This Standard is obsolescent.
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British Standard

Sewerage

Part 3. Guide to planning and construction of sewers in tunnel

Egouts
Partie 3. Guide pour la planification et la construction des égouts enterrés

Abwasserkanäle und -leitungen
Teil 3. Leitfaden zur Planung und Ausführung von erdverlegten Abwasserkanäle und -leitungen

Foreword

This Part of BS 8005 has been prepared under the direction of the Civil Engineering and Building Structures Standards Policy Committee and is directed at general engineering practitioners who may either be embarking on a career in sewerage or be dealing with a particular aspect for the first time. It is not intended to be exhaustive in any field but sets out to present guidance on basic principles and good practice, indicating where a more detailed and comprehensive study may be made. BS 8005 supersedes and enhances CP 2005 : 1968, which is withdrawn, although some of the material incorporated is a restatement of the earlier text.

BS 8005 gives guidance on the planning, design, construction, operation and maintenance of works to convey sewage, including storm sewage, surface water and trade effluents to a sewage treatment works, tidal waters or other final place of disposal. Recommendations are given for the repair, renovation and replacement of sewers.

Many end users of this British Standard, such as governments, public authorities, sewerage authorities and consultants, issue their own recommendations and specifications for sewerage which BS 8005 is intended to complement rather than replace.

BS 8005 is being published in six separate Parts, as follows.

Part 0.* Introduction and guide to data sources and documentation

Part 1.* Guide to new sewerage construction

Part 2.* Guide to pumping stations and pumping mains

Part 3.* Guide to planning and construction of sewers in tunnel

Part 4.* Guide to design and construction of tidal outfalls

Part 5.† Rehabilitation of sewers

It has been noted that substantial one-part codes and guides take a long time to revise and if they are reviewed at infrequent intervals, they tend to become out of date quickly, especially in a field where technological development is rapid. It is intended therefore to keep a constant watch on new developments and to update BS 8005, Part by Part, as soon as the work can be justified.

BS 8301 sets out recommendations for building drainage and, while it relates generally to smaller pipelines, there is some overlap between it and BS 8005. BS 6297 gives recommendations for the design and installation of small sewage treatment works and cesspools.

Apart from Part 0, which is directed more specifically at the UK sewerage field, BS 8005 is for use both in the UK and, in appropriate circumstances, overseas.

Suggestions for the improvement of any Part of BS 8005 will be welcomed by the Secretary of SEB/43 at British Standards Institution, 2 Park Street, London W1A 2BS.

Compliance with a British Standard does not of itself confer immunity from legal obligations.

* Published.

† In preparation.

Contents

Section one. General

1 Scope

This Part of BS 8005 provides guidance on the planning, design and construction of sewers constructed in tunnel. It includes guidance on tunnel linings and ancillary materials and recommends permissible deviations for internal faces.

NOTE 1. For the purposes of this Part of this standard a sewer constructed in tunnel means a pipe, conduit, or culvert constructed entirely below ground level within an excavation which is not open to the surface except at access shafts.

NOTE 2. The titles of the publications referred to in this Part of this standard are listed on the inside back cover. Attention is particularly drawn to BS 6164.

The titles of standards publications not referred to in this Part of BS 8005 but of interest as dealing with closely associated subjects are listed in appendix A of BS 8005 : Part 1 : 1987.

Other references will be found throughout the text, indicated by bracketed numbers thus [21], and these are listed in the bibliography in appendix A of this Part.

2 Definitions

For the purposes of this Part of BS 8005, the definitions given in BS 4118 and BS 6100 apply, together with the following, some of which enlarge upon those given in the above standards.

2.1 **primary lining.** The lining required to support the ground for the life of the structure.

2.2 **secondary lining.** Either:

(a) a tunnel lining to provide decorative finish, usually combined with waterproofing, or to improve the flow characteristics of a hydraulic tunnel; or

(b) a second structural tunnel lining.

2.3 **preliminary lining.** Lining provided to facilitate application of the primary lining.

2.4 **temporary lining.** A preliminary lining which is not relied upon to survive once the primary lining is constructed.

2.5 **competent ground.** Self-supporting ground which will not deform significantly until the primary lining to the tunnel has been installed.

2.6 **ground support.** Any support required to secure the ground until the primary lining is complete.

2.7 **caulking.** The operation of making a joint watertight to withstand pressure usually by packing with a filling material and ramming it home.

2.8 **shield.** A mobile steel structure, usually cylindrical, to protect miners excavating the tunnel and to enable the tunnel to be erected.

2.9 **de-watering.** The temporary lowering of groundwater using deep wells or well points, depending on the circumstances.

2.10 **heading.** An accommodation tunnel built to accept a smaller service.

3 Abbreviations and symbols

For the purposes of this Part of BS 8005 the abbreviations and symbols given in Part 1 of this standard apply.

Section two. Planning, design, safety and choice of lining

4 Planning

4.1 General

The principal reasons for constructing a sewer in tunnel are as follows.

(a) The sewer has to be at such a depth that other methods of construction are impossible or too costly.

(b) Disturbance of the ground surface and/or disruption of the general environment and traffic are to be minimized.

(c) The route of the sewer need not follow the highway or open ground and consequently a more direct route may be followed. While a tunnel may be within or outside the highway, if tunnelling is used to reduce traffic interference the shafts themselves should, if possible, be sited off the highway.

There are indications that tunnelling may on occasions be economic in urban areas at quite shallow depths. It is suggested that clients' interests would be best served by including the alternatives of trenching and tunnelling in sewer contracts in built-up areas.

4.2 Preliminary location of route

The cost of a tunnel can vary widely depending on ground conditions and, in urban areas, on constraints such as the types of structures, foundations and utilities in the vicinity. The principal objective at the outset should be to establish whether a tunnel scheme is a technically and economically feasible solution to the engineering problem in the prevailing ground and other environmental conditions. The correct choice of route is, therefore, of crucial importance. Particular care needs to be exercised near to mining works, made-up ground, derelict urban areas and rubbish dumps.

A number of feasible tunnel routes may arise from a study of existing records, geological maps and Ordnance Survey sheets. A detailed survey and study of each option is required. There may be little choice in the location of the beginning and end of a tunnel sewer or in intermediate connection points. Apart from this, the route plan should follow the best tunnelling ground available, consistent with the hydraulic design requirements and any legal constraints (see BS 8005 : Part 0). Directness of route may sometimes be a secondary consideration, e.g. if a relief sewer or flood alleviation tunnel can be built at a lower level and allowed to run full during its occasional use (to be emptied by a small pump later), then the level of the sewer should be constructed in the best ground available. Where it is proposed that the sewer pass under a canal, motorway or railway, the responsible authorities should be consulted.

Sewer tunnels may be constructed to curves but, if small radii curves are proposed, consideration should be given to the cost implications.

4.3 Topographical survey

The topographical survey should extend well beyond the zone likely to be influenced by tunnelling and shaft sinking. The most recent Ordnance Survey sheets should be used for this purpose but details should be checked, and amended as necessary on the ground, as change may have occurred since the last revision.

Sub-surface information can be even more crucial and more difficult to obtain. For tunnel design, details of all other tunnels or subways, with their shafts, and of deep basements and deep piled foundations should be obtained. In addition, boreholes for water supply may exist below premises and, if undetected, could compromise the works. Boreholes can be traced through the abstraction licence records of water supply authorities and companies and the borehole records of the National Geosciences Data Centre at the British Geological Survey, Keyworth.

Settlements due to tunnelling can affect structures and services near to the tunnel (see also 10.2.1). Particular care is needed in the case of gas mains, sewers and water mains because of the dangers of fire, explosion and flooding. Such facilities are often in poor condition and consideration may have to be given to replacing and/or re-routing these services before tunnelling commences.

The topographical survey should also identify possible working shaft sites of suitable size and accessibility.

4.4 Site investigation

4.4.1 *Geotechnical uncertainty.* Regardless of the thoroughness of the site investigation, there will always remain a degree of uncertainty in the ground conditions for all tunnelling projects. The uncertainty of these geotechnical factors has an influence on the choice of tunnelling technique and also on the construction cost. A detailed knowledge of the ground conditions is of fundamental importance to the minimization of risk, both technically and financially.

Various tunnelling methods carry different risks and may require different degrees of site investigation. For instance, the standard of geotechnical information required for tunnelling machines is much higher than for drill and blast methods and, while full-face tunnel boring machines may perform better than conventional techniques in uniformly good rock conditions, they can be inflexible and cumbersome in faulted ground. There are well documented examples (see Athorne and Barratt [1] Athorne and Snowdon [2]) where, over a complete tunnel drive, potentially slower but more flexible tunnelling methods have achieved a better outcome than sophisticated mechanical techniques.

4.4.2 *Objectives.* The objectives of the investigation are to provide the following information:

(a) the assessment of the technical and economic merits of alternative schemes;

(b) the selection of the most suitable alternative;

(c) the preparation of an adequate, economic and safe design;

(d) to allow construction costs to be estimated and difficulties and hazards that may arise during construction to be foreseen and provided for.

To allow these objectives to be achieved, the site investigation should establish in three dimensions the geological structure, the succession and character of the strata present on the site, the ground water conditions and

Table 1. Stages of site investigation
Stage 1. Preliminary appreciation of site and ground conditions (desk study)
Examination of the following readily available information to assess feasibility, to select possible routes, to make preliminary estimates of cost and to plan more detailed investigations. (a) Available information including geological and other maps and reports. (b) Geological and engineering enquiries. (c) Air photographs and surface reconnaissance including examination of local geological features and outcrops. (d) Interpretation together with recommendations for the ground investigation.
Stage 2. Ground investigation before construction
Ground investigation can be divided into four sub-stages as follows. (a) *Preliminary ground investigation.* An amount of work, where required, sufficient to confirm the feasibility and to establish the approximate cost of the project, to narrow route options and to aid in the planning of the main ground investigation. It may include selected boreholes or open excavations, perhaps a geophysical investigation and selected tests. (b) *Main ground investigation.* To obtain the information required for the final alignment, design and construction of the tunnel. It will usually include a programme of in situ and laboratory tests. (c) *Other investigations before construction.* For example, trial shafts, trial adits, assessing and, if necessary, recording the condition of structures and services, grouting trials, dewatering trials, rock bolt trials and monitoring of trial sections. (d) *Interpretation and recommendations.* To enable ground investigation during construction to be carried out.
Stage 3. Ground investigation during construction
Observation and investigation, where necessary, continued during the construction phase to confirm and supplement the earlier investigations, as follows. (a) Observations and records of ground conditions and pressure of water or gases should be made during construction. (b) Probing ahead in tunnels. (c) Other investigations and records during construction, e.g. of ground water levels, extra boreholes, observation of ground movement and settlement, grouting trials, rock bolt trials, monitoring of vibrations and monitoring of trial sections. (d) Review and amendment of plans and sections which is a continuous process throughout the work.

the presence of any special hazards. This involves examining existing information, making field investigations, taking samples and carrying out in situ tests, as needed. This includes a programme of laboratory tests to obtain the required information on the properties of the site materials. The investigation should also locate any existing underground works and services.

4.4.3 *Choice of construction method.* The choice of permanent structure and construction method is closely related in tunnelling work. It is, therefore, essential before survey work starts to determine the full range of relevant design and construction parameters for which survey information may be needed. The survey should then establish, as far as is practicable, the nature of the ground in relation to these parameters so that the structure and construction method adopted will be practicable and cost effective. Lack of information leading to the choice of a construction method wrong for the tunnel lining finally selected is a major problem which has to be overcome by the survey work.

4.4.4 *Investigation programme.* An effective site investigation is best achieved by carrying out the work in three broad stages as shown in table 1. Each successive step aims to narrow the choice towards the best line and level for the tunnel route as follows:

(a) to indicate the next steps required in the investigation;

(b) to fill the gaps in existing knowledge of the site;

(c) to confirm or correct earlier predictions.

In practice, each stage may not be sharply divided from that which follows it and either more or fewer stages may be involved depending on the circumstances and magnitude of the works. The programme should be flexible so that it can quickly adapt to changes occasioned by variation in ground conditions from those predicted by the preceding steps of the investigation.

4.4.5 *Methods of site investigation.* The various methods available for carrying out the site investigation are listed in table 1. As well as boreholes, consideration should be given to the use of trial shafts and adits which can provide more positive information than geophysics but which will be more localized and more expensive. It is important to record the condition of nearby buildings, particularly if blasting is to be involved or if settlement damage is possible during tunnel construction.

Boreholes may be required at every shaft and the method of sinking larger shafts will be governed by the nature of the soil at that point. If additional boreholes are required at intermediate points along the tunnel, they should be sunk sufficiently clear of the proposed tunnel line to avoid later interference with the construction, either through ingress of water or by loss of compressed air. It is important to take boreholes well below the proposed tunnel level because changes in design often result in an increase in the depth of the tunnel. All boreholes should be grouted-up after the necessary information has been obtained.

Stable water levels should be determined in permeable strata, if necessary, by sealing the boreholes to that level. It can be useful to keep representative samples of the various strata encountered. In stiff clay and rock, as large samples as possible should be obtained in order that fissures or jointed structure may be examined. Granular samples should retain their finer components. Continuity of data should be sought by comparing adjoining boreholes, e.g. claystones in London clay often follow a continuous horizon which may be traceable.

Following the main borehole investigation, it may be worthwhile considering using geophysical methods, such as seismic refraction, along all or part of the proposed tunnel line. These techniques may be able to show the form and approximate depth of, for example, soil over bedrock and may serve to locate soil-filled depressions or buried valleys in the rock and help identify areas of difficulty. These can then be investigated by further boreholes.

4.4.6 *Use of data.* The data obtained from the desk studies, boreholes and geophysics, where appropriate, should be used to produce geological sections along the proposed tunnel route. The cross sections produced should be geologically credible and it is prudent to undertake this work with the help of an engineering geologist.

The guidance given here is generally related to tunnelling for sewers but further details on site investigation in general are given in BS 5930, and for tunnels in particular by West, Carter, Dumbleton and Lake [3].

5 Design

5.1 Selection of tunnelling method

5.1.1 *General.* Tunnelling consists of the following two basic and interactive processes:

(a) the excavation and removal of the ground; and

(b) the maintenance of ground stability, including the provision of tunnel support.

At its simplest, excavation can be carried out by hand using spades and pneumatic tools in soft ground and drill and blast methods in hard ground. Alternatively, full-face and partial-face tunnel boring machines can be used where the tunnelling works are of sufficient magnitude to justify the use of such plant. Where uniform ground conditions are predicted, it is possible to tailor the construction method to suit thereby achieving rapid advance rates and economic lining design. Considerable economies have been achieved in this manner in proven ground or where very thorough site investigation has been performed. However, the choice of an inflexible tunnelling method may result in delays to the construction programme and an overall increase in expense in the event that adverse unexpected ground conditions are encountered. It is also possible that the safety margin would be diminished.

Many tunnel drives can be expected to pass through a range of ground conditions, often with both hard and soft ground at the tunnel face simultaneously. Tunnelling in such conditions is at least awkward and at worst it can be extremely difficult. There is therefore merit, where possible, in selecting an alignment for the tunnel which either eliminates or minimizes the extent of such mixed face conditions (see 10.2.4 and 10.3).

5.1.2 *Ground conditions.* (See also BS 5930.) Soils likely to be encountered during tunnelling are as follows.

(a) *Non-cohesive.* Sands, gravels, ballasts and silts. The standard penetration test (see BS 1377) can be used to indicate the relative density of sands and gravels (see 41.2.5 of BS 5930 : 1981).

(b) *Cohesive.* Clays and marls with up to 20 % gravel and chalk having a saturation moisture content of 20 % or greater. Where the moisture content is less than the plastic limit the soil would be described as stiff; soils wetter than the plastic limit would be described as very soft, soft or firm, depending on their undrained shear strength (see 41.2.5 of BS 5930 : 1981).

(c) *Mixed.* Sand and clay, sand and silt, non-saturated clays and silts.

(d) *Fills.* Mixed materials of recent deposit such as domestic refuse containing ash, vegetable matter, etc., and materials of varying content and compaction used for embankments for railways, roads, etc.

(e) *Rock.* Hard material of geological origin or other comparable hard inert material such as concrete, brickwork, etc. It also includes chalk with a saturation moisture content of less than 20 %. Both compressive strength and discontinuities are important determinants of rockmass behaviour and their use is described in 44.2.6, 44.2.7 and 44.3.3 of BS 5930 : 1981.

Comprehensive information on the description and classification of soils and rocks is given in section eight of BS 5930 : 1981.

In common tunnelling usage, ground consisting of items (a), (b), (c) and (d) would be described as soft ground while rock as described in item (e) would constitute hard ground. A mixed face is one where soils and rock each occupy part of the tunnel face.

To assess the method of excavation and lining of a tunnel through the soils described in items (a) to (e), the following conditions have to be considered.

(1) *Ground stability.* Stability at and above the tunnel working face is the first consideration in the building of a tunnel during the construction phase when a variety

of support either temporary or permanent has to be applied. In the long term, the tunnel lining has to be capable of sustaining the loads applied to it. The ability of the ground to be self-supporting even over quite short periods, i.e. its stand-up time, is therefore important in the choice of tunnelling method. (For cohesive soils see also appendix B regarding stability number.)

(2) *Presence of water and level of permanent water table.* Ground and groundwater conditions are a dominant engineering consideration in the choice of tunnelling method. Thus the tunnelling system chosen has to be capable of dealing with the ground conditions identified during the ground investigation. Particular importance should be attached to any information on the conditions experienced in previously constructed tunnels in the vicinity.

5.2 Structural design

The structural design of a sewer in tunnel differs from the design of a loadbearing pipe because allowance should be made for any flexibility in the lining both longitudinally and, transversely, the stresses caused during construction may be more critical than those occurring in the completed sewer.

In all areas where buildings, railways, roads, services and other structures lie adjacent to the line of the sewer, discussion should be held with their owners and the method of construction chosen should be such as to limit the ground movement to an acceptable amount. In the case of railways, roads and services it is usually a statutory requirement that notice be given of the proposed sewer. It is advisable and it may also be a statutory requirement to give notice to owners of private buildings. In extreme cases it may be necessary to underpin buildings overlying the area which may be affected by the tunnel (see Morgan [4]).

The types of tunnel lining available are described in section three and advice on their use as tunnel support is given in 10.2, 10.3, 10.4 and 10.5. In the UK it is seldom necessary to deviate from a conventional lining and to have to produce an original lining design from first principles. Nevertheless a check should be carried out to ensure that predicted stresses and deformations lie within acceptable limits.

5.3 Hydraulic design

The general principles of hydraulic design which should be followed for pipe sewers should also be followed for sewers in tunnel. However the economic size for construction purposes may be larger than that necessary for carrying the flow. This may result in low velocities and the deposition of solids. In these circumstances the invert of the sewer should be shaped to create the required velocity at low flow.

6 Safety

Reference should be made to clause 17 of BS 8005 : Part 1 : 1987 which outlines the legislative framework with regard to the health, safety and welfare of persons engaged on the planning, design and construction of sewers in tunnels. Detailed advice on safety in this field is contained in BS 6164. Particular attention is drawn to the fact that the safety of tunnelling works can depend critically on the adequacy of the preliminary ground investigation and the proper interpretation of the results obtained (see 4.4). Ground investigation work should be as comprehensive as possible and there should be the fullest disclosure of all information, or of all gaps in information, both factual and interpretive, to those responsible for the design and for the construction of the tunnel (see also 7.3 of BS 8005 : Part 1 : 1987 and BS 5930).

The constructors of the tunnel should report to the designers the occurrence of any unforeseen ground conditions or the presence of water or gases which could influence the safety of the permanent works. There is a particular need to consider and guard against the possibility of methane gas infiltration from geological or mining sources below or near the proposed line of any sewer tunnel.

Additional safety hazards may arise where a new sewer in tunnel is to be connected with, or constructed near to, an existing sewer system or when there are existing tunnels, basements or other services nearby.

7 Preformed primary tunnel linings

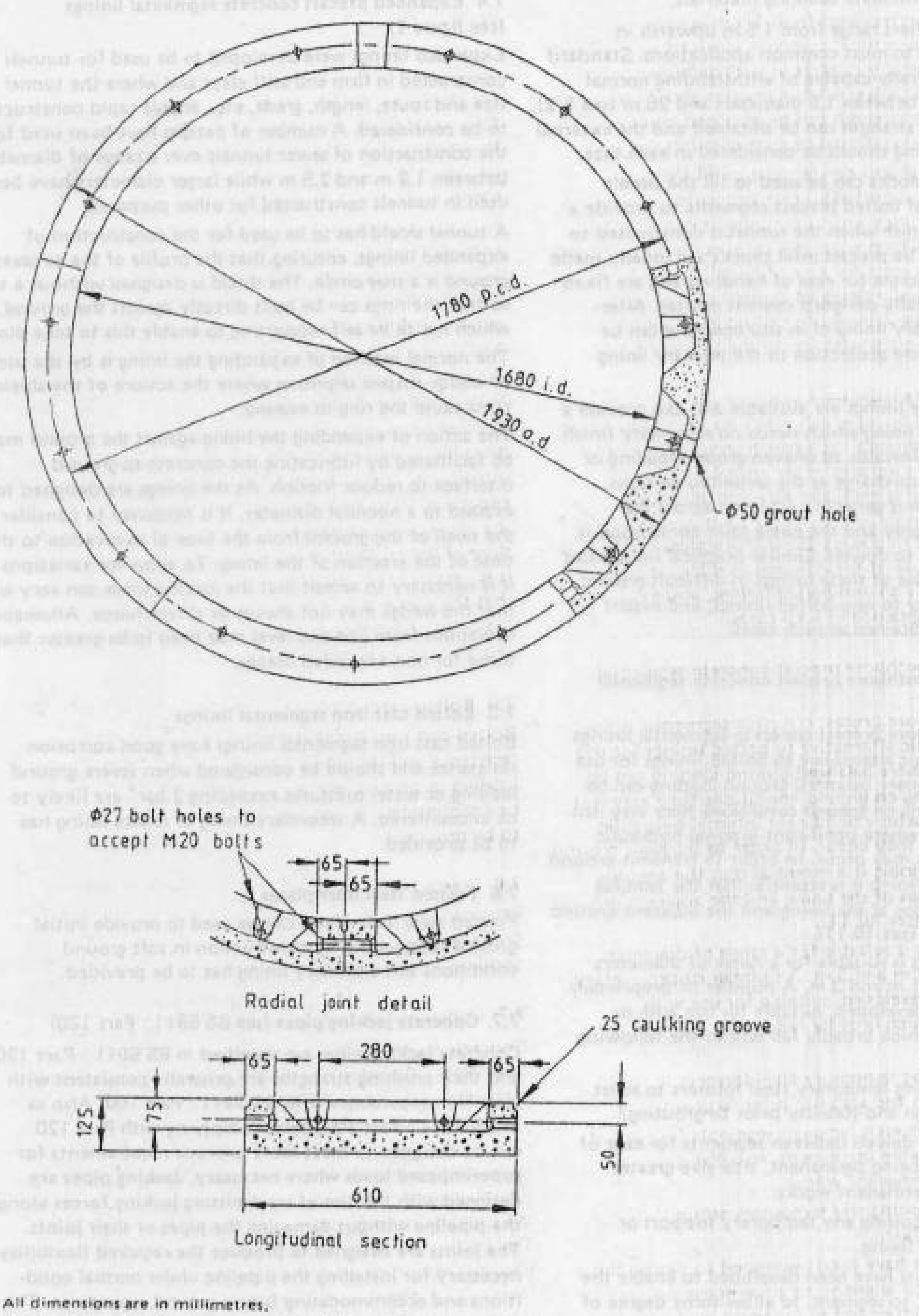
7.1 General

There are at present no British Standards for segmented tunnel lining components. These components are predominantly manufactured in concrete but other materials may be available. For precast concrete tunnel segments reference should be made to the Water Authorities Association's Civil Engineering Specification for the Water Industry [5].

A degree of protection against corrosive attack can be achieved for all precast concrete linings. The concrete cover to reinforcement should be carefully considered and designed, as appropriate to the severity of corrosive attack. The use of sulphate resisting cements in accordance with the recommendations of BRE Digest 250 [6] is satisfactory in most cases (see BS 8110). In severe cases, the use of applied bitumastic or epoxy coatings or the use of integrally cast protective systems will provide improved protection (see 5.17.1 and 5.17.2 of BS 8005 : Part 1 : 1987 and 9.4 of this Part).

7.2 Bolted precast concrete segmental linings (see figure 1)

Bolted precast concrete segmental linings are steel reinforced to accommodate temporary construction and permanent loads. They are formed of segments erected and bolted together, forming circular rings, each with a narrow key segment at the soffit. They can be erected with or without a shield, depending upon ground conditions. Once erected they are self-supporting, requiring no temporary supporting framework. In some circumstances rolling of the key may assist in maintaining the shape of the ring. Suitable packing materials can be inserted in the joints, and grummets are placed under the washers at the head and nut of each bolt. The joints between the segments are



All dimensions are in millimetres.

NOTE. This is a typical arrangement and dimensions are shown only for guidance

Figure 1. Details and dimensions of typical bolted precast concrete segmental lining

designed to accommodate caulking materials.

Production diameters range from 1.5 m upwards in increments suited to most common applications. Standard segments are generally capable of withstanding normal loading at depths between 1.5 diameters and 25 m (see 5.2). Linings of greater strength can be obtained and the external loading on the lining should be considered in each case.

Precast concrete blocks can be used to fill the panels between the ribs of bolted precast segments to provide a smooth internal finish when the tunnel is constructed to close tolerances. The precast infill blocks are usually made of lightweight concrete for ease of handling and are fixed in place with specially designed cement mortar. Alternatively, a secondary lining of in situ concrete can be provided giving more protection to the primary lining (see 10.5).

Bolted smoothbore linings are available and can provide a primary structural lining which needs no secondary finish. They are not as vulnerable to uneven ground loading or internal hydraulic surcharge as the unbolted versions. Positive connection is generally achieved across the longitudinal joint only and the circle joint connection is generally confined to dowels. Similar practical limitations may apply to the use of these linings in difficult ground conditions, as apply to non-bolted linings, and expert advice should be obtained in such cases.

7.3 Unbolted smoothbore precast concrete segmental linings

Unbolted smoothbore precast concrete segmental linings provide an economic alternative to bolted linings for use in circumstances where balanced ground loading can be achieved. Depending on ground conditions they may not be suitable for use where significant internal hydraulic surcharge pressures may occur. In order to transmit ground loadings on to the lining it is essential that the annulus between the extrados of the lining and the adjacent ground is filled with grout (see 10.11).

The unbolted lining is suitable for a range of diameters generally between 1 m and 3 m. A number of proprietary designs have been developed, suitable for use with or without a shield, which broadly fall within the following categories:

(a) those requiring temporary steel formers to assist with ring erection and stability prior to grouting;

(b) those having dowels between segments for ease of erection which, being permanent, also give greater rigidity to the permanent works;

(c) those not requiring any temporary support or permanent joint fixing.

Various shaped joints have been developed to enable the location of segment to segment, to allow some degree of flexibility in the structure and to facilitate load transfer. A caulking groove is normally incorporated and in some systems the joint profile is designed to accommodate a bituminous sealing strip.

7.4 Expanded precast concrete segmental linings (see figure 2)

Expanded linings were developed to be used for tunnels constructed in firm and stiff clays and where the tunnel size and route, length, grade, etc., enable rapid construction to be considered. A number of designs have been used for the construction of sewer tunnels over a range of diameters between 1.2 m and 2.5 m while larger diameters have been used in tunnels constructed for other purposes.

A tunnel shield has to be used for the construction of expanded linings, ensuring that the profile of the excavated ground is a true circle. The shield is designed without a tail so that the rings can be built directly against the ground, which has to be self-supporting to enable this to take place.

The normal method of expanding the lining is by the use of wedge-shaped segments where the actions of the shield rams cause the ring to expand.

The action of expanding the lining against the ground may be facilitated by lubricating the concrete-to-ground interface to reduce friction. As the linings are designed to expand to a nominal diameter, it is necessary to consider the swell of the ground from the time of excavation to the time of the erection of the lining. To allow for variations it is necessary to accept that the jacking space can vary and that the wedge may not always be driven home. Allowable deviations from line and level may need to be greater than those for non-expanded linings.

7.5 Bolted cast iron segmental linings

Bolted cast iron segmental linings have good corrosion resistance and should be considered when severe ground loading or water pressures exceeding 2 bar* are likely to be encountered. A secondary lining of in situ lining has to be provided.

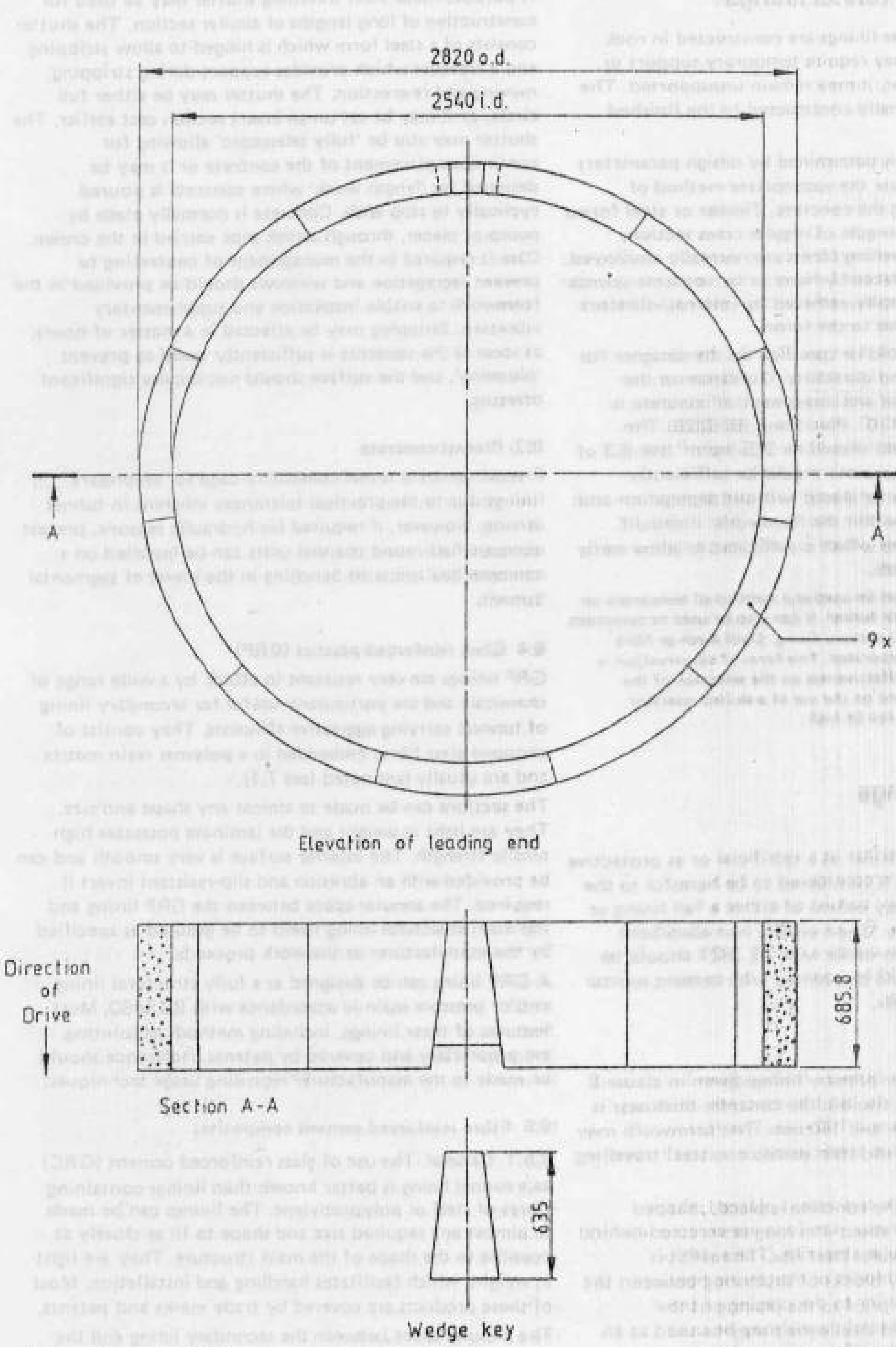
7.6 Pressed steel liner plates

Pressed steel liner plates can be used to provide initial ground support during construction in soft ground conditions but a primary lining has to be provided.

7.7 Concrete jacking pipes (see BS 5911 : Part 120)

Concrete jacking pipes are specified in BS 5911 : Part 120 and their crushing strengths are generally consistent with class H in accordance with BS 5911 : Part 100. Also as in BS 5911 : Part 100, pipes complying with Part 120 can be designed to meet more onerous requirements for superimposed loads where necessary. Jacking pipes are designed with the aim of transmitting jacking forces along the pipeline without damaging the pipes or their joints. The joints are designed to produce the required flexibility necessary for installing the pipeline under normal conditions and accommodating future ground movement. The joint also provides a high degree of watertightness. Where it is safe and practicable to do so, any minor leaks may be sealed by caulking or other approved methods. Pipes in accordance with BS 5911 : Part 120 are produced with a range of lengths, diameters and joint profiles. Special pipes are normally produced for use behind a jacking shield and at intermediate jacking stations.

* 1 bar = 10^5 N/m^2 = 100 kPa



All dimensions are in millimetres.

NOTE. This is a typical arrangement and dimensions are shown only for guidance.

Figure 2. Details and dimensions of typical expanded precast concrete segmental lining

8 In situ primary tunnel linings

In situ primary concrete linings are constructed in rock tunnels. The tunnels may require temporary support or, if the rock is competent, it may remain unsupported. The in situ concrete is normally constructed to the finished internal profile.

The finished thickness is determined by design parameters and also to accommodate the appropriate method of placing and compacting the concrete. Timber or steel forms are used and, for long lengths of regular cross section, purpose-made steel travelling forms are normally employed. The concrete may be placed by hand or by concrete pumps and compaction is normally achieved by internal vibrators and by vibrators attached to the forms.

The concrete grade should be specified by the designer for the required strength and durability. Guidance on the specification, production and assessment of concrete is given in BS 8007, BS 8110 : Part 1 and BS 5328. The minimum cement content should be 325 kg/m^3 (see 6.3 of BS 8007 : 1987). The concrete should be sufficiently workable to enable it to be placed without segregation and to be fully compacted within the formwork: it should develop strength at a rate which is sufficient to allow early stripping of the formwork.

NOTE. Sprayed concrete can be used as a method of temporary or permanent support to a rock tunnel. It can also be used to construct the finished profile within a primary lining. Steel mesh or fibre reinforcement may be incorporated. This form of construction is broadly dependent for its effectiveness on the selection of the appropriate concrete mix and on the use of a skilled operator, otherwise material wastage can be high.

9 Secondary linings

9.1 Brickwork

Brickwork may be used either as a sacrificial or as protective lining, where the sewage is considered to be harmful to the permanent lining, and may consist of either a full lining or a lining to the invert only. Good quality non-absorbent engineering bricks in accordance with BS 3921 should be used, and the joints should be pointed with cement mortar or a suitable epoxy mortar.

9.2 Concrete in situ

The recommendations for primary linings given in clause 8 also apply to concrete in situ but the concrete thickness is normally between 75 mm and 150 mm. The formwork may be ribs and laggings, ribs and steel panels or a steel travelling shutter.

With a static shutter, as the concrete is placed, shaped timber laggings normally about 4 m long are erected behind and are supported by circular steel ribs. The soffit is completed by placing key blocks of shuttering between the top laggings immediately prior to the placing of the concrete. However, curved steel panels may be used as an alternative to timber laggings. The concrete is normally placed either by hand or with the assistance of a conveyor or a small pump. Internal vibrators may be used to shoulder level but thorough hand ramming is necessary in the crown. The formwork should be stripped early enough to allow the fresh concrete to be given a smooth finish.

A purpose-made steel travelling shutter may be used for construction of long lengths of similar section. The shutter consists of a steel form which is hinged to allow stripping and a traveller which provides support during stripping, moving and re-erection. The shutter may be either full circle, or it may be set on an invert section cast earlier. The shutter may also be 'fully telescopic' allowing for continuous placement of the concrete or it may be designed for 'length work' where concrete is poured cyclically to stop ends. Concrete is normally place by pump or placer, through a slick pipe carried in the crown. Care is required in the management of concreting to prevent segregation and windows should be provided in the formwork to enable inspection and supplementary vibration. Stripping may be effected in a matter of hours, as soon as the concrete is sufficiently cured to prevent 'plucking', and the surface should not require significant dressing.

9.3 Precast concrete

Precast concrete is not commonly used for secondary linings due to the practical tolerances inherent in tunnel driving. However, if required for hydraulic reasons, precast concrete half-round channel units can be installed on a concrete bed and with benching in the invert of segmental tunnels.

9.4 Glass reinforced plastics (GRP)

GRP linings are very resistant to attack by a wide range of chemicals and are particularly useful for secondary lining of tunnels carrying aggressive effluents. They consist of chopped glass fibres embedded in a polymer resin matrix and are usually laminated (see 7.1).

The sections can be made to almost any shape and size. They are light in weight and the laminate possesses high tensile strength. The interior surface is very smooth and can be provided with an abrasion and slip-resistant invert if required. The annular space between the GRP lining and the main structural lining needs to be grouted as specified by the manufacturer as the work proceeds.

A GRP lining can be designed as a fully structural lining and/or pressure main in accordance with BS 5480. Most features of these linings, including methods of jointing, are proprietary and covered by patents. Reference should be made to the manufacturer regarding usage techniques.

9.5 Fibre reinforced cement composites

9.5.1 *General.* The use of glass reinforced cement (GRC) as a tunnel lining is better known than linings containing fibres of steel or polypropylene. The linings can be made to almost any required size and shape to fit as closely as possible to the shape of the main structure. They are light in weight, which facilitates handling and installation. Most of these products are covered by trade marks and patents.

The annular space between the secondary lining and the primary (structural) lining should be pressure grouted as the work proceeds. The type of grout depends on the circumstances of each case and is likely to range from a simple cement/water mixture through cement/sand/PFA or bentonite mixes to sophisticated chemical grouts designed for special conditions.

9.5.2 *Glass fibres.* In the case of GRC panels reference should be made to BRE Digest 33 [7].

Apart from the vulnerability of the Portland cement matrix to attack by solutions of acids and high concentrations of sulphate (both of which are very seldom met in normal town sewage), there is still the question of the long-term, 50 years and over, durability of the special alkali-resistant glass fibres.

9.5.3 *Steel fibres.* Steel fibres can now be obtained as either ordinary steel or stainless steel. As these are assumed to be (and in fact should be) uniformly dispersed throughout the matrix, their durability is unlikely to be adversely affected unless either the cement matrix is chemically attacked or physical damage occurs to the lining.

9.5.4 *Polypropylene fibres.* Polypropylene is inert in most conditions which are likely to be met in public sewers.

9.6 Other materials

Purpose-made linings of plastics have been used to reline or to renew the inverts of sewers. These materials are convenient to use where the surface of the sewer has deteriorated, as they provide a wear-resistant surface. They are usually built inside the sewer by bolting them together and to the fabric of the sewer. The annular space is then grouted under pressure. For man-entry sewers it is important that the surface of the invert is slip-resistant to men wearing steel-studded boots.

As a cheaper alternative to a brick lining, the invert of a tunnel sewer may be lined with flat tiles or channels.

In hotter climates overseas where hydrogen sulphide or other corrosive gases may occur in sewers, PVC has been used to line sewers above the level of the wetted portion.

Section three. Construction

10 Construction techniques

10.1 Ground treatment

10.1.1 *General.* Ground treatment may be used for one or other of the following purposes.

(a) *Groundwater control*, in order to:

(1) reduce ingress of water to manageable quantities;

(2) enable tunnels to be driven without compressed air or at reduced pressures;

(3) limit compressed air losses in ground which might otherwise be too open for the method to be viable.

(b) *Soil stabilization*, in order to:

(1) bring the soil to a condition so that the faces of the excavations are able to remain unsupported over limited spans whilst temporary or permanent supports are provided;

(2) prevent loss of fine material carried by inflow of water which could give rise to face instability or surface settlement.

Soil is most frequently treated by one of the following techniques:

(i) the injection of grouts or chemicals;

(ii) freezing;

(iii) other specialist techniques, e.g. displacement by cement based pastes and mixing in situ with grouts may also be used. Dewatering, using wells or well points may be used either on their own or in combination with grouting treatments to reduce groundwater pressure.

Advice should always be sought from experts in both ground treatment and tunnelling before attempting ground treatment. In selecting the appropriate method it is advisable at an early stage of design to give detailed consideration to the techniques of treatment and construction and their influence on programme and cost. It may also be appropriate to select a specialist contractor to carry out the ground treatment work so that the detailed techniques of treatment and construction and their influence on programme can be agreed.

10.1.2 *Ground information requirements for ground treatment.* For tunnelling in unstable ground, treatments are, for economic reasons, necessarily confined to small volumes of ground local to the tunnel. Thus, to predetermine the likely effects of various treatments, it is very important to obtain foreknowledge of the ground characteristics within the zones actually to be treated. The influence of fines content on the cohesion and permeability of potentially unstable soils is very significant and unfortunately varies rapidly and sometimes significantly within a stratum, which can make the difference between success and failure of a specific technique. Hence remote information can be misleading, e.g. the micro-fabric may be strongly anisotropic which will have a major influence on fluid flow through pore spaces. Fabric detail can have significant influence on the stability of a tunnel face depending upon relative orientation and scale.

Site investigation has already been considered (see 4.4). Where ground treatment is envisaged, studies to determine such properties as in situ strength parameters, both drained and undrained, in situ state of compaction (density), permeability and porosity and fabric description should also be considered. For grouting treatments, in situ permeability should be determined by methods simulating grout injection to obtain information on local variations. Here pumping tests from wells are inappropriate since they reflect only the permeability of the most open lenses and are thus applicable primarily to freezing and ground water control by pumping. Preliminary guidance on the likely range of permeability of common soil and rock types is given in Cripps, Bell and Culshaw [8] and should be checked for specific sites.

For preliminary assessments of appropriate stabilization techniques, the most useful types of information, to support accurate and detailed strata logs, are particle size distribution and index tests. Both are relatively easily and economically obtained. Their potential shortcoming is the small size of sample and destruction of fabric by mixing of constituent particles in disturbed samples, possibly with loss of fines, when samples are extracted from below the water table. It is important to grade the whole sample, including the finer particles, since it is these which dominate those properties such as cohesion and permeability which determine the treatments that may be appropriate. Strata fabric may be ascertained at depth in soils by continuous samplers of either the Delft type or by the use of consecutive open drive or piston tube samplers. These samplers are returned to the laboratory where they are split open, photographed and descriptions are made of the core fabric. These descriptions are extremely important where mixed soil conditions are encountered, particularly in the presence of thin bands of silt.

Figure 3 gives guidance on the usual ranges of possible treatments in relation to the fine end of the soil grading, i.e. the D_{10}* of the soil. The coarser fractions of the soil have little influence except with respect to ease of penetration by boreholes and drilling tools.

Good information on ground conditions and properties helps to reduce the risk of cost and time over-runs whilst facilitating better engineering decisions.

10.1.3 *De-watering.* De-watering is normally effective in sands and gravels which are free from clay or silt but may be extended into the silty range of materials by special measures.

Deep wells are more usually installed when pumping from depths of more than 6 m or when a discrete aquifer, such as a buried channel, is the source of water. The filter zone is set within the aquifer and a submersible pump is set in a sump within the casing below the filter zone. In finer materials drainage may be assisted by the application of a vacuum to the casing.

Well points are normally limited to a depth of 6 m by the capacity of the headermain vacuum to lift water from the well point filter level. In finer materials the following two techniques are available:

(a) the system may be installed some time (up to three weeks) before excavation to allow long term drainage;

* The sieve size which 10 % of the soil passes.

(b) a higher effective vacuum may be applied at the well point filter by using an aductor system*.

Settlement problems may arise, depending on ground conditions, if fines are pumped from soils of heterogeneous grading thereby inducing piping. Care should always be taken that no fines are extracted by the choice of filters appropriate to the soil surrounding the wells.

The lowering of the water table may also induce settlement by increasing effective stress even in sands. Consolidation effects are particularly important on normally consolidated and weak organic soils.

10.1.4 *Injection of grout and chemicals.* Injection of liquids of appropriate rheological properties can reduce permeability and improve the strength of cohesionless soils. Natural arching around tunnels also helps if the soil can be made dense enough to exploit it.

When tunnelling in ground which is naturally stable but is subject to water inflow, it is necessary only to reduce water inflow to manageable quantities.

An annular treatment some distance outside the tunnel excavation can help by providing a water barrier to reduce exit gradients at the tunnel excavation. In mixed ground conditions this barrier need not be continuous as it may be sufficient to treat only those coarser lenses which carry the water. This is comparatively easy to achieve but care has to be taken to ensure that filling course lenses or joints will not de-stabilize untreated zones of finer porosity by increasing the hydraulic gradients through them.

Instability of the ground may arise either from inward flows of groundwater or because of insufficient inherent strength in the soil. Strength can be improved either by uniform permeation to give particle to particle bonding or, sometimes at depth, large pressures can be applied safely to compress and consolidate the ground. This technique is applicable in soils of silt size, finely fragmented rock or similar materials. Improving the strength of a soil uniformly by injection is comparatively difficult and requires careful matching of grouting technique to ground type.

Cement grout particles are too large to pass between the pore spaces of sands, or gravels containing sand, without disturbing the soil structure. It is only possible to inject cement or bentonite grouts into such materials by fracturing them and so forming fissures. When the grouts are thin and water-like these fissures are usually a few millimetres wide and may extend tens of metres from the point of injection in directions controlled by the state of stress existing in the ground. Particles are not bonded in this process and the fissures may intersect contained lenses of coarse open gravel which are then permeated by the grout. Furthermore, fissures of grout often extend wastefully beyond the restricted zone of treatment normally required around tunnels.

Thickening cementitious pastes, however, limits extensive travel in fissures and can inhibit formation of fissures when stiffened to the consistency of mortar. Such grouts can be used to compress unconsolidated ground to stiffen it. Such grouts are also used behind tunnel linings (see 10.11).

At shallow depths for particle bonding and water stopping, permeation injections are essential and grouts have to be carefully chosen to be capable of permeating the finer soil types. Soils containing sand require a particle-free liquid chemical grout. These set to weak gels in controllable time intervals of about 10 min to 90 min according to proportions of their constituents and ground temperature.

The cost of injection usually increases considerably according to the fineness of the soil, as closely spaced injection holes (1 m to 1.5 m) are needed for contiguous overlapping treatments. Since the range of injection from a single pipe is limited both by low permeability and maximum pressure applicable to avoid fracture and uplift of the ground, the effectiveness of treatment is least if the initial permeability is at the low end of the appropriate range of application.

Many of the materials used for chemical grouting are considered to be unacceptable because of toxicity hazards (see CIRIA Report 95 [9]). The main hazard is due to temporary activities such as spillage of materials to drains and washing down plant during application of treatments rather than after injection and gelation in the ground. As a result, chemicals currently used are restricted mainly to those based on sodium silicate of which there are many variants.

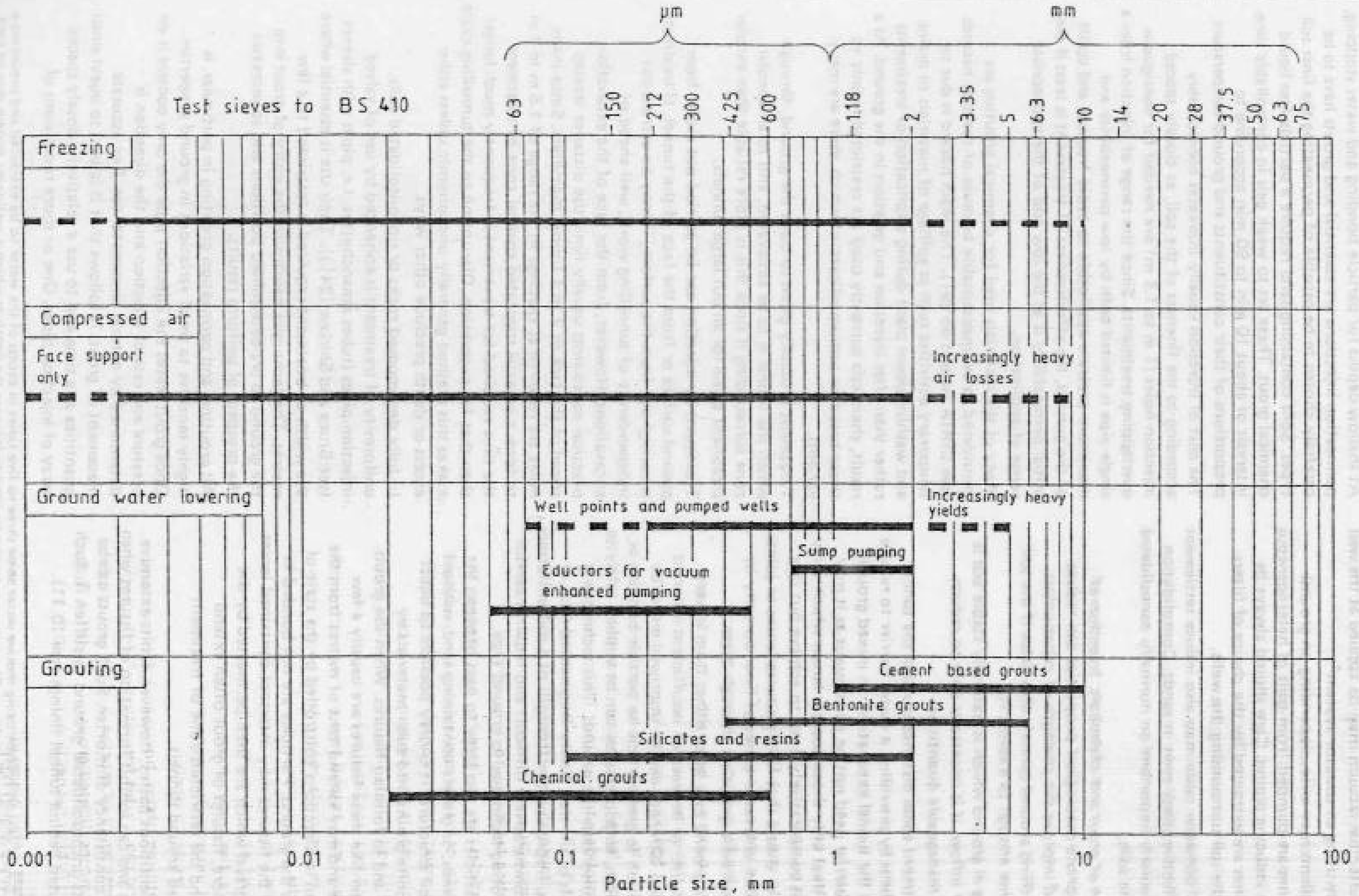
Treatment is usually given to both the ground, through which the tunnel is to be excavated, and to an annular zone surrounding it since this is more reliable than annular treatment alone for all but large tunnels.

Treatment by injection can be carried out either from ground surface or from the face of the tunnel. If treatment is carried out from the surface, it may be carried out independently of tunnelling work, well ahead of excavations. However, from the face of the excavation, practical constraints usually limit the distance treated ahead of the face to 2 to 4 tunnel diameters. Since many holes are required at spacings in the range of 1.5 m to 2 m to form a co-axial truncated conical zone of treatment, the time required for treatment is frequently much longer than that for excavation. Disruption to the tunnelling cycle makes this method generally uneconomic unless either access or depth preclude other ways.

In fully decomposed rocks or unconsolidated soils, uniformity of treatment is enhanced by use of valved injection pipes (tubes à manchettes, i.e. pipes with sleeves (see Bruce and Shirlaw [24])). Their use is essential where the width of treatment required is restricted to a few metres. The technique constrains the entry of grout into the ground to pre-determined positions and so increases the prospect of uniform results.

All grouting, and permeation grouting in particular, is highly sensitive to small variations in ground properties. Once grout leaves the injection pipe the only control is on pressure and rate of injection and the direction is determined by natural circumstances. For accurate placement of grout, it follows that it is best to inject small quantities at a time and to use a relatively closely spaced array of injection pipes. One or more repetitions of

* An eductor is a simple venturi system using motive water some three to five times in excess of the water to be extracted and producing a high level of vacuum at the level of the filter. The system may also be used to increase the depth at which a wellpoint system may be used beyond 6 m.



NOTE. This figure is based on Bruce and Shirlaw [25] and Somerville [26].

Figure 3. Guidance on the range of possible ground treatments related to D_{10} of soil

injection are also essential for formation of continuous blocks of treatment, as initial injections will naturally fill only the most open pore spaces. The quality of the result depends largely on these factors, which are directly related to the cost of the works.

10.1.5 *Ground freezing.* The technique of ground freezing, whereby the water in the ground is frozen, may be used to stabilize running and weak ground and/or limit the entry of water into shaft or tunnel workings during construction. It may be carried out from the surface or from the tunnel face depending upon local conditions.

Freezing is carried out by circulating refrigerated liquids through pipes installed in the zone of ground to be frozen. The liquid is usually recirculated and re-cooled so that the pipes consist of a concentric system with flow down the centre and up the annulus. Sometimes, if the cooled liquid is gaseous at atmospheric pressures, it may simply be evaporated within each freeze-pipe during a single pass.

The range of soils to which freezing may be applied is wide (see figure 3). Its use in the coarser soils, particularly in open gravels, is restricted by considerations of cost as the freezing load may be excessive, with large volumes of contained water to be frozen, and with free movement of water likely. High ground water flow may make the method impracticable in clayey soils. The type of clay minerals and their fineness sometimes inhibit freezing of pore water and occasionally cracking occurs in frozen clay soils, thus opening potential leakage paths and weakening the material. The treatment is best applied in any moist or saturated rock formation or in weak, running, or heterogeneous bouldery ground.

Ground investigation (see 4.4) should establish the soil types to be frozen and the volume of water in voids. The soil need not be fully saturated but this is preferable. Saline waters require freezing at depressed temperatures demanding more energy input. Freezing properties vary with lithology and may require differing freezing loads or freeze periods for specific strata*.

Soils such as silts are prone to ground heave because the balance between capillary suctions and comparatively high permeability is such that water can be drawn into them during freezing. If uncontrolled, this can amount to 5 % to 10 % of their frozen depth, however, control techniques are available which can limit such movement to small amounts. Heave arises from a continuous formation of ice lenses rather than from expansion of water on freezing. On thawing, any extra water drawn into the frozen zone results in softening of the ground and this has to be taken into account when considering the loading on the completed structure. Again the problem is most apparent in silty soils of low plasticity. Thawing may also require several weeks to complete and may hinder subsequent works.

Two methods of freezing are commonly used as follows.

(a) *Brine freezing.* The circulating liquid is usually a calcium chloride brine, cooled to temperatures of about - 25 °C to - 35 °C by means of a refrigeration compressor charged with ammonia or Freon. For most soils at these temperatures hole spacings are usually 1 m to 1.5 m. The freezing period required is about 3 to 12 weeks, according to circumstances, in order for the growing frozen columns surrounding each freeze pipe to coalesce and the resulting 'wall' to gain sufficient thickness.

(b) *Nitrogen freezing.* This method is quick, with freezing periods of 3 to 7 days, arising from freeze-pipe temperatures as low as - 100 °C to - 196 °C. After evaporation in the circuit the resultant gas is exhausted to atmosphere well clear of access shafts and the like. The single-pass non-recirculatory nature of the method is costly and because of this it is usually used either for small-scale works, to deal with emergency circumstances, or ancillary to brine freezing, i.e. for accelerated commencement or boosting at critical points.

A sufficient thickness of frozen ground should be maintained between the face being excavated and any pipes through which liquid nitrogen is being circulated. At all times the atmosphere in the tunnel (or shaft) should be monitored for oxygen deficiency. Appropriate evacuation procedures should be established on-site prior to work starting and all persons in the tunnel (or shaft) should be aware of these procedures.

10.1.6 *Specialist techniques.* Where comparatively small zones of soils have to be stabilized and where there is reasonable accessibility for plant, the cost of establishing freezing in silty ground may be prohibitive and grouting inappropriate. In such cases treatments, which displace natural soil and mix it in situ with cementitious grout or remove the soil and replace it with grout, are possibly applicable. This can be done either with direct mechanical mixing in situ by means of auger-like tools through which grout is injected or by jet grouting, which employs a rotating high pressure jet to erode and fluidize the soil replacing it or mixing it with grout simultaneously injected. These systems are appropriate in soils ranging from silts through to sandy gravels and in some weak clays. Jetting is restricted in strongly cohesive soils since these resist erosion.

Both systems produce columnar masses of treatment. Contiguous columns provide structures of any shape, subject to accessibility of equipment. Multiple thickness of columns may be necessary to reduce the risk of leakages of unstable soil or water at improperly overlapped joints.

Jetting systems recirculate suspensions of soil or grout at pressures of 400 bar to 600 bar. If the recirculation system becomes choked at the borehole these high pressures can be applied directly to the ground, resulting in considerable heave of the ground.

The resulting treatments are comparatively uniform and have the character of stabilized soil cement with unconfined crushing strengths upwards of 1 MN/m^2.

The mechanical mixers can be used in soft clays with high water content for lime stabilization.

* Note too that in tropical countries groundwater and ambient soil temperatures may be much higher than in temperate zones.

Expert specialist advice is necessary before applying these comparatively novel systems.

10.2 Preliminary tunnel support in soft ground

10.2.1 *General.* A shield is commonly used in soft ground to protect the miners excavating the tunnel and to enable the lining to be erected. Exceptions are stable formations such as stiff clay where the ground may be sufficiently strong to remain self-supporting for the time taken to excavate it and erect a ring of lining.

The shield is propelled forward by jacking-off the already erected primary lining and may also be fitted with face jacks and boarding to support the excavation.

Precast concrete linings, as described in clauses 7 to 9, are commonly used to provide both preliminary ground support and the primary structural lining of the tunnel. Bolted and unbolted precast segments are erected within the tail skin at the back of the shield. In special circumstances, shield driven tunnels can be installed to horizontal or vertical curves using normal segments packed as necessary or, for more severe curves, using specially cast tapered segments.

Tunnels located below the water table in cohesionless soils with an undrained shear strength (c_u) (see appendix B) at or close to zero, almost always need either internal support, such as compressed air, or ground treatment to increase the strength of the ground. Above the water table the apparent cohesion deriving from the capillary forces in the water in the voids of such soils is, in many instances, sufficient to provide a stable face. Even when such forces prove inadequate and instability develops the measures needed to maintain support are not great and face boards and/or sand tables are usually sufficient to control the ground.

As has been explained, the introduction of a tunnel disturbs the in situ stresses and inevitably causes movement of the ground. This is usually only a problem with soft ground travelling in built-up areas where buildings can be put at risk and services such as gas and water mains and sewers can be endangered. Guidance on the estimation of settlements caused by tunnelling can be found in Attewell, Yates and Selby [10], O'Reilly [11] and O'Reilly and New [12].

When unstable conditions are likely to arise during tunnelling, the methods of construction have to make provision either for support within the tunnel or increased ground strength by treatment, although occasionally, a combination of both techniques may be adopted. Ground treatment could consist of any of the techniques described in 10.1. Support within the tunnel is normally provided by compressed air but closed-face techniques using either pressurized fluids such as water/bentonite mixtures or earth-pressure balance systems are on the increase.

10.2.2 *Compressed air*

10.2.2.1 *General.* In a wide range of soils varying from weak clays to sands below the groundwater table, tunnel face stability may be improved by applying compressed air. This technique can also be used to reduce water inflow tending to dry the face and thereby making excavation and lining erection easier.

The detailed statutory requirements which apply are set out in Statutory Instrument No. 61 [13]. More recent detailed recommendations are given in CIRIA Report 44 [14] and also in BS 6164.

Although the maximum pressure permitted by Statutory Instrument No. 61 [13] is 3.4 bar, the technique is more commonly used at pressures below 1 bar. At working pressures over 1 bar more stringent conditions apply including the necessity for staged decompression of personnel and provision of medical locks and medical attendants. These factors impose economic penalties but, because sewers are generally located at shallow depth, it is unusual for compressed air above 1 bar to be used for the construction of sewers in tunnel.

Compressed air working requires installation of either shaft or tunnel locks and substantial mechanical plant and standby equipment. Installation usually represents a significant expense and can delay the start of tunnelling unless planned from the outset (see also 10.6.2).

The choice of air pressure for practical use is largely a matter of judgement. Compressed air may be applied to balance the hydrostatic pressure in the invert. This will keep the face dry but the out-of-balance pressure in the crown will tend to increase air loss and the risk of a 'blow', or sudden loss of air pressure, and consequent immediate reduction in face stability. It may be preferable to balance the hydrostatic pressure at some point below tunnel axis thereby reducing the risk of a blow, providing that the less stable tunnel invert can be accommodated. During the course of the tunnel drive, as topographical and geotechnical conditions vary, so the pressure applied should also be varied.

Air consumption is related to the permeability of the ground, the pressure applied compared with the hydrostatic pressure, the size of the tunnel and the length of tunnel previously driven which is not completely sealed. Apart from the consequences of a blow, air can also escape through nearby boreholes and drains or even through weaknesses caused by piling or basements. Due to the influence of all these variables air loss can be expected to vary from site to site. It is, therefore, necessary to provide reserve compressed air plant on site.

Boreholes should be located well clear of the tunnel alignment to avoid providing a direct route for air loss (see 4.4).

10.2.2.2 *Subaqueous tunnels.* For subaqueous tunnels, the site investigation should determine an accurate profile of the river or sea bed. Suitable techniques are described in 6.5 of BS 8005 : Part 4 : 1987.

In subaqueous tunnels, the compressed air pressure to be applied should be very carefully considered because the consequences of a blow may be very serious. There needs to be sufficient ground resistance above the tunnel to provide an adequate factor of safety against heave or a blow. Tidal fluctuations, including spring or neap conditions, should be taken into account. Previous case histories should be reviewed and an appropriate factor of safety should be derived, as a matter of judgement, after consideration of all factors. In some circumstances, it may be necessary to lay an impermeable blanket on the river limits on the vibrations caused by blasting and the time when it is done.

In recent years, mechanical excavation using partial-face (boom header) tunnelling machines and full-face tunnel boring machines, has increased but a substantial length of tunnel drive in uniform ground conditions is generally needed to justify the use of full-face machines.

With all types of plant and equipment used in tunnelling the danger of incendive sparking in rock formations likely to contain methane has to be recognized. Such considerations may require the use of explosion-protected electric apparatus and the limitation of cutting tool peripheral speeds in rock formations such as the Coal Measures in Great Britain.

The compressive strength of rock which can be cut using the more usual type of boom header machine is limited to about 50 MPa to 60 MPa, in general terms, but the spacing of discontinuities also has a considerable influence on machine performance. The heaviest boom headers can cope with rocks having strengths of up to twice the above values. Full-face tunnelling machines are capable of cutting stronger rock but are unlikely to prove economic for strengths much in excess of 200 MPa, particularly if the rock is abrasive and massive.

Any temporary support applied to the tunnel periphery has to be capable of supporting the ground until the primary structural lining can be placed, which usually consists of cast in situ concrete in sewer tunnels. The temporary support is nearly always left in place and therefore forms part of the permanent structure. The traditional method of temporary support in the UK has been steel ribs and lagging. More recently rock bolts and sprayed concrete have been used effectively as temporary support in larger tunnels but, as yet, have only rarely been used in the construction of sewer tunnels (see **10.3.4**).

Where either ribs and lagging or rock bolts are used as temporary support to the periphery of the tunnel they have to be capable of stabilizing the ground until the primary lining is placed. Guidance on the design of temporary support in rock tunnels can be found in Bieniawski [16] and Hoek and Brown [17]. Alternatively precast segmental linings or jacking pipes can be used to provide both preliminary tunnel support and primary lining simultaneously.

10.3.2 *Ribs and lagging.* Ribs (also called arches or steel sets) are cold rolled from standard steel sections and are usually fabricated in two or more sections to facilitate handling and installation. In the UK, ribs are usually arched in tunnels driven by drill and blast although full circle ribs may be used in squeezing ground and are also more suitable for circular machine driven tunnels. Foot blocks are usually placed at the feet of arches to transfer arch loads to the tunnel floor. Some typical rib profiles and sections are given in BS 227 and in Craig and Muir Wood [18].

Longitudinal support between arches is provided by tie bars or spacers. Spacing between arches is typically 1 m in a 3 m diameter tunnel, but would vary depending on ground conditions between about half and one-and-a-half times this value.

Lagging behind and spanning between the ribs can be of corrugated steel sheets, wire mesh, steel benk bars and trench sheets. The use of untreated timber in any quantity should not be allowed because it may decay in due course leaving a void behind the permanent lining. Rock loads are transferred to the ribs via timber wedges, grouted bags or packing around the tunnel periphery or directly through the lagging.

10.3.3 *Rock bolts.* Rock bolts may be used either as a form of rock reinforcement or to support individual blocks of rock in place. Many different types of rock bolt are available. The most commonly used types are described by Douglas and Arthur [19] and Hoek and Brown [17]. Rock bolts are typically half to two-thirds the tunnel diameter in length and installation is inevitably difficult in the smaller sewer tunnels. There are two main types of rock bolt as follows:

(a) the point (mechanically) anchored bolt; and

(b) the fully grouted (resin or cement mortar) bolt.

Prior to the installation of rock bolts it is essential to scale rock recently exposed by blasting. It is probably not practicable or economic to install rock bolts in tunnels of less than 2.5 m diameter.

10.3.4 *Sprayed concrete.* Sprayed concrete consists of a mixture of cement, sand, aggregate and water that is pneumatically applied at high velocity from a nozzle onto a surface. There are two different processes for conveying the material to the spraying nozzle as follows.

(a) *The dry process.* This is most commonly used in the UK. The solids are mixed dry and are conveyed pneumatically to the nozzle where water is added, under pressure in a fine spray, only a fraction of a second before placing the mixture. Additives to accelerate the setting of the concrete may be added to the dry mix or to the water at the nozzle.

(b) *The wet mix process.* A low-slump concrete is pumped at comparatively low velocity to the nozzle where compressed air is applied to accelerate the material to a higher velocity for placing.

The main advantage of sprayed concrete over poured concrete is that it can be placed without the use of formwork and can be applied to almost any thickness to suit the tunnel support requirements. It can be used with or without reinforcing mesh and also in conjunction with rock bolts, steel ribs or any other form of temporary support.

A major disadvantage of sprayed concrete, particularly using the dry mix process, is that the quality of the concrete is very dependent on the skill of the nozzle operator who controls the amount of water that is added to the concrete. A proportion of the sprayed concrete also rebounds and can amount to as much as a third of that applied. For effective spraying, the nozzle operator needs to be about 1 m to 3 m away from the surface being sprayed and, therefore, it is generally not practicable to spray concrete in tunnels smaller than about 3 m in diameter.

The use of sprayed concrete for tunnel support together with details of spraying machines and mix designs is described by Hoek and Brown [17].

10.4 Methods available for primary lining

10.4.1 *General.* The primary lining usually has to fulfil several requirements. With a shield drive, the lining provides a thrust block allowing the main rams to shove the shield forward. Where a primary lining only is used it should be designed to carry the worst combinations of ground and groundwater loading expected during the life of the tunnel. Where a secondary lining is provided, this is commonly designed to carry the groundwater pressure which may leak through the primary lining. The designer should be sure that the combined primary and secondary linings are capable of sustaining the most adverse combinations expected.

If a secondary lining is not provided the primary lining has to be fit to transport sewage. The design may need to take account of conditions when the sewer is temporarily empty for inspection as well as when under the designed operating head. The smoothness of the finished lining influences the sewer's hydraulic capacity. Conventional bolted preformed segments will normally require a secondary lining to provide a satisfactory surface finish. Smoothbore linings or (see 7.2) jacking pipes (see 7.7) can be used or an internal in situ concrete lining can be placed within the bolted segmental lining. Unbolted smoothbore linings should not be used in erodable ground where fluctuations of internal fluid pressure may lead to the formation of voids behind the lining.

In some circumstances the choice of preliminary ground support leads inevitably to a particular form of primary lining. For instance, in hard rock where rock bolting and mesh have been used as preliminary ground support it would be usual to opt for any in situ concrete lining. Methods which employ shields and mechanized tunnelling equipment for provision of preliminary ground support generally result in a preformed segmental lining or jacking pipes lining.

10.4.2 *Preformed primary linings.* Preformed segmental reinforced concrete linings (see clause 7) are available in a range of standard sizes. These take the form of either bolted or smoothbore linings connected by proprietary fixings or a range of either spigot and socket or butt-jointed pipes for pipe jacking (see table 2). For large projects or where loads are more extreme it may be economic to design a special lining for the purpose. Although cast iron segmental linings were used extensively in the past, they are rarely used at present, except under special circumstances, because of their high cost.

Whether standard or special linings are to be used, it is necessary to check that they are adequate, both in the temporary construction condition and in the final design. Guidance on the design of tunnel linings can be found in O'Rourke [20], Muir-Wood [21] and Curtis [22]. If necessary, special linings can be designed to cope with the specific conditions, for example, chemical attack.

In soft ground preformed segmental linings, as described in 5.2, are the most usual form of primary lining. Expanded preformed segmental rings (see 5.4) may be used where the stand-up time of the ground is sufficient to enable a complete ring to be constructed in the ring width exposed by the tail-less shield. They require use of a shield in order to cut a smooth profile and also to facilitate rapid erection and final ring completion which may either be by jacking or by the use of tapered wedge blocks.

Economies are often possible by the use of such expanded rings and rapid progress rates can be achieved. The lining may also be relatively cheap, as induced moments are low, due to the large number of segments so reinforcement can be minimized or absent altogether. Great care has to be taken, however, over the design of the joint configuration, which has to be able to accept a degree of building imperfection and yet tolerate high contact stresses. The high contact stresses induce tensile forces within the segments.

10.4.3 *Primary linings placed in situ.* An in situ concrete lining (see clause 8) may be used within a tunnel with temporary ground support such as steel arches and lagging, or rock bolts with mesh or sprayed concrete.

Shuttering can be in the form of ribs and lagging although for longer tunnels it may be economic to employ a travelling shutter.

In rock, in situ concrete is usually used to form a primary lining. As well as supporting any loading from the rock the concrete acts to prevent deterioration and erosion of rock and to provide the smooth finish required by hydraulic considerations.

Sprayed concrete linings may be applied to self-supporting rock or to rock reinforced by rock bolts and mesh, so that no other secondary lining is required. In rock excavated by drill and blast the resultant finish would be expected to be uneven due to the unavoidable overbreak variations. However, in rocks cut by roadheader, a more even finish may be achieved. A combination of a mass concrete invert and sprayed concrete arch may provide satisfactory surface finish. Sprayed concrete can be reinforced with steel fibres for extra strength and toughness.

10.5 Secondary linings and internal finishes

A smooth internal finish is preferred in sewers to satisfy hydraulic requirements of the sewer. When these conditions are not met by the primary lining then a secondary lining is placed within either all or the lower half. The usual form of secondary lining within a segmental tunnel is in situ concrete. Infill panels may be used to improve the hydraulic capability of ribbed segmental linings. The secondary lining can also be used as a sacrificial lining if the sewer is to carry corrosive liquids. Alternatively, a brick or ceramic tile lining may be used with epoxy pointing as a corrosion resistant layer. Precast concrete pipes or channels are also used as secondary linings.

In variable or unstable ground or where reverse gradients are critical it is often difficult to drive the tunnel and erect the primary lining to tight tolerances as required by hydraulic design considerations. In such circumstances it may be appropriate to increase the internal diameter of the initial tunnel and use the secondary lining to regulate the profile and so achieve the final alignment required.

Table 2. Range of preformed linings available in the UK

Type of lining	Typical range of internal diameter	Normal conditions of use	Limitations	Remarks
	m			
Precast concrete segments: bolted and ribbed (see 7.2)	1.5 to 7.6	Almost universal application	Needs secondary lining or infill pellets above invert if a fully smooth circumference is required	Bolted cast iron marginally effective in most arduous conditions. Can be provided with solid invert segments
smoothbore bolted (see 7.2)	1.2 to 4.6	In all ground conditions except weak clays	Not appropriate in weak difficult ground	Economic if conditions suitable
smoothbore unbolted (see 7.3)	1.0 to 4.6	In ground conditions where loading is uniform	Not appropriate in weak difficult ground and in conditions of significant hydraulic surcharge	Economic if conditions suitable
expanded (see 7.4)	1.5 to 2.9	With a shield and in firm or stiff clays which can be cut to an accurate profile	Not appropriate in other ground conditions	Very small choice of diameters
Bolted cast iron segments (see 7.5)	1.5 to 7.6	Universal application	None	Can be used in the most arduous conditions. Expensive
Pressed steel liner plates (see 7.6)	1.5 to 3.0	Light loadings and dry conditions	Needs an in situ concrete primary lining	Rarely used in UK
Pipe or culvert units inserted by jacking* (see 7.7)	0.9 to 2.4	Almost universal application in soft ground	Not usually appropriate in any but the weakest rocks. Curves require particular care	Becoming more widely used. An economical solution in appropriate conditions

* Rectangular sections are also available in a range of purpose made sizes.

10.6 Shaft sinking

10.6.1 *Ground treatment.* Situations sometimes arise where the soil conditions, as indicated by the ground investigation, require some form of treatment to be applied before shaft sinking can commence (see also 10.1). The type of treatment applied is often influenced by the system of shaft sinking adopted. The following include those most commonly used:

(a) sump pumping during excavation;

(b) de-watering prior to and during excavation;

(c) chemical or cement grouting;

(d) chemical injection and stabilization;

(e) ground replacement of the defective zone;

(f) ground freezing.

These and other ground treatment techniques, which may be used in the tunnel drive itself, are described in 10.1.

10.6.2 *Method of forming shaft.* Shafts are usually constructed in advance of, and on the line of, tunnel driving although occasionally, it may prove economical to sink some of the shafts for manholes after continuous construction of the tunnel. These shafts should normally be sited in public ground and may be used to form access manholes on the completed sewer. There are three common methods of forming a shaft as follows.

(a) Excavating ring by ring and lining the shaft by underpinning the previously constructed rings with the next consecutive ring built. Linings are usually of precast concrete but cast iron segments or steel liner plates are used in appropriate circumstances. The upper rings, at ground level, are usually held in an in situ concrete collar. The remaining rings are grouted as each stage is completed to fill any voids behind them so that the overall structure is in equilibrium.

(b) Open caisson method, where normal practice is to use segmental rings. To start sinking a shaft, a segmental cutting edge ring is erected at ground level followed by a choker ring which both have an external diameter slightly greater than the standard rings. Standard segmental rings are then built above and the whole is sunk to level in stages by excavating within, allowing the rings to settle and building further rings at the top. Settlement may be under self-weight but is usually assisted by kentledge weights which can also be used to control verticality. The cutting edge may be constructed of steel or of reinforced concrete, the choice depending on ground conditions and the depth of the shaft.

A bentonite slurry is maintained in the annular space around the shaft, as sinking proceeds, to support the ground and lubricate the shaft extrados. The slurry is kept topped up and care has to be taken not to allow its escape beneath the cutting edge. The bentonite is displaced with grout when sinking is complete, commencing at the bottom and working upwards in a continuous operation. A concrete cutting edge is normally left in place while a steel cutting edge may be recovered or left in place depending on ground conditions.

(c) Excavating and then placing a lining of in situ concrete as work proceeds. This process is more common in rock strata and for mine shafts where an irregular shape may be required.

The air lock needed when driving a tunnel in compressed air may be located vertically in the driving shaft but is more usually placed horizontally at the start of the tunnel. Shafts sunk in advance of tunnelling, and through which the tunnel is to pass, are fitted with air decks. Alternatively minimum-diameter shafts can be sunk to the side of the tunnel when driving is complete and it may be necessary to re-pressurize the tunnel to effect the connection with the shaft. The choice of option is largely a question of economics and ground conditions at the manhole positions.

10.6.3 *Access to shaft.* Adequate means of access into and egress from the bottom of a shaft should be provided. Ladders with cages and rest platforms should be installed as well as any mechanical crane or hoist arrangements. Where possible, the access part of the shaft should be physically separated from the materials handling area.

10.7 Excavation

In hard rock, excavation is commonly achieved by drilling and blasting. In softer rocks, mechanical boring systems are available which are similar in principle to those used in mechanical-soft-ground tunnelling. The systems used are frequently associated with a particular method of support. The least disturbance is caused by a tunnel constructed rapidly at depth with face support, if necessary, and with early expansion or grouting of the lining.

In soft ground, the release of stress at the tunnel face will allow some movement of the ground both laterally and vertically which will result in subsidence at the surface. The amount of movement will depend on the method of construction used and the care with which the tunnel is driven. Expert advice from an experienced engineer should be sought who would have available a soils report and recommendations from a specialist in soils mechanics.

If a shield is not used during excavation, other ground support such as timbering may be necessary. In no case should the exposed working face remain unsupported for a sufficiently long period either to endanger life or the safety of the tunnel. Tunnel driving in water-bearing ground may require the use of compressed air, bentonite support or, where the ground is particularly unstable, ground treatment using any of the methods described in 10.1. In any of these cases, in soft ground and where the excavated profile is irregular, a bolted concrete lining is generally used.

10.8 Headings

Headings are used for replacing and enlarging existing conduits, in which case the hazards associated with working in sewers should be taken into account (see Safe working in sewers [23]).

A heading is normally employed where other tunnelling techniques are unsuitable. As a non-standard form of structure it is well suited to changes of direction and shape not appropriate to prefabricated standard tunnelling designs. The cross-sectional area can be varied according to the service or services to be carried, it is normally used in short lengths and is capable of being used under a wide variety of ground conditions. However, as for all tunnelling methods, unless carried out under strict supervision the subsequent effects of using this method may be settlement and damage to property. The support of the periphery of the excavation, whether timber or other materials should be designed to prevent the loss of ground outside the excavation line. Proper consideration should be given to the following:

(a) soils information from borings;

(b) the location and proximity of adjacent services and water courses;

(c) adjacent structures.

A heading may be rectangular, trapezoidal or arched and it should be large enough to allow the efficient laying and jointing of the pipes or other constructions and the subsequent placing of approved packing. The smallest size for proper working, allowing for man entry, is about 1300 mm clear height and 750 mm clear width at the bottom but with space for ground supports to be considered in addition. Timber is the most common form of temporary structural support.

The materials used for packing the space between the temporary ground support and the installed conduit should not be placed until each unit of the latter has been laid and inspected for line, level and integrity. Great care should be taken when packing a heading to eliminate voids and pressure grouting may be necessary. Any timber to remain in place should be pre-treated with a suitable preservative (see 10.2.4). Testing should be carried out progressively. When de-watering is necessary the precautions which are normally taken in open-cut construction should be followed in constructing headings, i.e. the prevention of the removal of fine material, the compaction or consolidation of the ground and the grouting up of sub-drains on completion, etc.

Settlement may occur later as a result of the deterioration of buried timber.

10.9 Pipe jacking

Pipe jacking is a method of sewer construction which provides a strong monolithic preformed lining incorporating flexible joints. It has considerable advantages when compared with other methods such as watertightness, fewer joints, no requirements for secondary lining and good flow characteristics.

The pipe jacking process consists of transmitting a horizontal force from a vertical ground surface or anchorage by means of large capacity hydraulic rams that jack pipes forward, whilst excavation is taking place at the shield face. The material at the face is excavated either by machine or manually by a skilled operator who also controls the direction of the pipeline. The excavated material is removed along the pipeline either in tyred or rail mounted skips or by conveyors or ducts.

The shield at the front, which provides protection of the working space, is usually designed to enable the face to be boarded as required and has mechanical or hydraulic rams acting against the lead pipe to effect steering. Deviations from line and level are to be expected but these should not normally exceed ± 75 mm in line and ± 50 mm in level. Engineering supervision should be provided in order to control the accuracy of the pipeline and the use of lasers considerably assists this task.

In soft-ground conditions, remotely controlled methods are now available for pipe jacking using bentonite or slurry machines. This technique is especially suitable for tunnels of non-man-entry size.

For pipe jacking, the thrust pit has to be long enough to accommodate the jacking rig, pipe and thrust wall. The thrust pit may vary to accommodate the different types of equipment and pipe lengths and diameters in use, but is normally 4 m to 6 m long and 3 m to 5 m wide. Longer pits may be necessary for mechanical shields.

The reception pit dimensions need only be sufficient to enable the shield to be retrieved. Pipe jacking is regularly carried out from the following types of thrust pit:

(a) trench sheeted;

(b) sheet piled;

(c) segmental shaft;

(d) battered excavation.

In shallow application or where jacking is taking place at ground level, such as through an embankment, the jacking reaction is often provided by constructing a purpose-made thrust block.

What is achievable in pipe jacking lengths is dependent upon a number of inter-related and variable factors as follows:

(1) the arching and friction characteristics of the ground;

(2) the self-weight and strength of the pipes;

(3) the diameter of the pipe;

(4) the type of shield;

(5) the available jacking reaction;

(6) any specialist engineering techniques used to lengthen the distance jacked.

The use of intermediate jacking stations introduced into the line between two special pipes, as the thrust proceeds, reduces the jacking force required and assists in permitting installation of long pipelines.

The pipe jack shield is designed to produce a small overbreak to the external diameter of the pipeline. By injecting a lubricant into this overbreak the friction force between the pipeline and the surrounding soil can be reduced, resulting in a reduction of the jacking force required to install the pipeline.

The major constraint will be the nature of the ground and water content. Inappropriate use may result in an aborted push and costly recovery measures.

The pipe jacking contractor will normally prefer to install the pipeline in an upgrade direction. This enables any water met at the face to be dealt with at the thrust pit. Jacking down gradient can take place in wet conditions by using face pumps to control the water.

Pipe jacked pipelines can be installed to horizontal and vertical curves dependent on ground conditions and pipe design but this is an aspect where specialist advice should be obtained.

10.10 Auger and thrust boring

Auger and thrust boring are methods of pipeline construction for pipe of diameters usually smaller than man-entry size. Because of lack of accuracy, they are usually confined to short lengths such as under road or rail embankments or suburban streets.

Auger boring entails the advance of a steel outer tube within which a screw auger both removes the spoil and carries it back into the pit. The tube cannot be steered and the location should be carefully chosen to avoid obstructions or ground conditions which could divert the line as a true line and level is extremely difficult to achieve. Further sections of pipe and auger screws are connected as the line advances until breakthrough is achieved.

Thrust boring is sometimes used as a misnomer for auger boring or pipe jacking. The term is usually applied to mechanical mole devices which force their way through the ground powered by air or hydraulics. They are generally incapable of being steered and are subject to the same limitations as auger boring. The device is often 50 mm to 100 mm in diameter. If a larger thrust boring is required, enlarging cones are pulled back and forth through the initial bore to compress the ground round the bore until the required diameter is achieved.

10.11 Grouting

When non-expanding linings are used, particular attention should be paid to filling the annulus formed during construction between the extrados of the lining and the adjacent ground. The essential requirements for such linings are:

(a) to ensure that the ground loading is transferred to the lining in a manner consistent with the structural design; and

(b) to limit any ground movements and water ingress.

In the case of bolted linings, the frequency of the grouting should be determined after due consideration has been given to the stability and characteristics of the ground following the release of stress by excavation. In any event, grouting should be carried out at least once per shift.

With shield-driven flexible-smoothbore segmental construction, the grouting operation should be undertaken immediately following the exposure of the built ring to the ground beyond the limit of the shield tail. As in any method of construction, the efficiency of the grouting will contribute to the watertightness of the tunnel. The principal techniques which may be employed are as follows:

(1) the pressure application of cementitious grouts;

(2) the placement of granular material followed by the injection of either cementitious or chemical grouting material into the interstices of the granular fill;

(3) the pressure application of flexible chemical grout. Pressures in excess of 7 bar should be avoided so as not to distort or damage the lining.

The placement of granular materials and grout should be verified volumetrically and related to the calculated volume of the annulus. Cast iron or precast concrete linings are usually provided with pre-formed holes to assist in grouting. In cases where the tunnel is lined with a primary lining of brickwork or concrete placed in situ, holes may be drilled through these materials for grouting purposes. Alternatively, grout ducts may be built-in during construction.

It is usual to grout at the outside of an in situ concrete lining, either in a rock tunnel or within a segmental lining, to seal any voids. Exploratory holes are drilled in the area of the crown at regular intervals and voids grouted to refusal. Additional holes may be required at some locations.

Normally, caulking (see 10.12.4) is carried out after grouting, but in some cases this may have to precede the grouting operation to assist in the retention of the grout under adverse conditions. In these cases the integrity of the caulking should be checked on completion of the grouting operation. Special procedures for caulking are required where compressed air is used.

10.12 Waterproofing segmental linings

10.12.1 *General.* Waterproofing is expensive and time consuming and the cost of producing a totally watertight tunnel should be a consideration for the design engineer and promoter. It may not be necessary for a tunnel to be completely watertight.

Although grouting behind the tunnel lining is not specifically designed as a seal against water intrusion it can assist in this respect and can be supplied with waterproofing additives. However, due to shrinkage and ground movement the grout can develop leaks and it is therefore normal to undertake other waterproofing methods. Choice of lining system may be influenced by the degree of watertightness to be achieved.

It should be noted that no grouting is used during the construction of expanded precast concrete segmental linings.

10.12.2 *Grummets.* Precast concrete and cast iron bolted segments have shaped bolt hole recesses designed to accommodate grummets made from sisal hemp impregnated with a mineral gel. Plastic grummets have also been used.

The grummets are placed under the washers at the head and nut of each bolt and, as the bolt is tightened, the material flows into the bolt hole to form a seal.

10.12.3 *Caulking.* Segmental linings are usually provided with caulking grooves. In dry conditions these are filled with sand-cement mortar with or without a waterproofing additive. In wet conditions a compound of cement and asbestos-free fibrous fillers, loosely braided into the form of a cord, may be used for caulking. The compound is first dampened with water before being compacted into the joint using a hammer and caulking tool. In extremely wet conditions lead wool caulking may have to be used to ensure watertightness.

Other proprietary brands of caulking and sealing materials are available.

10.12.4 *Joint gaskets.* For high water pressures, segmental linings are provided with gasket grooves between the bolt hole and the extrados of the segment. Accurately-made chloroprene rubber or natural rubber, formed into rectangles with moulded corners, are fixed into the gasket grooves with adhesive. Segments may need to be made thicker to accommodate the grooves in the circle and cross joints.

A high degree of accuracy in erecting the segments, together with a high standard of cleanliness, is required for the gaskets to be completely effective.

10.12.5 *Sealing strips.* Some precast segmental linings have specially profiled joints that are designed to permit the use of bituminous or a similar jointing strip, which is applied to both longitudinal and circumferential joints. The effectiveness of the seal achieved depends on the accuracy of erection and the general standard of cleanliness.

10.12.6 *Hydrophilic seals.* Joint seals of hydrophilic-rubber composition, which expand up to 10 times in volume on contact with fresh water, can be placed in either specially formed grooves in the circular and radial segment joints or fixed to the face of the segment joint with adhesive.

The seal absorbs the water leaking through the segment joint causing the seal to expand and effectively stopping any further leakage. Care has to be taken that the seal is not dislodged during the installation of the segments.

11 Permissible deviations

Where a tunnel is to have a secondary lining, permissible deviations for the primary lining may be specified with regard to the nature of the work.

Where the avoidance of reverse gradients is critical, suitable limits should be detailed. However, the deviations given in table 3 are maximum permissible deviations, which may be improved upon by imposing stringent controls on workmanship, for the internal face of any shaft, pipeline or tunnel.

Table 3. Recommended permissible deviations for internal faces

Feature	Maximum recommended permissible deviation*
General	
Verticality of shafts	1 in 300
Finished diameter of tunnels and shafts	± 1 %, but not exceeding 50 mm
Maximum lip between edges of adjacent segments	5 mm
Tunnels where a secondary lining is to be provided	
Line (shield drive)	± 75 mm
Line (hand drive)	± 50 mm
Level (shield drive)	± 50 mm
Level (hand drive)	± 25 mm
Pipe jacking	
Line	± 75 mm
Level	± 50 mm
Angular deflection between pipe joints	< 0.5°
Secondary lining within a tunnel or pipejack	
Line	± 20 mm
Level	± 10 mm

* In variable or unstable ground it may be appropriate to allow a larger tolerance as regards levels.

Appendices

Appendix A. Bibliography

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Appendix B. Stability number

A convenient indication of the short term stability of the tunnel face and its vicinity is the 'stability number'. This figure is derived from the difference between the external and internal pressures divided by the undrained shear strength and is dimensionless. The stability number, N*, is derived from the following equation:

$$N = \frac{(\sigma_z - \sigma_T)}{c_u}$$

where

σ_z is the total vertical pressure at the tunnel axis including surcharge and ground water;

σ_T is the temporary support pressure above atmosphere in tunnel, e.g. compressed air or bentonite slurry;

c_u is the undrained shear strength.

The equation was developed by Broms and Bennemark [26] and as quoted in Craig and Muir-Wood [18]. See also Somerville [25].

Previously a value below 6 was considered to give stable conditions although, in stiff fissured clays, the shear strength indicated by 38 mm diameter samples needs to be reduced to allow for the effects of fissuring.

More recently it has been shown (see, for example, Atkinson and Mair 1981 [27]) that the situation is more complex and depends on the length of tunnel behind the face before full support is provided by a grouted or expanded lining. For shallow tunnels with an unsupported length (P) of three to four tunnel diameters (D) behind the face. Figure 4 shows that the stability number at tunnel collapse needs to be below 3, whereas deeper tunnels with lining support up to the face are probably stable up to a value of 9.

NOTE. It should be remembered that, in this context, a shield does not usually support the ground, its primary purpose being to protect the tunnellers and enable the lining to be erected (see 10.2.1).

It should be noted that conditions are most stable when the amount of unsupported ground between the face and lining is a minimum. Additional stability is obtained by reducing the area of face unsupported at any time (see Davis, Gunn, Mair and Seneviratne [28]). Thus if tunnel diameter (D) is halved the C/D ratio is automatically doubled.

Localized instability at the tunnel face is related to the following formula†:

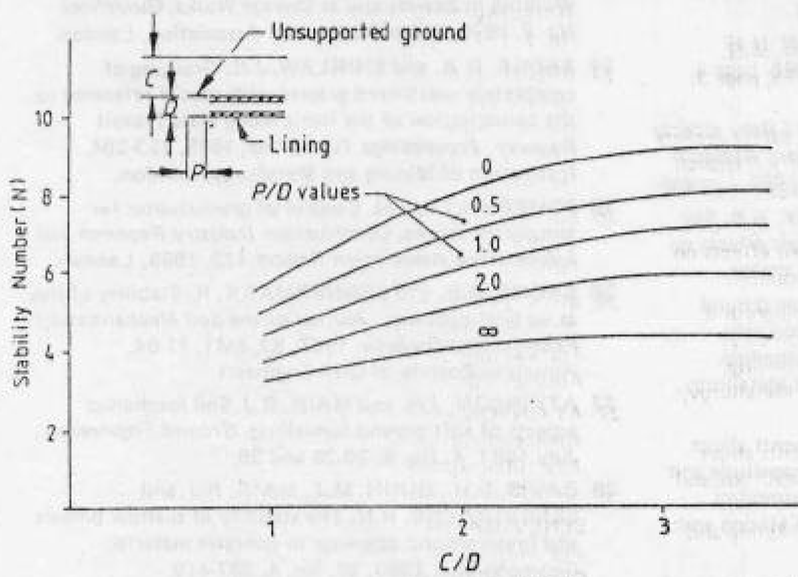
$$\frac{(\gamma D)}{c_u}$$

where

γ is the unit weight of soil;

D is the diameter of tunnel;

c_u is the undrained shear strength as above.



NOTE 1. This graph has been taken from Atkinson and Mair [28].

NOTE 2. Where C is the cover, i.e. the depth to the tunnel crown from the ground surface; D is the diameter of the tunnel; P is the unsupported length.

Figure 4. Influence of heading geometry on stability number at tunnel collapse

* The same value of the dimensionless number is obtained using any consistent set of units. For example, both pressure and strength are in kN/m^2.

† The formula is dimensionless. The same value is obtained using any consistent set of units. For example, unit weight in kN/m^3, diameter in metres and strength in kN/m^2.

Index

Clause references given in bold denote that they are the main reference for the subject.

Publications referred to

BS 227	Specification for H-section steel arches for use in mines
BS 410	Specification for test sieves
BS 1377	Methods of test for soils for civil engineering purposes
BS 3921	Specification for clay bricks
BS 4118	Glossary of sanitation terms
BS 5268	Structural use of timber
	Part 5 Preservative treatments for constructional timber
BS 5328	Methods for specifying concrete, including ready-mixed concrete
BS 5480	Specification for glass fibre reinforced plastics (GRP) pipes and fittings for use for water supply or sewerage
BS 5911	Precast concrete pipes, fittings and ancillary products
	Part 100 Specification for unreinforced and reinforced pipes and fittings with flexible joints
	Part 101 Specification for glass composite concrete (GCC) pipes and fittings with flexible joints
	Part 120 Specification for reinforced jacking pipes with flexible joints
BS 5930	Code of practice for site investigations
BS 6100	Glossary of building and civil engineering terms
BS 6164	Code of practice for safety in tunnelling in the construction industry
BS 6297*	Code of practice for design and installation of small sewage treatment works and cesspools
BS 8004	Code of practice for foundations
BS 8007	Code of practice for design of concrete structures for retaining aqueous liquids
BS 8110	Structural use of concrete
	Part 1 Code of practice for design and construction
BS 8301*	Code of practice for building drainage
BS 8005	Sewerage
	Part 0 Introduction and guide to data sources and documentation
	Part 1 Guide to new sewerage construction
	Part 2 *Guide to pumping stations and pumping mains
	Part 4 Guide to design and construction of outfalls

* Referred to in the foreword only.

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