a comprehensive investigation of the ground conditions and infrastructure at the site of the works is stressed together with the need for a client to engage the services of a specialist engineer and specialist contractor experienced in the Ropkins System^{TM}.

***Key words**: Ropkins System^{TM}; jacked box tunnelling; tunnelling; underbridges; infrastructure; geotechnical*

1. INTRODUCTION TO THE TUNNELLING TECHNIQUE

Figure 1 illustrates the first underbridge installed under a live motorway in the UK at Junction 15A of the M1 Motorway. The underbridge measures 14m wide, 8.5m high by 45m long and was cast as an open ended monolithic reinforced concrete box on a jacking based constructed adjacent to the motorway embankment as illustrated in Figure 2(a). A purpose designed cellular tunnel shield was provided at its leading end with thrust jacks mounted at its trailing end reacting against the jacking base. Internal equipment included face excavation and spoil handling equipment, ventilation fans and ducting, essential services and rear access for personnel. Top and bottom proprietary anti-drag systems were installed to minimise both ground drag and friction developed on the extrados/ground interface during box installation; these are described in Section 2.

The box was then jacked to the embankment and entered through the headwall into the ground using a carefully controlled and phased sequence. Tunnelling commenced by carefully excavating 150mm off the face and jacking the box forward a corresponding increment, this sequence being repeated many times [Figure 2(b)]. On the exit side of the embankment a temporary berm was constructed to buttress the embankment during the final stages of jacking. When the shield had reached its final position [Figure 2(c)] the shield and all internal and jacking equipment were dismantled and removed, the box extrados/ground interface grouted, portal wing walls, parapets and roadway constructed, internal services installed and the underbridge opened to traffic [Figure 2(d)].

The technique is suitable for use in many types of ground, both natural and made ground, where face stability can be controlled using open faced shield tunnelling methods. On some projects ground conditions have been improved by geotechnical processes such as dewatering, grouting or artificial ground freezing in order to render them suitable for jacked box tunnelling [Table 1].

In all cases a comprehensive, site specific, site investigation and archive survey is essential to identify the ground types, their strength and stability characteristics, and the ground water regime. Surveys of overlying infrastructure are essential to establish infrastructure type, form, structural integrity and ability to withstand displacements. Using this information the specialist engineer will design a fully integrated tunnelling system comprising tunnel shield and excavation method, spoil handling, jacking system, anti-drag systems and jacking base together with the embankment headwall entry and exit procedures. Risk assessments form an essential part of this process.

2. GEOTECHNICAL INTERFACES AND GROUND CONTROL MEASURES

The principal geotechnical interfaces which have to be addressed during the system design are[1]: -

- the tunnel face
- the box extrados/ground interface
- the jacking base/ground interface

Careful consideration must be given to the method of constructing the jacking base, shield and box, and the allowable construction tolerances and surface finishes.

2.1 The Tunnel Face

Face excavation causes three- dimensional stress redistribution in, ahead of, and around the advancing face accompanied by ground relaxation which results in surface settlement. As the box advances a shallow settlement trough develops along the box alignment whose magnitude and extent are dependent upon the physical properties of the ground, box dimensions, depth of cover, rate of box advance and, most importantly, the support or buttressing characteristics of the tunnel shield.

Each shield is purpose designed to suit the ground conditions predicted from the site investigation and to provide the face support necessary to maintain the integrity of overlying infrastructure. Shield structural components comprise the extrados body with peripheral cutting edges, intermediate stiffening/working decks and dividing walls. Face rams, gun struts, etc. are commonly installed to provide additional face support particularly during headwall entry procedures. Decks and walls give the shield a robust cellular configuration. Cell width and height are determined from the ability of the ground to span and the buttressing force is developed by the thrust jacks reacting against the jacking base and transmitted through the box and shield into the face. The buttressing force has to be carefully calculated and controlled to prevent distress to the shield's cutting edges and ground heave ahead of the face.

Face excavation can be either by hand or machine, or a combination of each and is determined from a study of the ground, sensitivity of adjacent and overlying infrastructure and contractor's preference.

For example, Figure 3(a) illustrates a composite steel and reinforced concrete shield for a 17m wide by 6.2m high rail tunnel installed with 1.7m of cover to the railway tracks at Lewisham railway station, London. The shield was designed for a mixed face comprising loose silt and sand in the top half and soft clay in the bottom half. The steel section had a sloping face and hood section designed to be thrust into the loose silt and sand to provide face and roof support and protection to the miners. The lower concrete section, with its relatively thick walls, was designed to support the soft clay. Each top compartment was hand mined and 360^{o} excavators positioned on the box floor excavated the remaining face area [Figures 3(b) and 3(c)]

2.2 The Box and Shield Extrados/Ground Interface

During conventional pipe jacking[2; 3] instantaneous stress redistribution takes place around the shield resulting in the development of soil arching around the advancing pipeline. Arching reduces the radial loading carried by the advancing pipeline and hence pipe extrados/ground skin friction. Similar effects do not develop around a rectangular section at shallow depth. Instead, the box and shield carry the full overburden and superimposed loads on their flat roofs and transmit them into the ground below. Ground closure occurs along the side walls. Referring to Figures 2(b) and 2(c) it will be seen that as the box is jacked forward there will be a tendency to drag the ground due to the development of skin friction on the shield and box extrados faces. Critical interfaces are as follows.

Box roof
At each stage of advance the shield and box roofs carry a prism shaped volume of ground. If the skin friction developed on the shield and box roof/ground interface exceeds the sum of the shear resistance of the ground along the longitudinal sides of the prism and the passive resistance developed at the front of the prism, the prism of the ground will be dragged forward with the box causing disruption to overlying and adjacent infrastructure.

Box underside
The weight of the box, shield and internal equipment together with the roof loading are transmitted into the ground underneath. As the box advances friction developed on its underside will tend to drag the ground, resulting in shearing and remoulding accompanied by a loss in soil volume which will cause the box to dive.

Anti-drag systems
The technique of jacked box tunnelling, as developed in the UK, has its origins in the pipe jacking technology of the 1960s when a number of rectangular section pedestrian tunnels and bridge abutments were installed using pipe jacking equipment and techniques[4]. These early examples used precast concrete sections, and many were installed at shallow depth through highly variable and weak ground conditions.

The problems associated with ground drag were recognised and a number of anti-drag measures were developed including lubrication with bentonite slurry and the use of reinforced rubber "drag sheets". These had some success but it was not until 1986 when Ropkins™[5] developed its highly successful proprietary wire rope anti-drag system that it became possible to effectively control ground drag.

A typical anti-drag system is illustrated in Figure 4, comprising arrays of closely spaced greased wire ropes anchored to the jacking base with their free ends passed through guide holes in the shield and stored inside the box. As the box advances the ropes are drawn out through the guide holes in the shield and form a stationary layer between the moving box and the adjacent ground. In this manner the ground is isolated from the drag forces and remains largely undisturbed. Further developments and refinements have led to the development of a fully integrated tunnelling system which has made it possible to install the new generation of jacked box tunnels shown in Table 1.

Box sides
It is standard practice to inject a lubricant such as bentonite into the box sidewall/ground interfaces immediately behind the shield's cutting edge.

Grouting
Anti-drag ropes are normally left in situ because their removal would create voids and induce additional unnecessary settlement. Grouting of the box/ground interfaces using cement based grouts commences at the invert, gradually working up the sidewalls and finally over the roof.

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Figure 1: Underbridge at Junction 15A, M1 Motorway

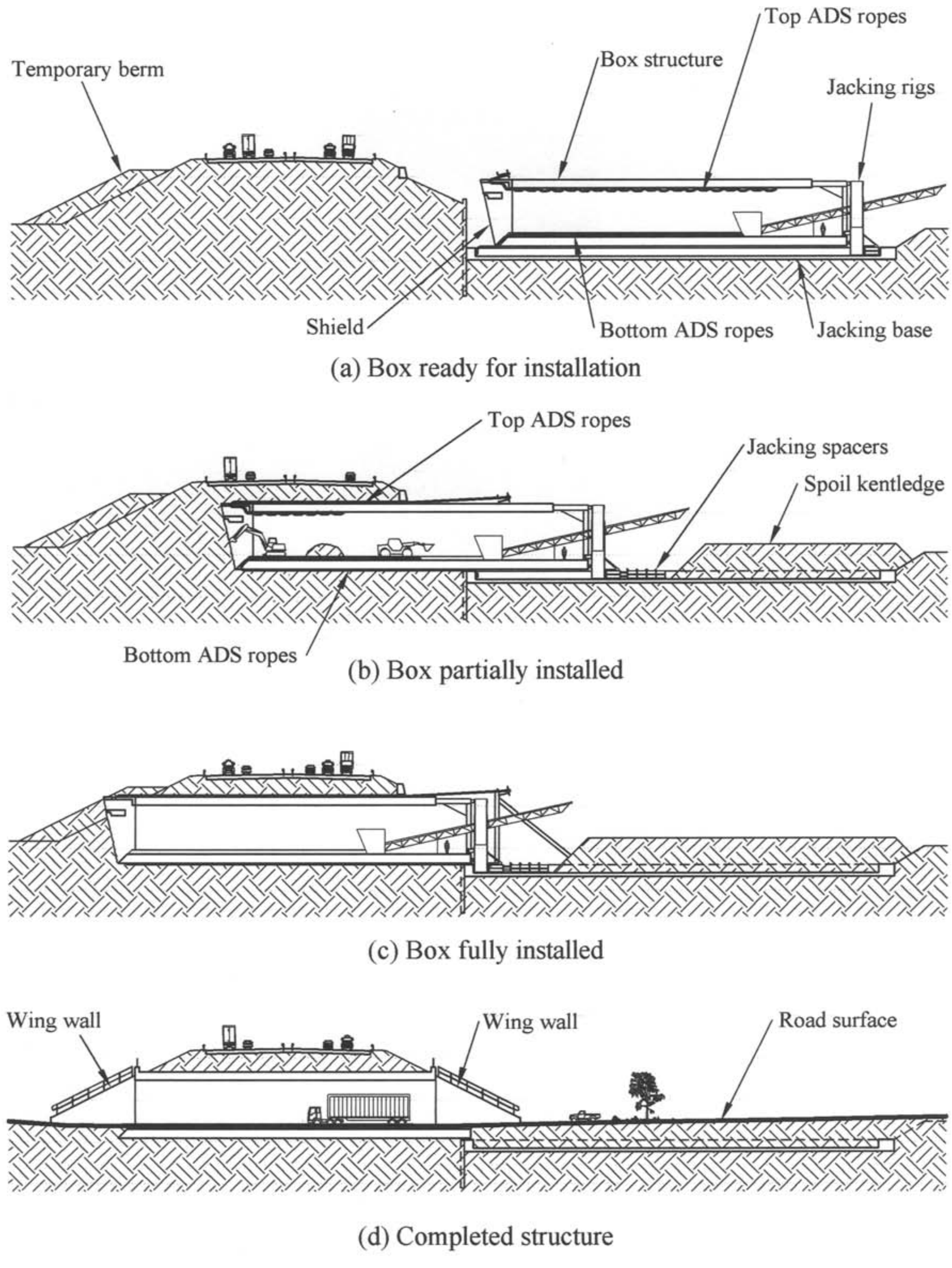
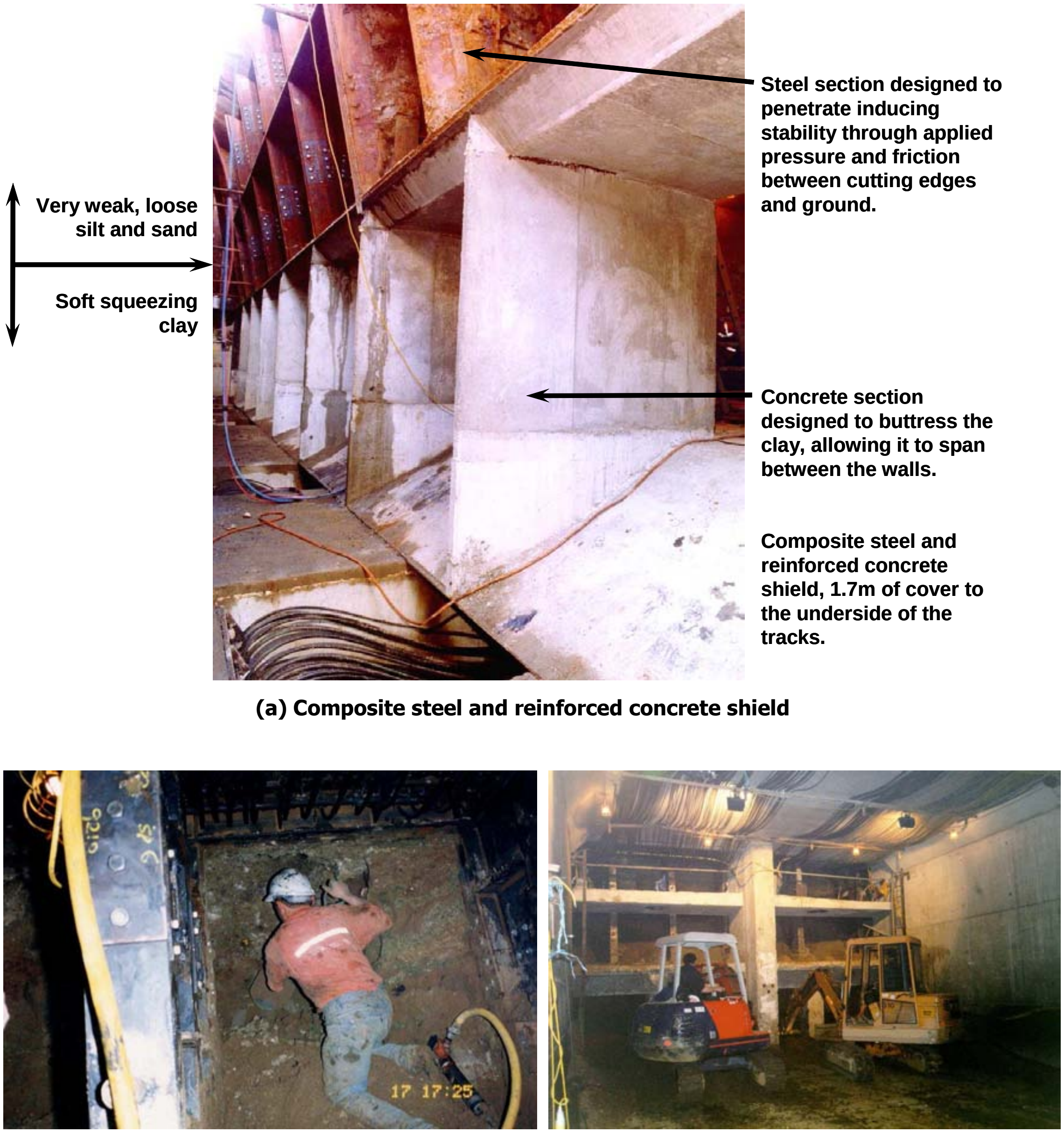


Figure 2: Jacked box tunnel installation

Projects	Size	Cover	Date	Ground Conditions	Ground Treatment	Working Jacking Capacity
Pedestrian and cyclist subway Didcot, Oxfordshire, UK	30m long 5.9m wide 3.6m high	2.0m	1989	Silt-stone fill overlying soft clay	None	2400 tonnes
Highway tunnels West Thurrock, Essex, UK	30m long 16.5m wide 9.5m high	8.0m	1991	Chalk with swallow holes loosely filled with sand	None	6000 tonnes
Highway underbridge Silver Street Station, London, UK	Twin tunnels each 44m long 12.5m wide 10.5m high	7.0m	1995	Water-bearing gravels above over-consolidated clay containing sand layer with water under artesian pressure	Grouting of water bearing gravels. Dewatering of sand layer	6400 tonnes
Rail tunnel Lewisham Railway Station, London, UK	48.0m long 17.0m wide 6.2m high	1.7m	1998	Loose silt and sand overlying soft clay	None	4800 tonnes
3 No subways Lewisham Railway Station, London, UK	Up to 32.0m long 4.4m wide 3.65m high	2.0m	1998	Loose silt and sand overlying soft clay	None	1200 tonnes each
Flood relief culvert Dorney, Berkshire, UK	50.0m long 23.0m wide 9.5m high	6.0m	1999	Clayey granular fill overlying water bearing sands and gravels, overlying weathered chalk.	Artificial ground freezing	7200 tonnes
3 No highway tunnels Boston, Massachusetts, USA	Up to 106.8m long 24.0m wide 10.8m high	6.0m	2001	Weak water bearing strata with numerous man-made structures, tidally influenced water table	Artificial ground freezing	44989 tons maximum
Highway underbridge M1 Junction 15A, Northamptonshire, UK	45.0m long 14.0m wide 8.5m high	1.6m	2002	Pulverised fuel ash and clay fill overlying stiff clay with rock inclusions	None	4800 tonnes

Table 1: Jacked box tunnel projects using the Ropkins System™



(a) Composite steel and reinforced concrete shield

(b) Hand mining in a top compartment through very weak, loose sand and silt

(c) Rear view of the face during excavation

Figure 3: Docklands Light Railway, Lewisham, London. 17m wide, 6.2m high, 48m long

2.3 Jacking Base/Ground Interface

The jacking base / ground interface is a critical surface and must satisfy in-house acceptance criteria based on granular soil density and/or clay soil shear strength. A rigorous testing procedure is undertaken in the interface horizon before finally trimming and compacting to level using a sheep's foot roller. Material which fails to meet the acceptance criteria is excavated and replaced with Type 1 material placed and compacted in 150mm thick layers.

It is common practice to construct the reinforced concrete jacking base inside a shallow depth piled cofferdam and to cast the concrete against the piles to mobilise additional jacking reaction in the form of shear resistance along the side wall pile / ground interface.

In frictional material additional shear resistance can be mobilised by surcharging the jacking base with tunnel spoil.

During box installation the tension developed in the top anti-drag system is taken into the jacking base where it helps to oppose the jacking thrust.

2.4 Construction Tolerances and Surface Finishes

The shield perimeter cutting edge is designed to cut a hole slightly oversize of the box sides and roof in order to relieve ground pressure on the shield and box, and accommodate constructional tolerances. Clearly the designed overcut must be kept to a minimum to control settlement caused by ground movement towards the box sides and roof. This is achieved by ensuring that all external box and shield surfaces are straight, smooth, free from steps and defects, and that opposite faces are parallel and free from twist and distortion. Rigorous control of setting out is essential together with the use of high quality shuttering materials, checking finished surfaces and the rectification of defects.

The top surface of the jacking base must be smooth and constructed accurately to line, level and plane in order to satisfy launch and installation requirements.

Typical construction tolerances applied to the jacking base are:

- ± 15mm variation from target plane for the top surface
- ± 5mm variation in top surface flatness over a 3m baseline
- ± 3mm variation in inclination from the target plane of the jacking thrust reaction plates

Typical construction tolerances applied to the box and shield are:

- ± 15mm variation from the target plane for the top and external sidewall surfaces
- ± 5mm variation in roof flatness and external sidewall straightness over a 3m long baseline
- ± 10mm variation from the target plane of the bottom 300mm of the external sidewalls
- ± 3mm inclination from the target plane across bearing surfaces at the rear of the roof and floor where the jacking frames are attached

It is standard practice to accurately define the three-dimensional shape of the box and shield and make adjustments to the trimming beads attached to the shield cutting edges to allow free passage of the box through the ground. Clearly oversize trimming beads increase overcut around the box sides and/or roof with the potential for increased settlement.

3. SETTLEMENT PREDICTION

Surface settlement caused by conventional tunnel construction results from ground relaxation into the tunnel face and time dependent relaxation of the cut bore prior to primary grouting of the tunnel lining.

A number of empirical models have been developed to predict the profile of the settlement trough. For example, the model developed by Attewell, Yeates and Selby[6], illustrated in Figure 5, is based on an assumption that the transverse ground settlement profile is a normal probability, or Gaussian, form. Empirical evidence supported by settlement data, ground conditions and methods of tunnelling suggests that this is generally valid for many soil types and is substantially insensitive to the method of tunnelling.

Using the model presented in Figure 5 assumptions can be made as to the rate of change of both longitudinal and transverse cross fall, essential in predicting the behaviour of a road surface or rail track(s) in response to an advancing settlement trough.

The authors have accepted the transverse surface settlement trough, illustrated in Figure 5, and applied settlement data and engineering judgement to develop a preliminary model for in house settlement prediction. This recognises the fundamental differences between conventional tunnelling at depth and jacked box tunnelling, notably:

- the rectangular cross section results in a flat section developing along the longitudinal axis of the trough;
- the shallow depth results in an increase in the rate of change of both longitudinal and transverse cross fall and reduction in the overall width of the settlement trough; and
- the potential for time dependent settlement is increased because the box extrados/ground interface cannot be grouted until box installation has been completed.

3.1 Settlement Limits

During the selection and development stages for a jacked box tunnel solution the specialist engineer will establish with his client the settlement criteria for the overlying infrastructure consistent with its safe operation. For underbridge construction beneath existing highways and railtracks this takes the form of a specified speed reduction and settlement criteria typically including the rate of change of longitudinal and transverse cross fall with a maximum value for vertical settlement/heave. Settlement criteria must be related to trigger levels and robust procedures established for monitoring settlement and the actions to be taken if trigger levels are exceeded.

4. TUNNELLING SYSTEM DESIGN

The main components of the Ropkins System™ are the shield, anti-drag systems, jacking system, jacking base and jacking pit headwall entry procedures. These interact with one another and the specialist engineer must determine their optimum configuration to achieve simplicity of design and installation.

4.1 Calculation of Jacking Loads

Figure 6 illustrates the principal forces during box installation. The jacking load developed by the jacking rigs is a combination of the shield buttressing load and the drag loads at the top, bottom and sides of the shield and box structure.

In both frictional and cohesive soils where the box is separated from the soil by anti-drag ropes, drag is frictional, concrete against wire rope and is a function of contact pressure. Where frictional and cohesive soils are in direct contact with the box drag is a function of friction and adhesion respectively. Lubrication of sliding surfaces is essential to minimise these forces.

Extensive in-house testing has been performed to determine the friction factors for greased wire ropes sliding on a variety of concrete surface finishes. By rigorously back-analysing jacking loads recorded during tunnel jacking projects it has been possible to validate and refine design friction factors and adhesion values.

4.2 Jacking Equipment

Jacking equipment comprises jacking rigs, cross beams with spacer pieces and power pack with operator's console.

Each jacking rig has a vertical structural member which rests on the floor at the rear of the box and bears against a thrust plate and anchorage cast into the end face of the box floor and roof respectively, see Figure 7. A cluster of six 1.2m stroke double acting hydraulic jacks is mounted on the lower end of the jacking rig. Jacks are rated at 200 tonnes, giving a maximum jacking thrust of 1,200 tonnes per jacking rig. Hydraulic power is provided by a diesel hydraulic power pack.

The longitudinal and cross trench system, illustrated in Figure 7, is accurately constructed in the jacking base for each jacking rig. This permits the jack cluster to thrust against a cross beam located on the horizontal axis of the jacking base thus minimising bending moments and simplifying base reinforcement. Thrust is transmitted through the jacking base into the surrounding ground. Spacers are inserted between the jacking cluster and cross beam as the box advances and then the cross beam is advanced incrementally.

The number of jacking rigs required is determined from the jacking load calculation. Each rig is piped directly to the operator's console so that he can advance the box using the required jacking thrust.

4.3 Jacking Procedure

In house procedures have been developed for each stage of box advance. It is essential that a designated person is at the face during jacking to observe the box advance and ensure that the shield does not sustain damage. He must be in direct voice contact with the console operator so that the advance can be halted at any moment.

The console operator sets the hydraulic pressure and relief valve to satisfy the requirements of a “chainage/jacking load” graph, prepared by the specialist engineer, and operates within strict guidelines.

5. TEMPORARY AND PERMANENT WORKS DESIGN ELEMENTS

The shield, jacking base and jacking pit structure are classed as temporary work elements. Following box installation the shield is dismantled and the jacking base backfilled to form a sub-base for the road or track bed. Jacking pit piles are either extracted or built into the permanent retaining walls depending upon local topography, alignment and gradient requirements.

The box is classed as the permanent works structure and must satisfy stringent design and material specification requirements based on strength and durability with a design life of typically 100 years.

Steel reinforcement must be detailed to give the required concrete cover, normally 50mm, to prevent rusting of the reinforcement. Where reinforcing bars have to be lapped for continuity the laps and intensity of reinforcement must be detailed so that the concrete can be placed and vibrated to ensure an intimate bond with the reinforcement. Typical reinforcement densities in a reinforced concrete shield and box are 120 and 220kg.m^{-3} respectively.

Concrete durability is critical and can only be achieved using good quality aggregates and cement, correctly proportioned, mixed, placed and compacted to achieve a dense impervious concrete. When concrete cures heat develops from hydration of the cement which may lead to cracking and long term deterioration of the structure. This can be controlled by one or more of the following measures, anti-crack reinforcement, the replacement of up to 35% of the weight of the cement with pulverised fuel ash and a carefully designed and detailed sequence of concrete pours. Proprietary concrete additives may be introduced into the concrete mix to aid workability, particularly where the concrete is placed by pump.

Typical concrete strengths for a shield and box at 28 days are 40 and 50Nmm^{-2} characteristic strength respectively. These mixes will give permeabilities in the order of 10^{-14}m.sec^{-1}.

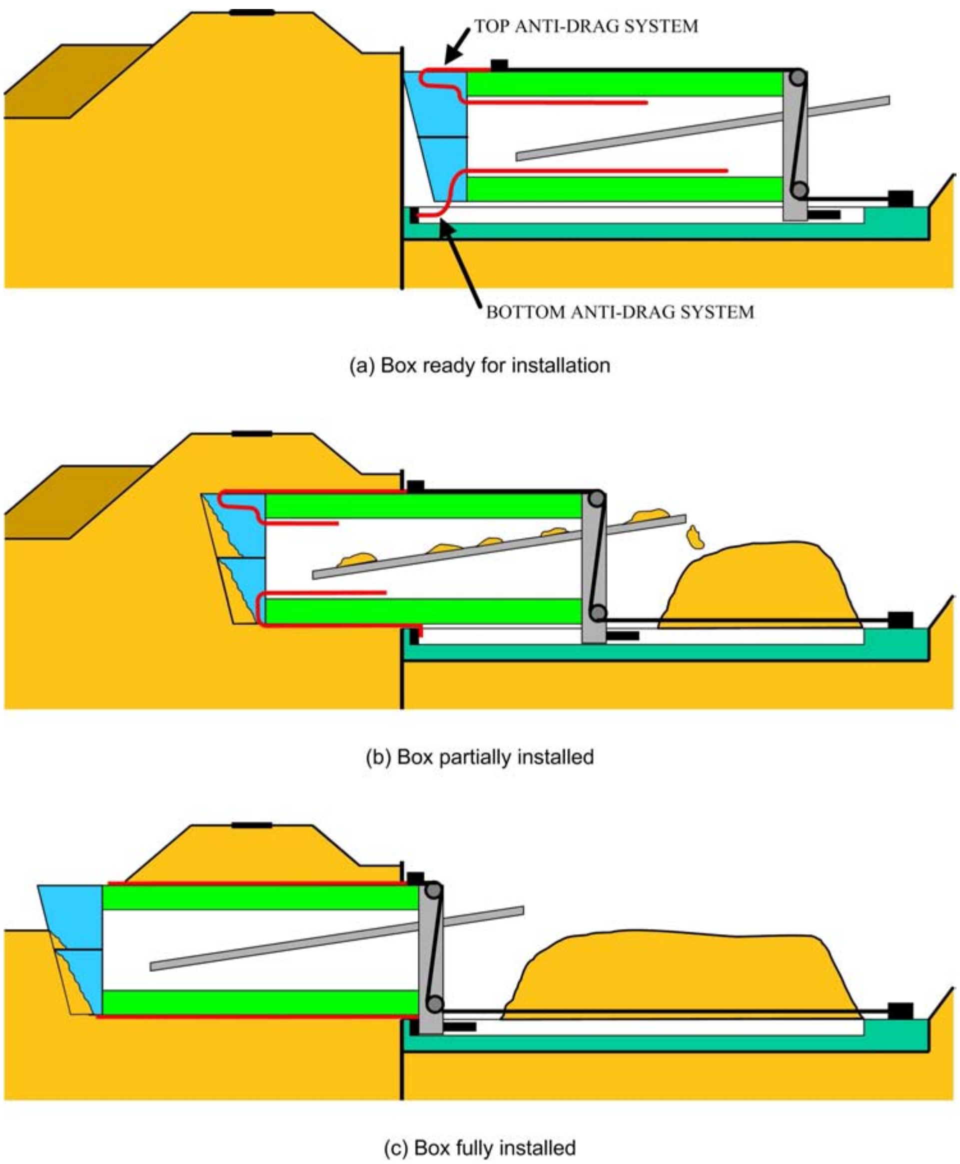


Figure 4: Top and bottom anti-drag systems

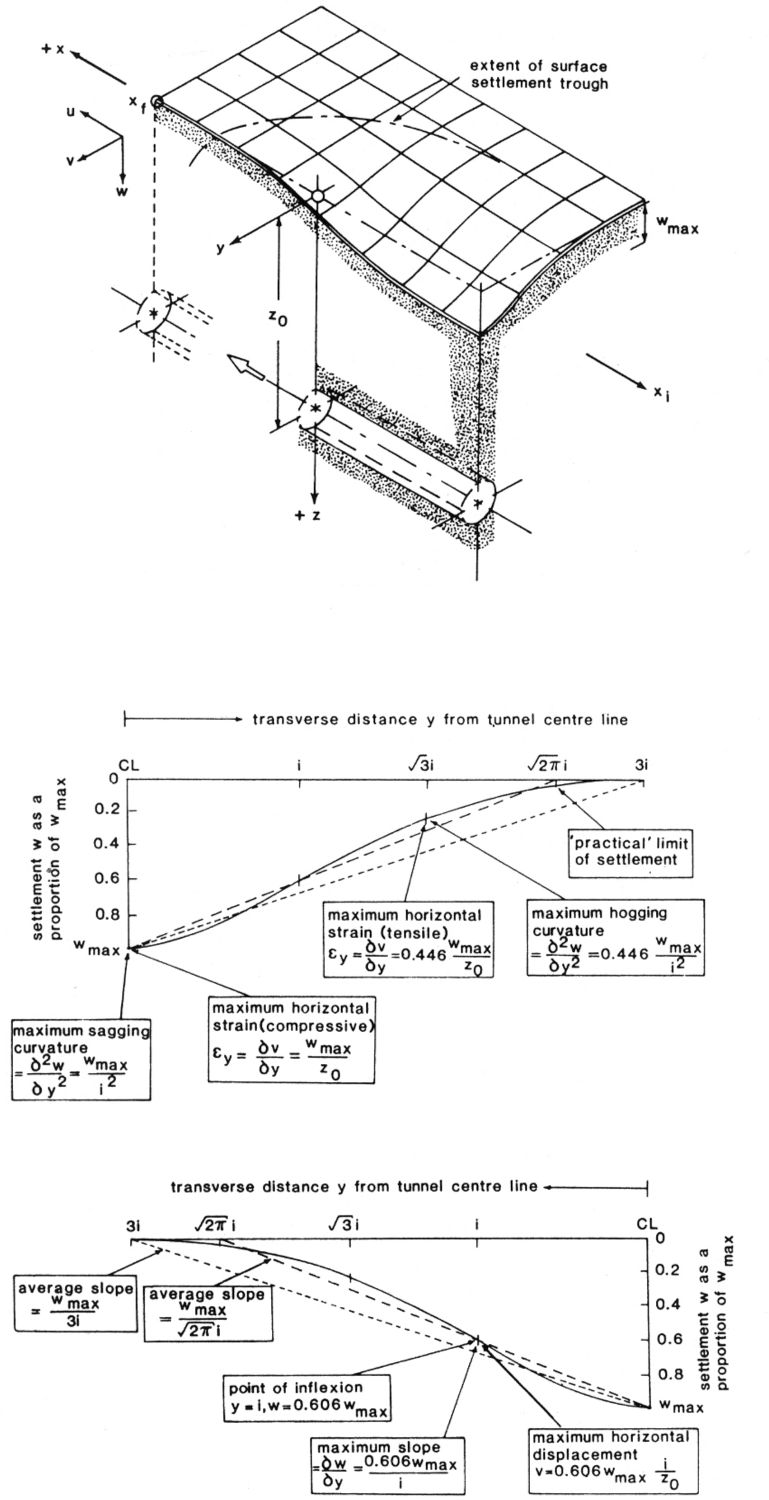
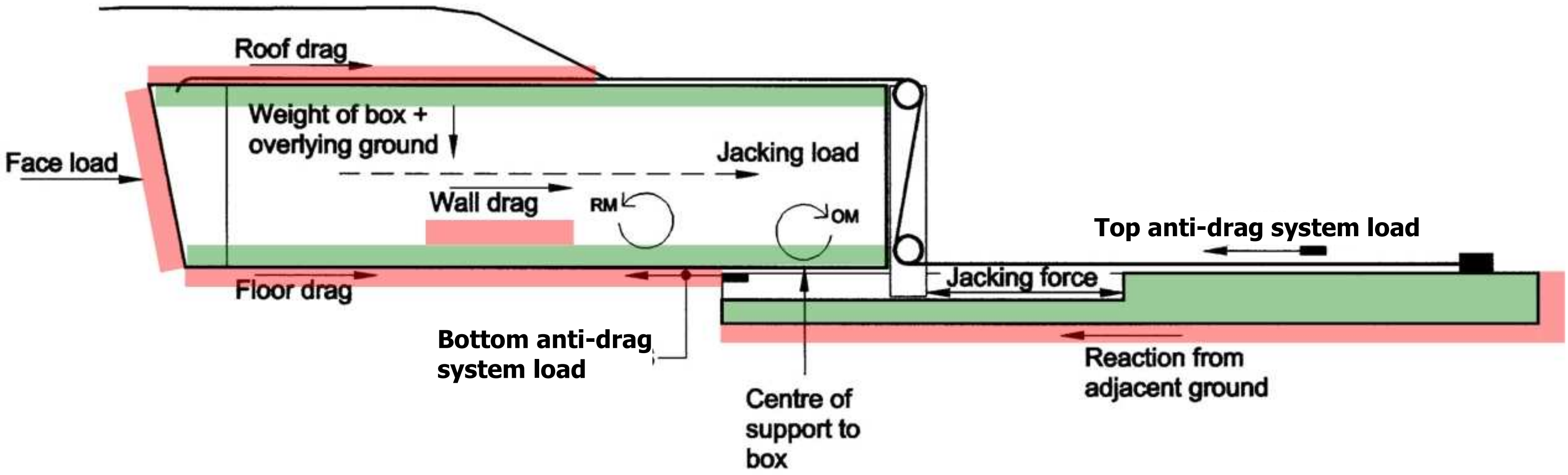


Figure 5: Settlement trough development for a circular tunnel at depth (after Attewell, Yeates and Selby, 1986)



Jacking force = face load + roof drag + floor drag + wall drag.
Reaction from adjacent ground > jacking force - anti-drag system loads.

As the box advances:

- **its centre of support on the jacking base changes, and**
- **an overturning moment (OM) is induced by the eccentricity of the jacking force which is countered by the restoring moment (RM) of the embedded section of**

Figure 6: Jacked box tunnel installation forces

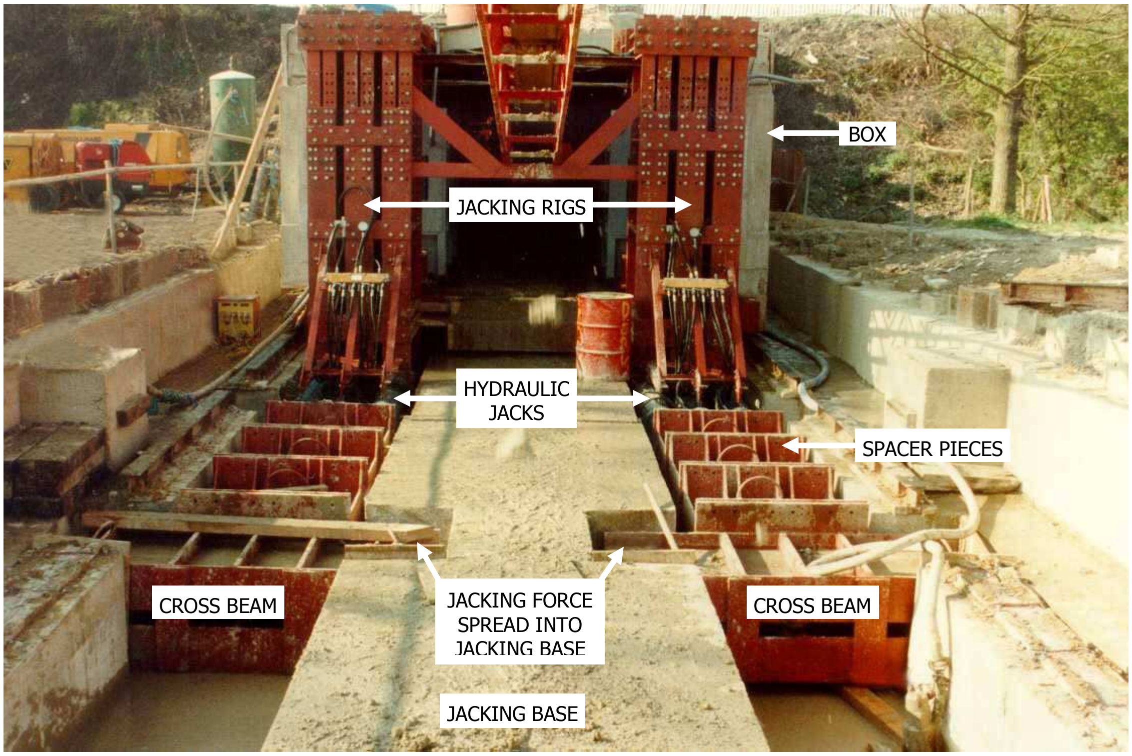


Figure 7: Jacking equipment

6. EXAMPLE PROJECTS

The full range of projects carried out using the technique described in this paper is listed in Table 1. Three interesting UK projects are described in this section to illustrate the versatility of the technique

6.1 Silver Street Railway Station, London

This project was to construct a 44m long section of vehicular underpass beneath the platforms and railway tracks of Silver Street railway station in Edmonton, north London comprising two boxes placed side by side each 12.5m wide and 10.5m high, see Figure 8. Ground conditions comprise made ground overlying water bearing gravel, which in turn overlies London Clay beneath which there is a layer of water bearing sand. The ground water table is situated just above the top of the proposed underpass.

Sensitivities

- Railway tracks and station to remain fully operational at all times in accordance with Network Rail Operating Requirements.
- Integrity of station structures to be maintained and the maximum cumulative settlement limited to 75mm.
- Signalling, communication and power cables mounted on the station structures to be safeguarded.
- Adjacent abutment supporting railway bridge over busy main road to remain undisturbed.
- Restricted working space, access and noise limits due to close proximity of main road and residential properties.

Particular hazards

- Very difficult ground conditions with a high water table.
- Uncontrolled ground loss in the tunnel face.
- Excessive drag induced ground movement.
- Excessive ground movements associated with the construction of the jacking pit.
- Damage to station structures and adjacent bridge.
- Damage to, or failure of, the signalling, communication and power cables.

The solution

Prior to tunnelling the water bearing gravel was stabilised by grouting using the tube-a-manchette system within a jet grouted cut off curtain surrounding the perimeter of the proposed tunnels. Horizontal pressure relief drains were installed to relieve artesian water pressure in the sand layer.

Because of the limited working space it was not possible to construct a large jacking pit capable of containing the tunnel boxes required. Instead a relatively small jacking pit was used and the boxes were constructed at ground level as a series of 1.5m long counter-cast interlocking segments, see Figure 9. Each tunnel comprised 30 segments each weighing 160 tonnes.

Vertical thrust walls were incorporated at the rear of the jacking pit to receive thrust from the jacks mounted in the purpose designed thrust frame illustrated in Figure 10. Jacks were arranged and valved in groups to provide the vertical and horizontal steerage necessary in the early stages of tunnelling when the length of each box being jacked was short. Two reinforced concrete cellular shields, illustrated in Figure 11, were constructed in the jacking pit and subsequently mated with tunnel segments lowered by crane from the surface, see Figure 12.

The north tunnel was installed first followed by the south tunnel which was jacked sideways off the north tunnel during its installation in order to maintain adequate support to the ground underlying the adjacent bridge abutment. The proprietary wire rope anti-drag system was used at the top and bottom of both tunnels and on the side of the south tunnel adjacent to the bridge abutment. On completion of each tunnel the face was boarded, the top anti-drag system ropes were pulled out for re-use elsewhere, the box extrados/ground interfaced grouted and longitudinal tendons connecting the segments were fully stressed.

Performance achieved

Once the difficult operation of shield entry through the jacking pit headwall had been achieved each tunnel took approximately 4 weeks to install with a maximum jacking thrust of 5,500 tonnes. In spite of the need for careful steerage in the early stages of tunnelling both boxes were installed to a positional accuracy within 25mm on both line and level.

As the railway tracks carried a temporary speed restriction of 20mph during tunnelling it was only necessary to fettle the tracks on the completion of each tunnel.

The maximum aggregate settlement of the ground and overlying structures arising from the installation of both tunnels was within the 75mm maximum specified and because the settlement curve was extremely shallow the very small gradients induced were easily accommodated by the brickwork structures. It was only necessary to resurface the platforms and relay the platform copings in the settlement zone.

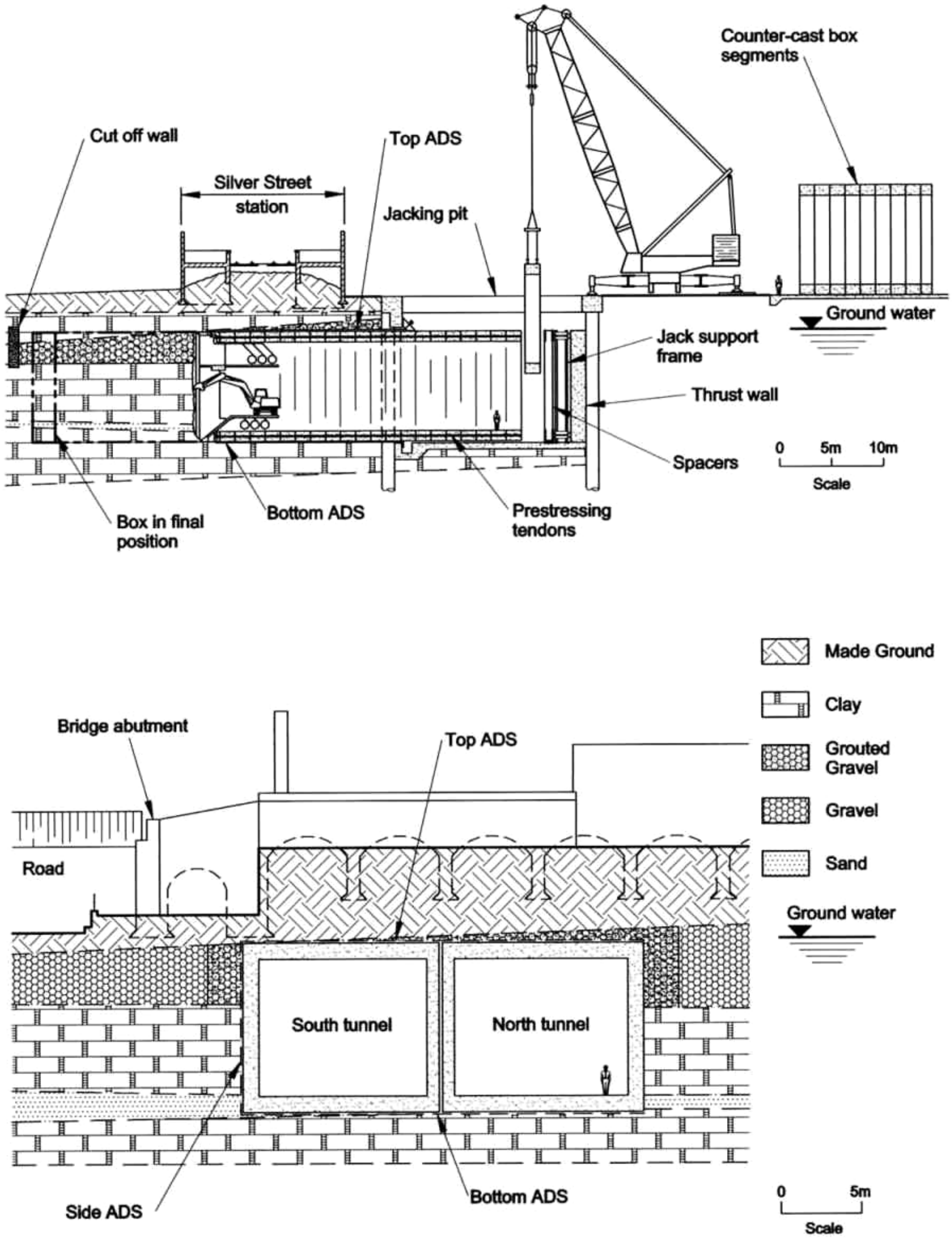


Figure 8: Vehicular tunnels, Silver Street, London

Figure 9: Counter cast tunnel segments at ground level in constructed site

Figure 10: Purpose designed thrust frame reacting against headwall and counter cast segments

Figure 11: Reinforced concrete cellular shields

Figure 12: Lifting counter cast tunnel segment into jacking pit with railway station and bridge in the background

6.2 Flood Relief Culvert, Dorney

Known as Dorney Bridge the culvert forms a vital link in the Environment Agency's Maidenhead, Eton and Windsor Floor Relief Channel carrying peak flows of the River Thames under the London to Bristol main line railway near Dorney. The culvert measures 23.0m wide, 9.5m high by 50m long, and is the largest jacked box tunnel in the United Kingdom[**7**], see Figure 13.

Ground conditions at the site were challenging requiring extensive ground water lowering in the jacking base area, and ground freezing, both along and around the culvert alignment to stabilise the waterbearing sands and gravels

Sensitivities

- Work to be undertaken alongside and through a 12m high railway embankment.
- Railway equipment, signalling, communication and power cables to be safeguarded.
- Rail tracks to remain fully operational at all times in accordance with Network Rail Operating Requirements.
- Line speed of 125mph to be maintained throughout the construction period with a reduction to 60mph during culvert installation.
- Integrity of the railway embankment not to be compromised.
- Noise restrictions because of adjacent residential properties and local amenity centre.

Particular hazards

- Challenging ground conditions, sands and gravels overlying the chalk aquifer with a ground water level 1.5m below ground level.
- Original railway embankment constructed circa 1835 to 1839 using sands and gravels, then widened on the north side with clay at a later date.
- Uncontrolled ground loss in the tunnel face.
- Excessive drag induced ground movement.
- Failure of the railway embankment and railway infrastructure.
- Unauthorised intrusion of plant and personnel on the operational railway.

The solution

The jacking base area was constructed in a battered and dewatered excavation 7.7m deep enclosed on three sides with a bentonite/cement slurry cut off wall linked into diaphragm walls designed to form the permanent training walls, see Figures 14 and 15. Temporary steel sheet piles were driven between the diaphragm walls to close the cut off wall and form an entry headwall to the railway embankment.

The presence of a high water table in granular soils cannot be tolerated in a jacked box tunnel because the water will wash soil from the tunnel face and box/soil interface leading to inundation and surface settlement. It was considered that this problem could be solved by injecting weak cement grout into the tunnelling horizon in advance of tunnelling, as had been done at Silver Street, London, see Table 1. Grouting trials carried out in the railway embankment and jacking pit indicated that the security of the railway embankment could not be guaranteed by the grouting methods proposed. Therefore, based on studies carried out by the authors for the jacked box tunnels on the Boston Central Artery, USA[**8**], it was decided that artificial ground freezing would be used to control the ground water and stabilise the tunnelling horizon in the manner illustrated in Figure 16.

The conventional method of freezing ground is to drill an array of vertical holes for freeze pipes through which chilled brine at -30℃ is circulated. This was not practicable acros s the railway embankment. Instead, a total of 180 number, 125mm diameter freeze pipe and instrument holes were horizontally directionally drilled through the railway embankment on a 1.5m grid extending 3m outside the box extrados to form a cut-off zone. Plastic freeze pipes were installed inside the box and steel freeze pipes in the cut-off zone. Careful monitoring of both ground and return brine temperatures and the use of valves to adjust the brine flows enabled the frozen zone to be developed and maintained with the minimum of ground heave.

It took 3 months to completely freeze 20,000m^3 of ground using four brine chilling plants producing a total cooling capacity of 880,000 Kcal hr^{-1} at -25℃. Each chilling unit was rated at 250hp and used ammonia as a primary refrigerant[**9**].

At the phase change from water to ice there is a dramatic increase in ground strength and stability, see Figure 17. Unfortunately this is accompanied by expansion resulting in ground heave which is more severe in clay soils than in permeable ground such as sands and gravels because the water cannot readily escape ahead of the freeze surface due to the lower permeability. However, heave development was predictable and slow, and readily accommodated by occasionally fettling the tracks during night-time maintenance possessions in response to track level readings. Maximum heave occurred in the clay embankment widening where an aggregate heave of 180mm was recorded, whilst in the original embankment aggregate heave did not exceed 40mm.

Top and bottom anti-drag systems were installed. The top comprised 800 number, 13mm diameter steel wire ropes covering 90% of the box roof and the bottom had 800 number, 13mm diameter steel wire ropes arranged in four 3m wide "tracks". Calculations were performed to ensure that the box/ground and box/anti-drag system interfaces would not freeze resulting in the box becoming locked into the ground. These showed that the residual heat of hydration contained in the box concrete plus the heat generated by the excavation and soil disposal equipment would be sufficient to avoid freezing of the roof and wall interfaces. However, electric trace heating would be needed in the box floor to prevent the floor interface from freezing. During box installation low temperature grease was injected into both anti-drag systems and a salt gel bentonite used to lubricate the sidewalls.

The dramatic improvement in the strength and stability of the tunnelling horizon permitted an open face shield configuration to be adopted with four full height compartments each 5.75m wide. The shield cutting edges were heavily armoured and designed to trim localised pieces of frozen ground and provide a guide for the face excavation equipment.

The top half of the face and lower sidewalls were excavated using a Webster Schaeff 120HD Transverse Cutting Unit mounted on a purpose designed boom assembly carried on a Komatsu PC340LC, 360° tracked excavator[**10**], see Figure 18. Careful sizing of the carrier and boom geometry, which incorporated an "inline" rotation facility, permitted the transverse cutting unit to accurately cut and profile the sidewalls and roof of the rectangular excavation and cut into the top corners. The lower half of the face was excavated using the Webster Schaeff 2000CL Excavating Machine[**11**] illustrated in Figures 18 and 19, equipped with a fully articulating cutting head. Both cutting units were dressed with Kennametal point attack picks.

Jacking thrust was provided by six jacking rigs each equipped with six number, 1.2m stroke double-acting hydraulic jacks, giving a total working capacity of 7,200 tonnes and an ultimate capacity of 10,800 tonnes.

Performance achieved

Ground freezing proved to be highly successful as did the measures taken to prevent the box/ground and box/anti-drag system interfaces freezing. The tunnelling operation proceeded in an orderly fashion taking a total of 32.5 days working 24 hours per day, 7 days per week.

Maximum aggregate settlement recorded during culvert installation was 55mm, comfortably within the 75mm predicted. As settlement occurred very slowly only occasional night-time fettling was needed to maintain the railway tracks within the specified operating tolerances and train services were not interrupted.

The freeze plant was turned off at breakthrough and the culvert/ground interfaces grouted. It took three months for the frozen ground to thaw completely and the maximum recorded aggregate track thaw settlement was 40mm. Again occasional night-time fettling maintained the railway fully operational at all times.

Figure 13: Flood alleviation culvert, Dorney, Berkshire

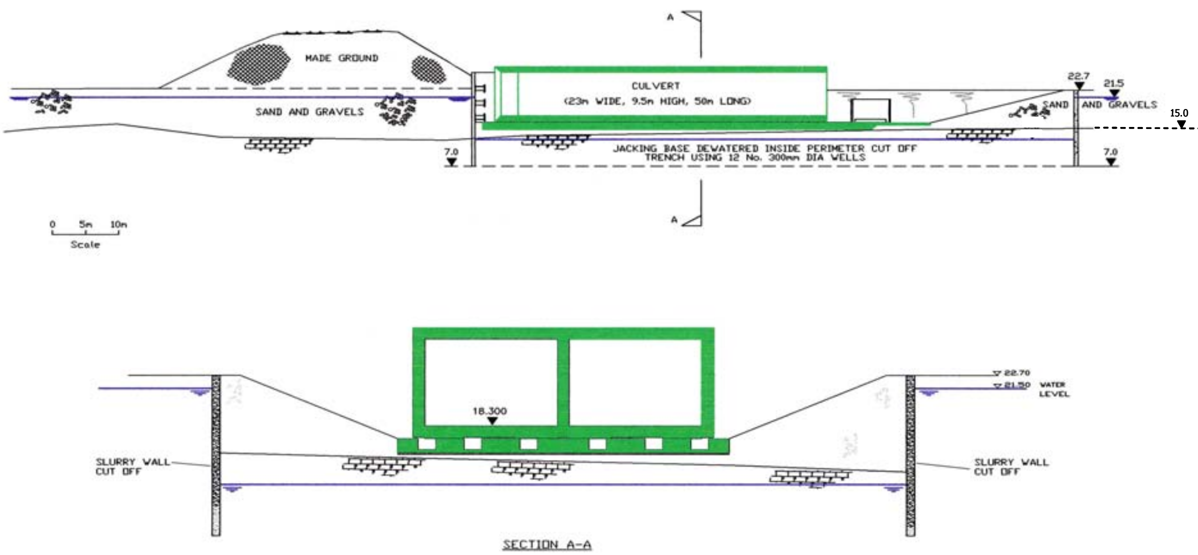


Figure 14: Section showing the box prior to installation

Figure 15: Box under construction

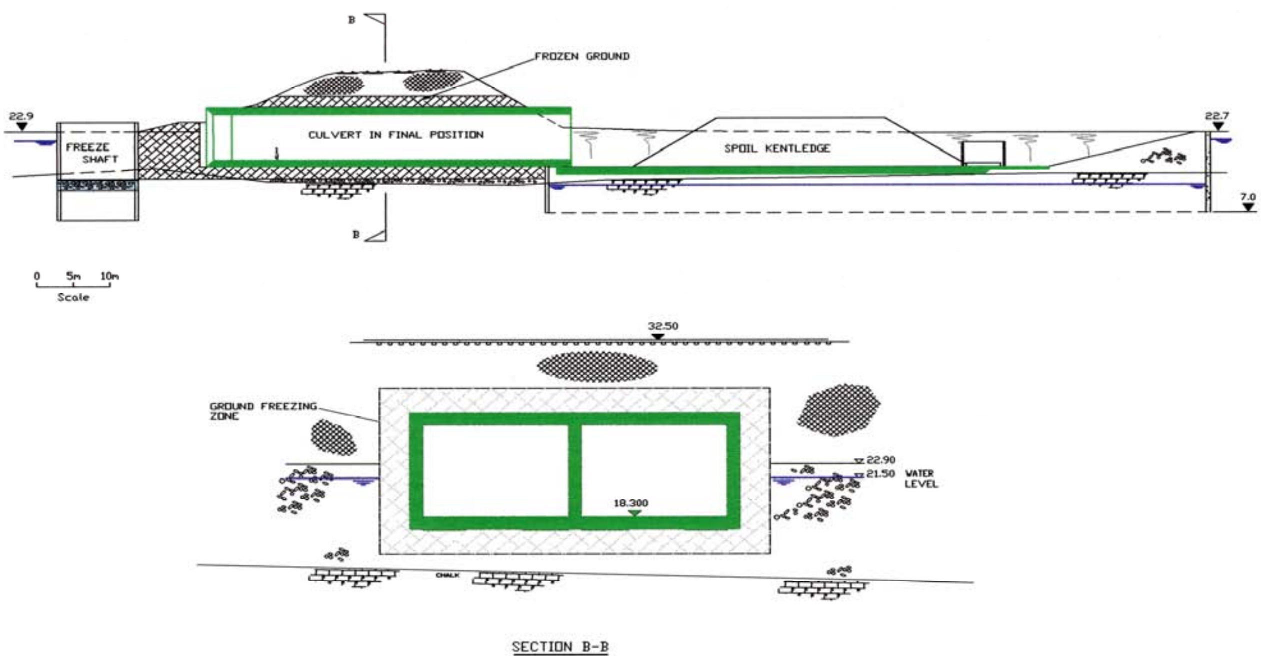
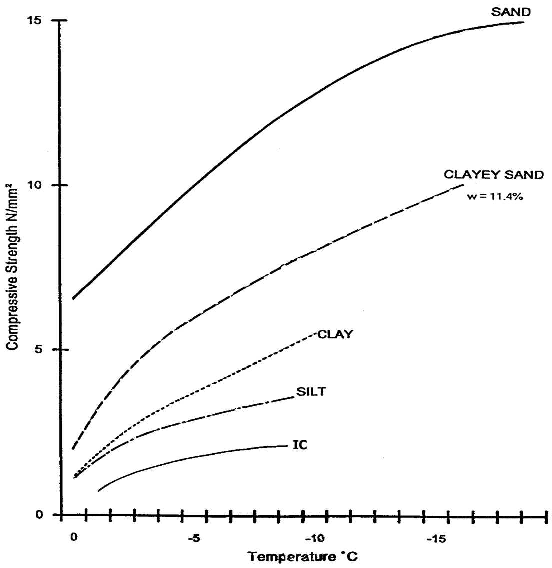
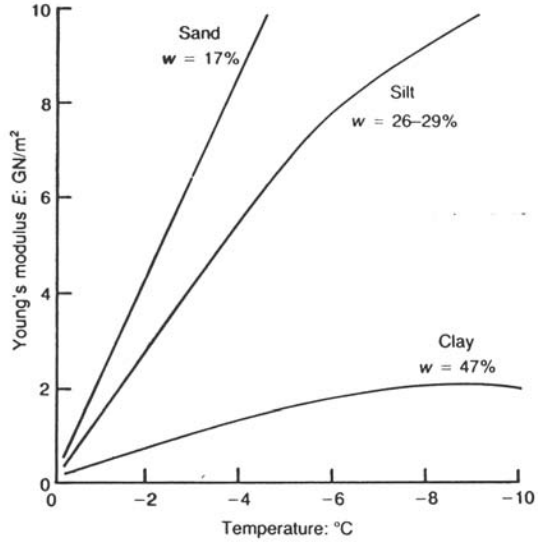


Figure 16: Section showing the culvert fully installed with the frozen ground section

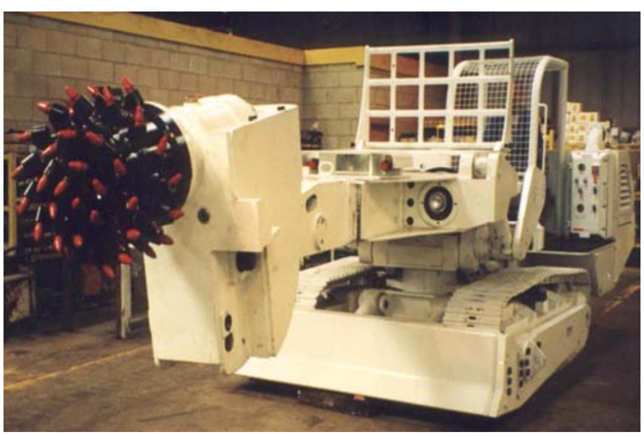




Typical Compressive Strengths of Frozen Soils in Relation to Temperature
(Handbook of Geophysics - US Air Force)

Average Young's Modulus from cyclic loading tests
(Handbook of Geophysics - US Air Force)

Figure 17: Increase in ground strength and Young's Modulus with decrease in temperature



90kW diesel powered Webster Roadheader with fully articulated cutting head

120kW Transverse Cutting Unit on a fully articulated arm, mounted on a 34 tonne Komatsu excavator

Figure 18: Excavating equipment

Figure 19: Excavating the lower half of a 10m high face through frozen waterbearing sands and gravels

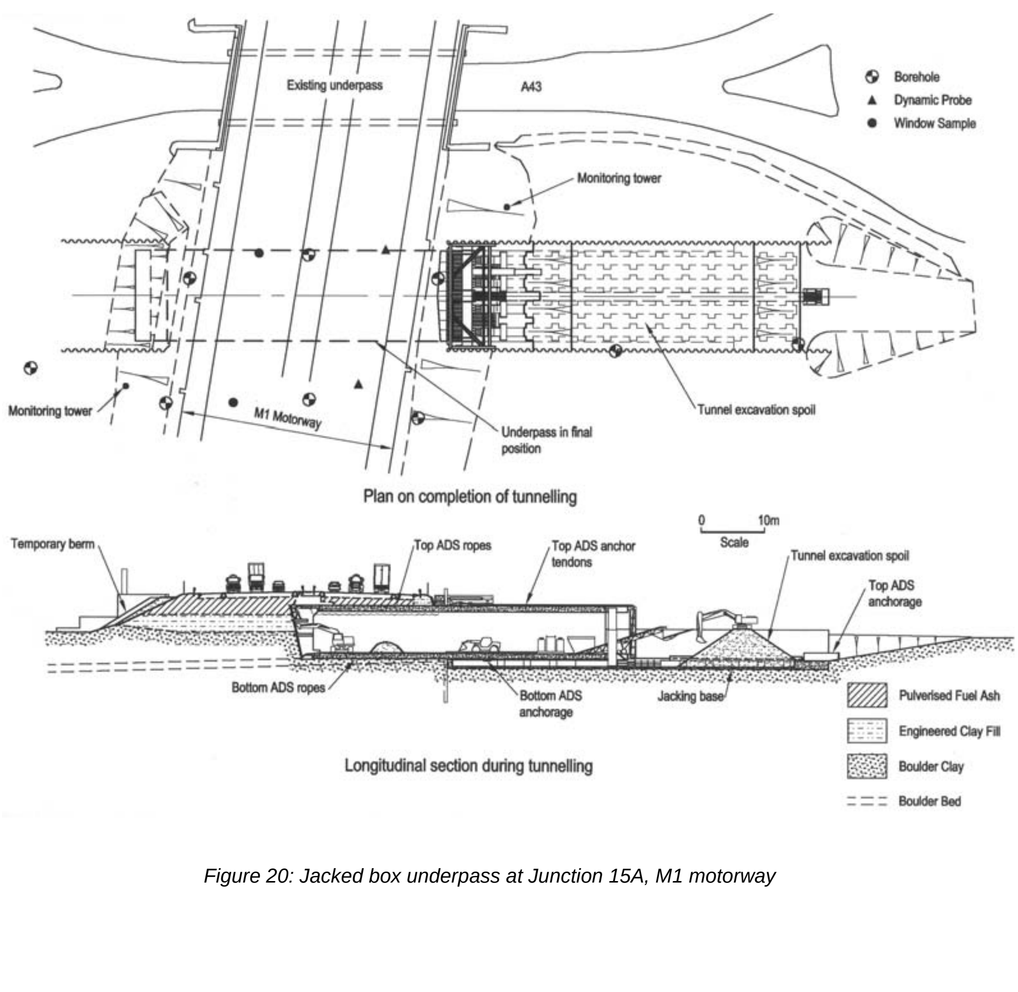


Figure 20: Jacked box underpass at Junction 15A, M1 motorway

6.3 M1 Motorway, Junction 15A, Northamptonshire

This project was to enhance the capacity of Junction 15A by constructing a vehicular underpass 14m wide, 8.5m high by 45m long alongside the existing A43 underpass, see Figures 1 and 20. Existing roundabouts at both ends of the A43 underpass were required to be remodelled to handle traffic flows from the new underpass and to provide dedicated accesses to future industrial developments.

Existing road levels, gradients and underpass dimensions dictated a minimum clearance of 1.6m between the vehicular underpass roof and the motorway running surface.

Ground conditions through the motorway embankment comprise 0.8m of road construction overlying 3.5m of compacted pulverised fuel ash, which in turn overlies 1.7m of engineered clay fill founded on boulder clay. The ground water table is approximately 1.0m beneath the underpass structure.

Sensitivities

- Motorway used by 112,000 vehicles each day.
- Major motorway intersection.
- Adjacent to an existing underpass.
- Close proximity to the Grand Union Canal, streams, industrial estate and motorway service areas located on both sides of the motorway.
- Stringent settlement criteria based on trigger levels, which if exceeded, could lead to the cessation of construction works, closure of those sections of the motorway affected and resurfacing.
- Scaled lane rental costs for the closure of one or more lanes, commencing at £1,000 per hour, per direction, per lane increasing to £20,000 per hour (£480,000 per day), per direction, for the closure of one carriageway.

Particular hazards

- Insufficient ground information, in particular the degree of cementing of the pulverised fuel ash.
- Uncontrolled ground loss in the tunnel face.
- Uncontrolled heave of ground ahead of advancing shield.
- Excessive drag induced ground movement.
- Excessive ground movements associated with the construction of tunnel portal works.
- Damage to the existing A43 underpass structure.
- Damage to the motorway drainage system with possible water ingress into the advancing tunnel face.

The solution

To jack a monolithic box 14m wide, 8.5m high by 45m long under the motorway from a steel sheet piled jacking pit, with integral jacking base[**12**]. Figure 21 shows the box partially installed. Special provision was made at the jacking pit headwall to facilitate an efficient and timely entry into the embankment. The reception works comprised steel sheet piled wing walls with a clay berm to buttress the embankment during shield breakthrough, see Figures 20 and 23.

Immediately following contract award a number of slit trenches were machine excavated in the embankment side slopes to investigate the short-term strength and stability of the pulverised fuel ash and engineered clay fill, and to observe the integrity of the interfaces. One trench remained open for six months without showing signs of instability.

Motorway drains running under the hard shoulders were intersected at each side of the box alignment and diverted down the embankments into the A43 drainage system.

An open-face cellular reinforced concrete shield divided into three working levels, each with seven compartments, was designed to support the face, see Figure 22.

Key aspects of shield design were:

- The top-level compartments were designed for hand excavation and the lower and middle level compartments for machine excavation using 360º excavators, with an option of hand excavating inside the middle level compartments should the need arise.
- The top and middle level decks were conveniently positioned at the pulverised fuel ash/engineered clay and engineered clay/boulder clay interfaces respectively.
- Each of the top and middle level compartments was equipped with face rams designed to support face timbering and automatically pressure-relieve as the box advanced.

Prior to finalising the shield's internal dimensions, particularly the working deck length, a full-size vertical section of the shield was constructed in scaffolding. This permitted a number of excavator types to be entered to determine their footprint, working space requirements, most advantageous boom/dipper configuration, and bucket curl geometry.

During face excavation a minimum cutting edge penetration of 450mm was maintained. Seven miners excavated the top-level compartments with pneumatic clay spades, and two Mecalac 360° excavators types 10MSX and 12MSX, excavated the middle and lower compartments. A dedicated Mecalac excavator, type 14 MXT, equipped with a 30kW Webster Transverse Cutting Unit[**10**] was used to excavate an unexpected boulder bed encountered across the box invert gently dipping towards the reception structure. Excavated material was loaded by the face excavators into dumpers for stockpiling on the jacking base, or uplifted by a Manitou 732 wheeled loader shovel and fed into a discharge conveyor system.

Top and bottom anti-drag systems were used comprising 13mm diameter ropes. The top ropes were spaced at 26mm centres across the box roof and the bottom ropes were laid touching in two distinct rope tracks, each 3.5m wide. Friction loading developed in the top anti-drag system was transmitted into the rear of the jacking base using a compensation jacking system.

The contract documents stipulated the following settlement criteria for the carriageway surfaces based on a green zone, amber-1 and amber-2 trigger levels and a red zone:

- The amber-1 trigger level was set at a surface settlement / heave of 40mm at any point, and/or a variation in longitudinal grade of 0.35%, and/or a change of cross fall of 0.5% across any traffic lane or hard shoulder. Once this level had been reached measures were required to prevent the amber-2 trigger levels being reached, these may include resurfacing.
- The amber-2 trigger level was set at 60mm settlement/heave, and/or a variation in longitudinal grade of 0.5% and/or a change of cross fall of 1.0% across any traffic lane or hard shoulder. Reaching the amber-2 trigger level would result in a cessation of construction works and emergency remedial measures to the motorway.
- Exceeding the amber-2 trigger level and entering the red zone required a cessation of all operations, closure of those sections of the motorway affected, and resurfacing.

During the tunnelling operation it was anticipated that a shallow trough-shaped settlement profile would slowly develop centred on the box alignment. This profile featured a maximum calculated settlement of 60mm, which exceeded the amber-2 trigger level for variation in longitudinal grade by approximately 100%. This problem was solved by laying at night, and in advance of tunnelling, a 25mm maximum thickness “hump” on both carriageways of the motorway to mirror the surface settlement profile allowed by the amber-2 trigger level. Provision was made for additional night-time resurfacing during box installation should the need arise.

Two direct-reading reflex geodometers, mounted on towers, one each side of the motorway, continuously monitored settlement points on a 5m square grid established across both carriageways covering the predicted settlement zone. Results were presented in three-dimensional and graphical format on a real time basis

Performance achieved

Box installation through the jacking pit headwall, under the motorway and into the reception pit took 4 weeks working 24 hours per day, 7 days per week. Maximum progress was 2.7m in 24 hours and the maximum jacking thrust used was 4,200 tonnes. Figure 23 shows the shield partly exposed in the reception structure on the completion of tunnelling.

Face excavation and box advance were carefully controlled in response to the settlement monitoring results. This together with the use of top and bottom anti-drag systems resulted in a maximum recorded settlement of just 26mm. As a result additional surfacing was not required. Figure 1 shows the underpass in use.

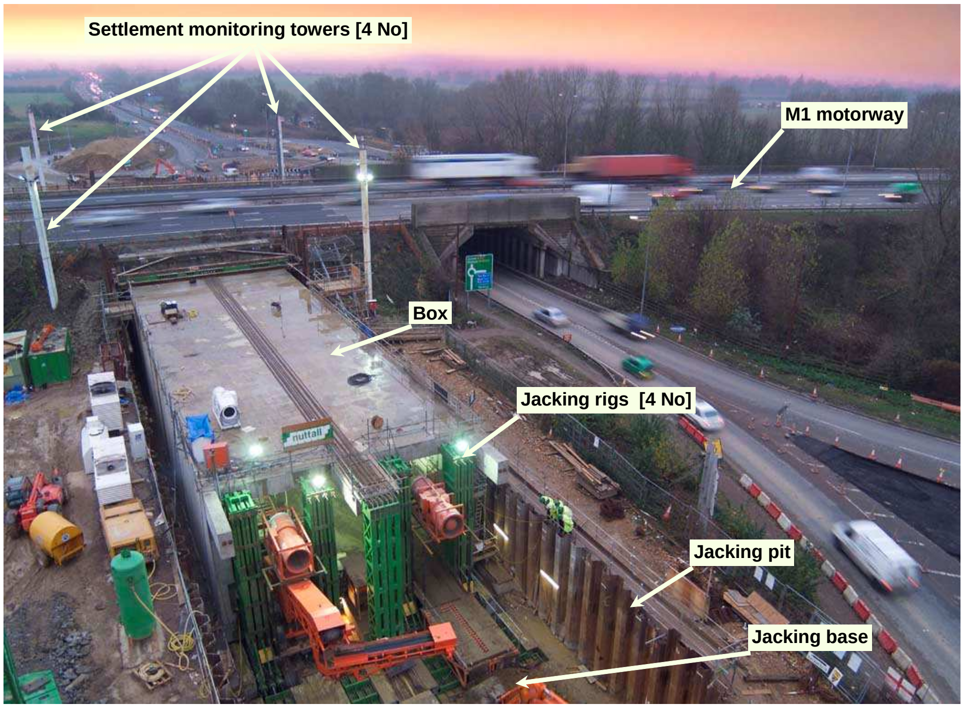


Figure 21: Box partially installed

Figure 22: Rear view of the cellular reinforced concrete shield

Figure 23: Shield exposed on completion of tunnelling

7. SELECTING, DEVELOPING AND EXECUTING A PROJECT USING THE ROPKINS SYSTEM™

The jacked box tunnelling technique described above is a specialised field of work based on a proprietary system for controlling ground drag. It competes with other forms of construction which may be more familiar to engineers in general but which result in greater disruption of overlying infrastructure. Selecting, developing and executing a Ropkins System™ solution requires a client to engage the services of a specialist engineer and a specialist contractor experienced in the Ropkins System™.

7.1 Selection

The selection of a Ropkins System™ solution is likely to be the result of a comparative study of alternative solutions carried out by a client, or by a consulting engineer acting on his behalf. To facilitate the study the client, or his consultant, will engage the services of the specialist engineer to advise on feasibility and budget cost. Ideally the specialist engineer will visit the site with the client to fully understand and appreciate the client's requirements. The specialist engineer will then examine and assess all ground, archive, infrastructure and topographical information available and based on his highly specialised knowledge of geology, soil mechanics, structures, tunnelling, etc make a judgement as to whether the Ropkins System™ is feasible and the manner in which it should be undertaken.

His assessment will typically consider:

- What is the box application, its dimensions and ground cover.
- What are the strength and stability characteristics of the ground.
- Where is the ground water.
- Site access routes and the most suitable position for the jacking base.
- Topography and infrastructure, including condition surveys.
- Previous site use, contamination, influence of external factors such as mining, etc.
- Should the strength and stability of the ground be improved by adopting a geotechnical process such as dewatering, grouting or freezing, and how will this affect overlying infrastructure.
- What face support will be required and how will the face be excavated.
- What anti-drag measures will be required and how can their forces be dissipated.
- Anticipated jacking loads and how can they be transmitted through the jacking base.
- What will be the magnitude of the ground movements and their influences on overlying infrastructure.

Clearly, experience prioritises all of the information available, highlights shortfalls and identifies any additional information necessary to develop the project from conception through to completion.

If a Ropkins System™ solution is considered feasible and the budget cost is of interest the client will request development of the solution supported by detailed geotechnical input.

7.2 Development

The specialist engineer requires geotechnical input in order to predict the behaviour of the ground and to design both the tunnelling system and the permanent works. This normally requires the specification of a detailed site investigation, its execution and subsequent interpretation to provide the following information:

- Soil types and their elevations
- The elevation(s) of any ground water table(s)
- Borehole tests to include standard penetration tests, permeability tests and pumping tests.
- In-situ densities
- The nature and extent of any buried obstructions.
- Short-term (undrained) strength parameters for the design of the tunnelling system
- Long-term (drained) strength parameters for the design of the permanent works.

With regard to the tunnelling system the main aspects of design in which geotechnical input is required are as follows:

- Tunnel shield and the method of face excavation
- Soil movements induced by tunnelling
- Estimate of jacking thrust required to advance the shield and box through the ground
- Top and bottom anti-drag systems
- The provision of reaction to the jacking thrust from a stable mass of adjacent ground.

The above information enables the specialist engineer to prepare conceptual drawings, outline method statements and a schedule of quantities. These are then reviewed jointly with the specialist contractor who is then able to prepare an accurate budget for works.

If the scheme proposals and budget meet with the client's approval the next stage is for the client to enter into a contract with the specialist contractor for the detailed design and construction of the works.

7.3 Execution

The specialist contractor with assistance from the specialist engineer will prepare detailed designs, construction drawings, specifications, method statements and programmes for the works. He will obtain all necessary approvals to these documents and proceed to construct the works on site accordingly.

8. SUMMARY

The authors have presented a highly effective and well proven technique which enables engineers to construct underbridges beneath existing infrastructure. Undoubtedly the success of the technique is due to the ability to control ground movements, which is at the heart of the Ropkins System™. Table 1 illustrates the full range of projects constructed using this technique. Most are located in cities and all have avoided disruption and minimised environmental intrusion. Many have been undertaken in very difficult ground conditions necessitating sophisticated ground stabilisation measures, including grouting, ground water lowering and artificial ground freezing.

In each of the projects illustrated in Table 1, clients, in particular railway authorities and more recently highway authorities, have achieved significant cost benefits from minimised disruption to both surface infrastructure and the travelling public.

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