Structural Repair by Grouting

by D.A. Bruce
BSc, PhD, C.Eng., MICE, MASCE, MIWES, MHKIE, FGS

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1. INTRODUCTION

The requirement for structural repair and renovation arises from a wide range of factors.

With regard to *well established structures,* a slow deterioration of the fabric and/or foundations over a long period may naturally occur to a significant degree. Alternatively it may be necessary to upgrade the capacity of otherwise satisfactory older structures to meet modern standards of safety or performance in situations where the construction of a replacement is not possible for various reasons including:

- historical – it is deemed necessary to preserve as much of the existing structure as possible
- practical – site and operating circumstances render the option of demolition and rebuilding an impossibility
- commercial – the cost of new construction is far in excess of the cost of repair, particularly if the possible interim loss of revenue or services from the existing facility is considered.

With regard to *newer structures,* say less than ten years old, it may be that inadequacies in the design and/or construction, or unforeseen performance of the founding material will result in structural problems which impair to an unacceptable level the functional capacity of the structure. Such cases are particularly common throughout the Middle East where the contributory effects of harsh and aggressive climatic regimes are now being revealed with increasing severity.

For repairs principally to the fabric of structures, effective and often highly sophisticated systems have been developed. Mostly these feature resin injection, supplemented by mechanical reinforcement in the form of bolts, dowels or bonded external metal strips[1]. Other principles of repair such as the vacuum impregnation technique may also be cited but these generally have a more restricted field of potential application. To effect such repairs in a suitable and long lasting manner, careful design and meticulous execution are essential. Coupled with the inherently high cost of the special materials involved, the overall financial implications of such repair options may prove prohibitive, and dictate demolition and replacement rather than restoration.

However, within the broad field of structural repair, there remains great scope for applying in an efficient and cost effective manner the basic principles used by geotechnical engineers mainly involved in foundation problems. In Britain especially, much emphasis is being placed on works within the theme of "maintenance engineering", as opposed to new construction. Thus there are programmes underway to improve and upgrade the performance and efficiency of vital existing services and facilities such as dams, bridges, railways and harbours, and sewers. There are schemes to improve the environment of the so called "inner city" regions. Steps are being taken to invigorate industrial areas associated with traditional industrial bases, such as coal and steel. In each of these fields, repair by grouting has a fundamental contribution to make.

Photograph 1. View of Pont Rug, N. Wales

It is acknowledged of course that other geotechnical processes are often used together with, or instead of, grouting as a repair technique, and these include

- minipiling[2]
- soil nails[3]
- jet grouting[4]
- conventional underpinning[5]
- fabric formwork[6]
- conventional sewer sealing repairs[7]

The practical application of grouting and these related geotechnical processes are described in the case histories presented below. The following major categories of application are considered:

- bridges, roads and embankments
- base slabs
- tunnels, culverts and sewers
- ports and harbours

The purpose of this review therefore is to illustrate the range of techniques which imaginative engineers may exploit in problem solving. It is also intended to provide sufficient technical information on the techniques to confirm the high level of scientific control which practitioners can now exercise, bearing in mind the traditional view of grouting as an art made less pretentious by a levening of black magic.

2. BRIDGES, EMBANKMENTS AND RETAINING WALLS

2.1 Walls and Bridges, N. Wales

The Sites

Throughout N. Wales, including Snowdonia National Park, there is a multiplicity of structures such as bridges and retaining walls built between 150 and 200 years ago (Photograph 1). Many are designated as National Monuments. They comprise blockwork skins to heartings consisting of a variety of materials from boulders to silts. The masonry usually consists of flat blocks up to 200mm thick and an average 150 x 300mm in elevation, but more variable for the bridge details. The blocks consist of hard fine to medium Palaeozoic sediments such as slates and greywackes.

The Problem

The natural effects of time, exacerbated by the general increase in vehicle loading and vibration levels, have led to deterioration and distress of certain structures: for example pointing has corroded and crumbled away (exploited by root networks) permitting easy access of water to the hearting, which now bears cavities; structural cracks develop, associated with outwards movements of spandrel walls or differential settlement of abutments.

The Solution

A range of processes (Table 1) is used to renovate and secure such structures, and Figure 1 illustrates the work involved for a certain group of adjacent structures recently repaired in Gwynedd. In addition of course, various ancillary elements are necessary, such as placing of scaffolding, removal of vegetation, contingencies against grout contamination of the associated rivers, minor excavations, concreting and black topping, and washing down and making good of the structures to best visual effect. More detail is provided on another local structure – the Pont Lima Bridge – by Truman-Davies and Kemsley[8].

The Execution

Deep penetration pressure pointing was a first necessity on particularly open blockwork to minimise subsequent grout loss, to reinstate "bond" between adjacent blocks, and to prevent further deterioration of the masonry facing. Such pointing can also be used as the sole means of repairing walls where no later consolidation grouting is to be undertaken. Clearly the geometry of the joints reflects the style of the blockwork, but typically they are 10-50mm wide, 50-100mm deep and provide about 10 linear metres/m^2 of face. They were first cleaned and raked out before being flushed further with air and water. For vertical faces the typical mix was a 2:1 silica sand:cement grout W* = 0.35, plus about 12% pfa (by weight of cement) to make the colour of the pointing in situ more acceptable visually. For overhanging faces (e.g.

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*Formerly Contracts Director, GKN Colcrete, Thorp Arch Trading Estate, Wetherby, West Yorks. LS23 7BJ.

*Throughout this paper, W, the water:cement ratio, is expressed as a weight ratio.

Structure	Remedial Work Requirement	Main work Executed	Notes
Retaining Wall: Bwlch y Gwyddel Pen y Pass	. Deep penetration pointing of structure to supplement and replace existing. . Weep holes. . Minor repairs and reconstructions.	. 688m² of pointing plus 135 weep holes . Almost 4m³ dismantling and rebuilding parapet walls/copings.	In Snowdonia National Park therefore very high standard of workmanship required.
Bridge: Pont Rug	. Surface pressure pointing to arch rings, soffit, abutment, piers, training walls, spandrels. . Consolidation grouting of hearting. . Weep holes. . Tie bars.	. 116m² pointing. . 191 grout holes involving 13.7 tonnes neat OPC grout and 43.1 tonnes of 2:1 sand cement grout at WSR. . 11 Nr 7.2m long bars, 32mm diameter (UTS 800kN).	Removal of vegetation showed masonry locally to require replacement. Hearting found to be very permeable and required grout curtain at East end plus full consolidation before stitching. Special care taken to prevent contamination of trout stream.
Bridge: Pont Rhyd-y-Fuwch	. Deep penetration pointing for abutments, piers, training walls, arch rings and soffits. . Weep holes.	. 172m² pointing plus 17 weep holes.	
Bridge: Pont Newydd-Aber	. Limited amount of surface hand pointing to spandrels, retaining wall, arch rings, soffits, abutment and training walls. . Consolidation grouting of hearting. . Weep holes.	. 14.6m³ excavation and back filling. . 69.4m² pointing plus 22 weep holes. . 102 grout holes involving 6.2 tonnes OPC and 3.4 tonnes 2:1 sand cement grout.	As for Retaining Wall above. National Monument.
Bridge: Pont Dylif	. Hand pointing. . Consolidation grouting. . Weep holes. . Tie bars.	. 44m² of hand surface pointing +10 weep holes. . 34 grout holes involving 2.6 tonnes OPC and 3.4 tonnes sand cement grout. . 7 Nr 16mm tie bars 1m long to arch soffit.	Inspection of arch soffits had shown flattening of arch in both directions. Subsidence noted at 2 d/s abutment corners.

Table 1. Details of Structures Repaired, N. Wales

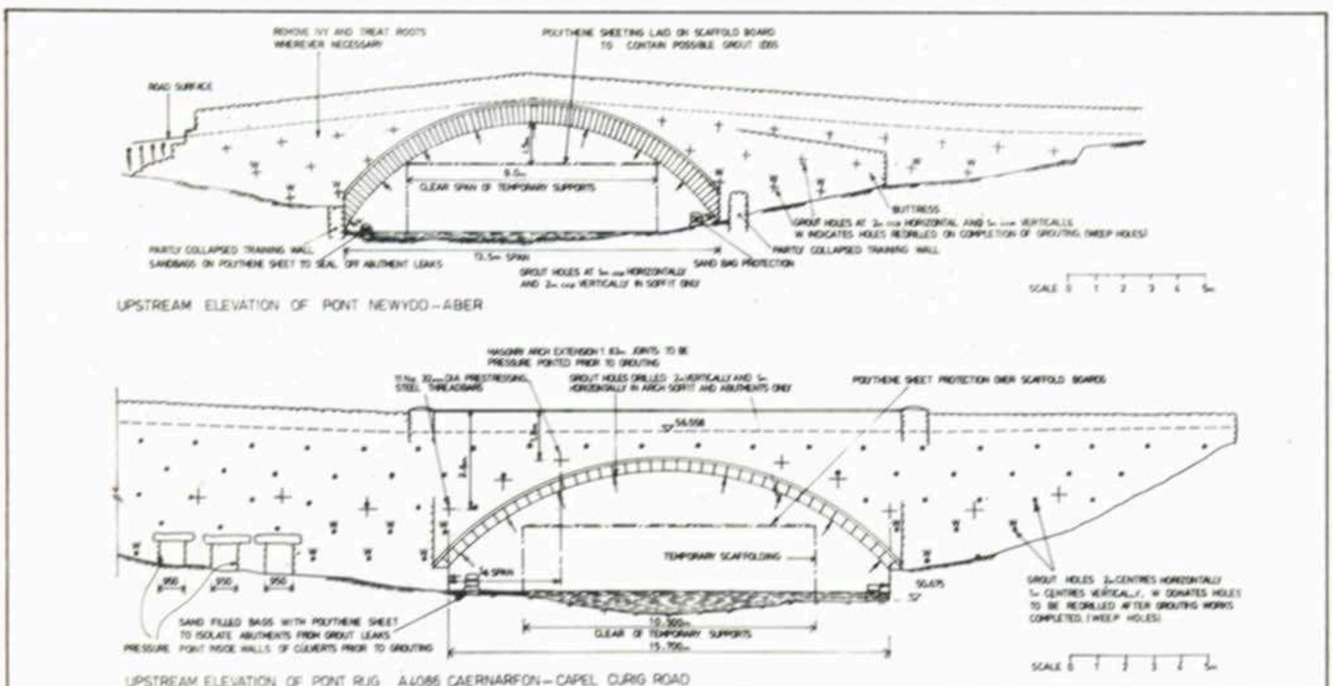


Fig 1. Details of Repairs Conducted on Two of the Bridges referred to in Table 1, above.

soffits) the mix reverted to a 1:1 grout of W = 0.32 (plus pfa). The mortar was prepared in a roller mixer and pumped to the 18mm pointing nozzles by a small Colmono pump. Afterwards prior to final set, the pointing was wire brushed, and the surface washed to improve final appearance. Alternatively the pointing may be left with a "weld", trowel or recessed finish.

The consolidation grouting was executed through 50mm pvc ducts mortared or drilled in and used the range of colloidal neat cement, and sand cement grouts noted in Table 1. Grouting sequences, volumes and pressures were carefully designed and controlled so as not to damage delicate structures by, for example, exerting significant bursting pressures through large volumes of fluid grout. The effectiveness of the consolidation grouting was verified by grout take analysis, and by grout and/or water absorption tests in random holes, whilst its thoroughness was further demonstrated by interconnections between adjacent holes.

Weep holes consisting of 50mm pvc tubes were set through the blockwork in selected locations. These are a wise precaution in that the composition of the hearting is very varied and may contain ungroutable zones of silt or clay. These allow seepage of water, which if unrelieved, can initiate further damage by pressure build up, or frost heave in winter.

Tie bars were placed in 89mm dia. cored holes, drilled by diesel hydraulic frame mounted rigs. These tie bars reinforce the structures and/or foundations and inhibit any future tendency for movement. They were doubly protected against corrosion by grease impregnated tape and corrugated plastic tubing, and were fully grouted in place. The headplates were likewise protected and bedded on a sand cement mortar base. A nominal bar stress was applied. They were thereafter trimmed and dressed to blend into the surrounding masonry. Occasionally, where it is desirable to conceal even the headplates, these are placed in the cavity created by removing the appropriate facing stone, later replaced. The bars may be introduced before consolidation grouting (they would help to resist any grouting induced stresses), but it is more common to place them after consolidation when the hearting is more stable and easier to drill.

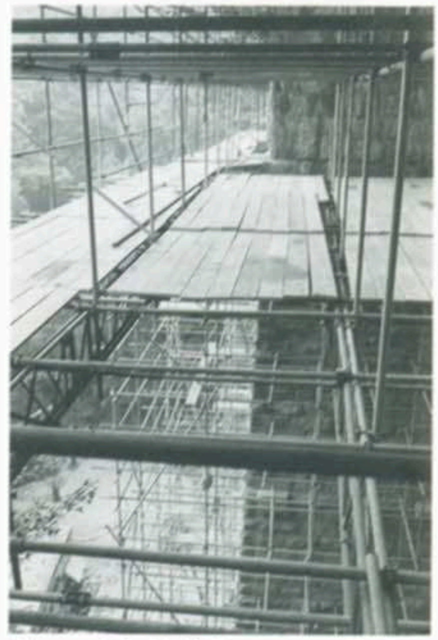
2.2 Kincardine Viaduct, Scotland
The Site
British Rail Underbridge 56A, Kincardine Viaduct, is located in the Kincardine Castle Estate near Auchterarder in Perthshire. Built in 1848 the masonry structure composed of sandstone blocks with granular fill, is about 157m long and over 29m above ground level at its highest point (Figure 2). It comprises two abutments, each with a small approach span and 5 intermediate piers a maximum of 4m square in section. The structure is typical of many such bridges built at the time and still in operation today.

The Problem
Investigation of the bridge confirmed that water from track level had penetrated through the whole structure. The action of this flowing water (and the ice effect in winter) had over the years led to a significant deterioration of the sandstone blockwork and a removal of fine granular material from the spandrel walls and piers, through the joints of the masonry. To maintain this major structure at an acceptable standard, a comprehensive scheme for both waterproofing and strengthening was therefore evolved.

The Solution
Since details of pier construction were not available, a preliminary site investigation was undertaken to determine the thickness of the walls, the nature of the hearting, the footing dimensions and the allowable bearing pressure. Advice was obtained from the Geology Department of Dundee University on the characteristics of the pier stone, badly fractured and spalled in places. A small grouting trial of a pier base was also undertaken. Although it is normal to point masonry before attempting pressure grouting and/or stitching (as this prevents easy escape of the grout), the joints were relatively tight in this structure and so this preliminary was not held to be necessary. Thus the solution proposed to repair the structure was as follows:

- Grout and stitch all five piers from ground level to springer level. The grouting would restore the integrity of the hearting, whilst the stitching bars would secure the walls to the core, thus resisting any bursting forces.
- Grout all six arches and spandrel walls, and the arches and abutments of the approach spans, and install bolts where necessary.
- Conduct cosmetic repairs such as limited pointing, removal and replacement of damaged masonry sections, cleaning and washing etc.
- In addition a separate contract was let for waterproofing of an existing concrete mat over the viaduct. This involved a cut off drain at the high end of the viaduct together with a complete tanking of the deck in mastic asphalt, and formation of weepers through the spandrel walls.

The Execution
Working programmes for the grouting were dictated by the intricacies of the scaffolding (Photograph 2), but generally about three piers were worked on at any one time, each at about the same level. Four lifts of primary drill holes



Photograph 2. Scaffold access at mid pier level, Kincardine Viaduct, Scotland

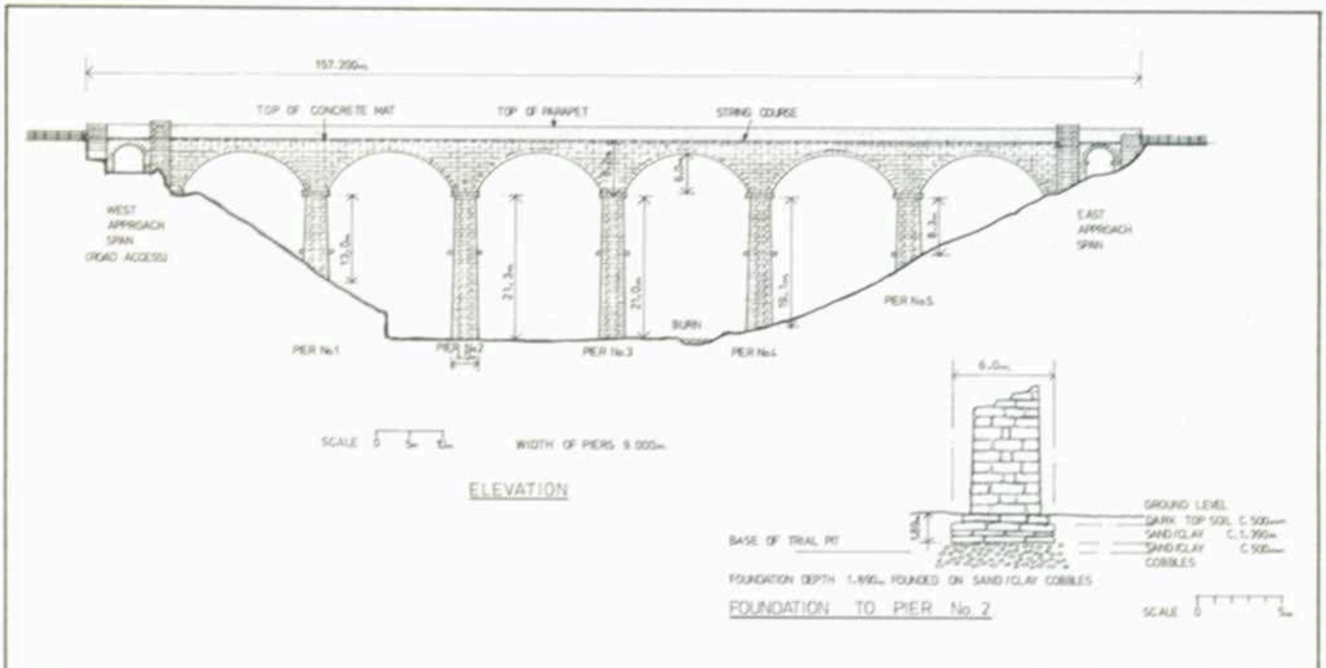


Fig 2. West Coast Main Line Underbridge No.56A Kincardine Viaduct, Auchterarder

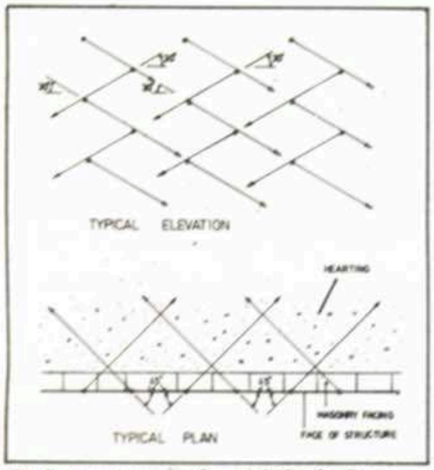


Fig 3. Arrangement of grout holes for Piers, Kincardine Viaduct

were permitted at any one time prior to grouting. When eight lifts of primary grouting had been completed, the secondary holes were drilled in intermediate locations. Each lift of 500mm involved 10Nr 2.5m long holes. Throughout the treatment of the piers from ground level to springing level the drilling and grouting of secondaries were always four lifts below completed primaries.

The 38mm holes were drilled with water flush hand held pneumatic powered rigs. The 1:3 OPC:PFA mix at about 0.5 WSR was mixed in a colloidal mixer, and injected via Colmono pumps.

The extent of the work over the 1960m² treated area was as follows:

Pier 1	24 Primary	and	24 Secondary	lifts
Pier 2	38	" "	38	" "
Pier 3	36	" "	36	" "
Pier 4	37	" "	37	" "
Pier 5	24	" "	24	" "

accounting for 1469 Primary and 1453 Secondary holes in the pattern of Figure 3.

Thereafter, the piers were redrilled and stitched over the same height (primary locations only) with 2.5m long 16mm HY rebars grouted with the same cementitious mix. The total grout consumption in the Piers was 144.9 tonnes of solid materials.

Attention was then transferred to the Eastern and Western Approach structures. The 900m² of spandrel walls and East and West Approach Spans were grouted as shown in Figure 4, to depths of 2.5 and 5.0m. The holes were drilled at 90° to the vertical and horizontal in the same type of programme as for the piers with 514 Nr 2.5m Primaries and 175 Nr 5m Primaries, followed by 87 Nr 2.5m Secondaries. These holes consumed 175.5 tonnes of grout.

The 1300 m² of arch soffits were drilled using a Falcon Stoper to depths of 1-2.5m as per Figure 5. The 948 Primary and 306 Secondary holes consumed 79.8 tonnes of grout. These lower average takes reflected the spread of grout from the earlier treatment of the spandrel walls.

Weep holes up to 5m long were also installed to relieve seepage water pressures in the abutments.

Tie bars of 25mm dia. were drilled and placed throughout the six arch spans, to a depth of 2.5m. Each hole was flushed clean and anchoring of the distal end of each bar was achieved with a Selfix resin capsule. After the resin had hardened, the remaining annulus was filled with a neat cement grout. End plates and nuts were treated with 'T' wash protective before fixing, and were later coated with a bitumastic paint. A total of 120 ties were placed throughout the arch spans, viz. 10 North and 10 South of each span.

449m² of hand pointing was then conducted in selected areas, principally below the spring course level, and some cutting out and replacement of damaged masonry (with Fyfestone) carried out, mainly beneath arch soffits and pier covers.

As a final cosmetic step, the whole structure was cleaned with a 2000psi jetting pump to remove all grout stains and plant growth.

2.3 Pont Glan Gwynedd, N. Wales

The Site

Pont Glan Gwynedd is located astride the boundary between the counties of Powys and Gwynedd, 2 kilometres north of Machynlleth on the B4404 road. The original part is believed to have been built in the early 19th century, having been designed to carry pack mule traffic. It was considered to consist principally of facing stones up to 500mm thick with fill comprising rubble.

The Problem

At some stage in its history the bridge was widened by butting an arch extension against the existing structure without any physical connection. Subsequently a concrete invert and apron were constructed for each of the spans. This had been destroyed within Span 3 (Figure 6) presumably at the same time as Pier C was undermined by scour. This pier was subsequently supported by concrete bags as an interim measure.

The subsequent structural problems appear to relate directly to the original method of construction, and the widening works. Separation of the extension from the old bridge was indicated by a crack running longitudinally along the bridge. Masonry pointing had also locally deteriorated, assisted by the subsequent growth of vegetation, and physical separation of some of the masonry blocks from each other was occurring. There was further evidence of scouring beneath the bridge piers.

As a result of all these factors, the maximum gross vehicle loading on the bridge was reduced to 7.5 tonnes.

The Solution

In order to repair the structure the following measures were designed (Figure 6):

- Minipiling: to transfer the load of the bridge to sound bedrock (Table 2). These 220mm diameter cast in situ piles were to be installed at about 330mm centres to support the piers and abutments and to form a perimeter for the subsequent consolidation works.

  The piles were designed to penetrate about 600mm into rock. They were also to be permanently cased from bedrock to about 2m into the bridge, using 168mm OD steel casing

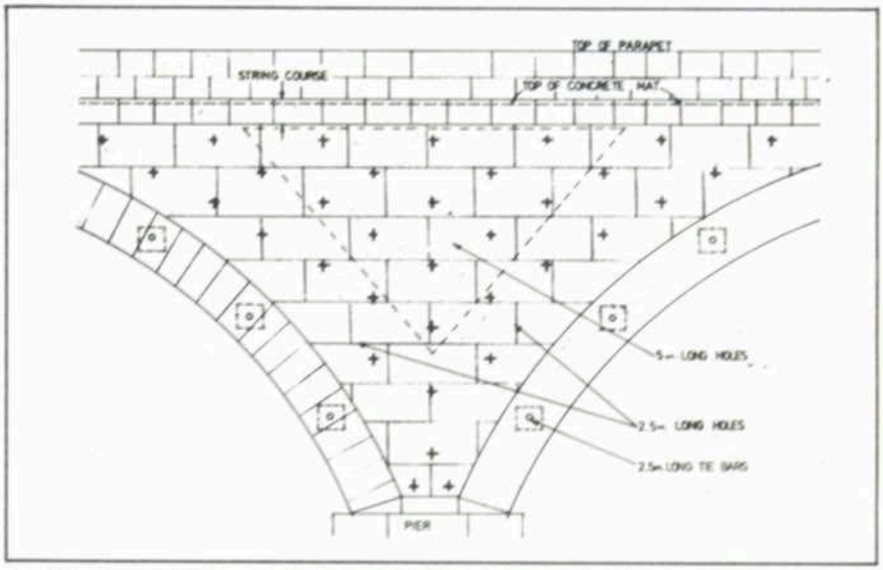


Fig 4. Arrangement of grout holes for Spandrel Walls, Kincardine Viaduct

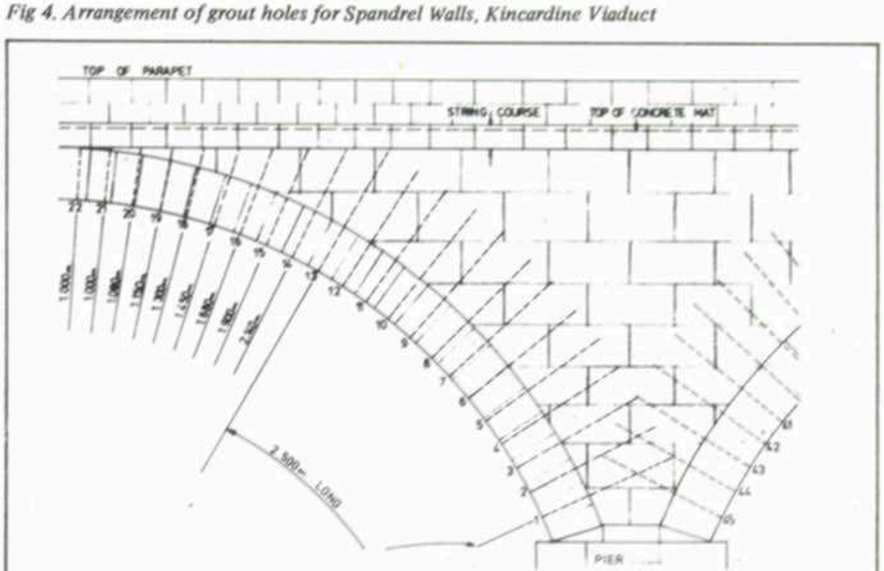


Fig 5. Arrangement of grout holes for Arch Soffits, Kincardine Viaduct

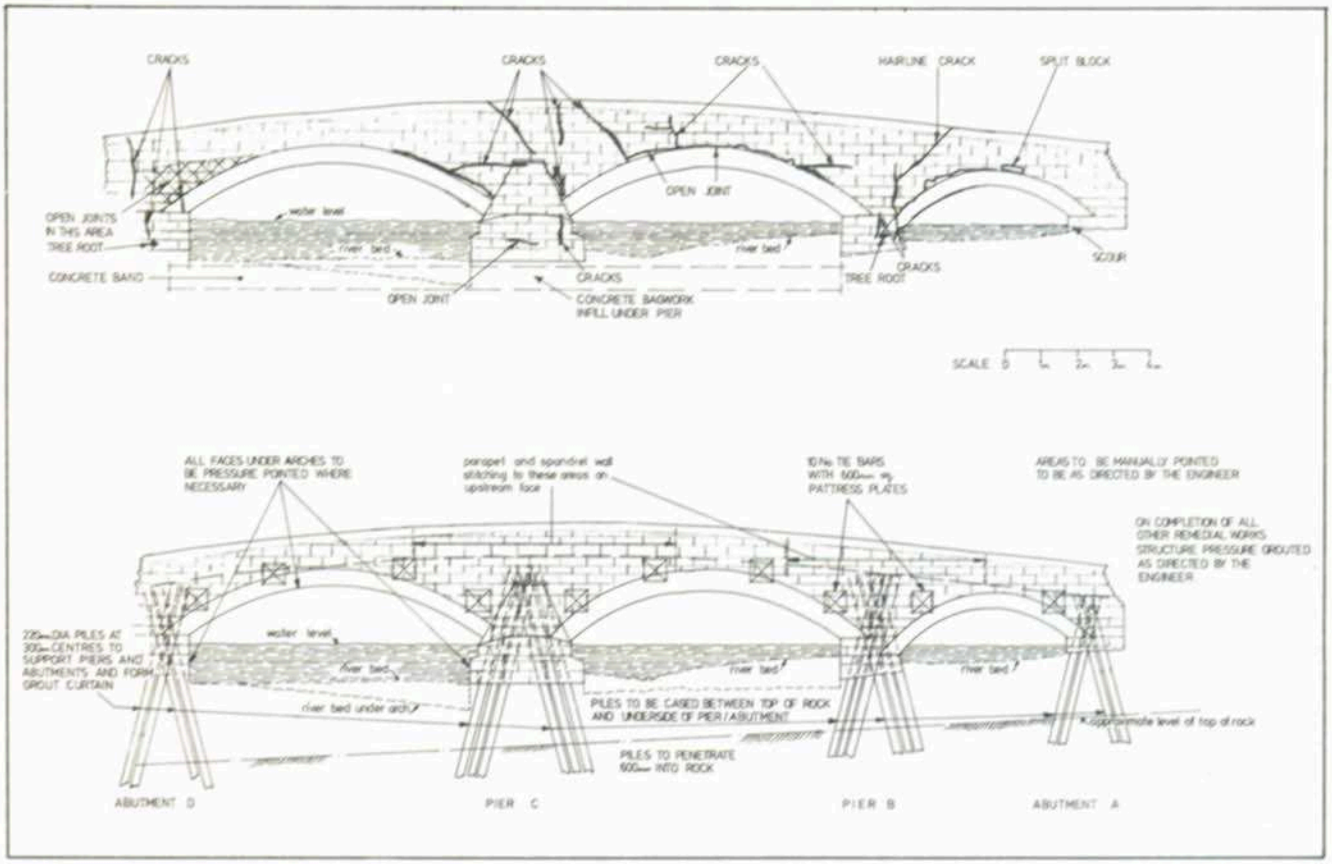


Fig 6. Upstream Elevation of Pont Glan Gwynedd showing extent of deterioration and repairs

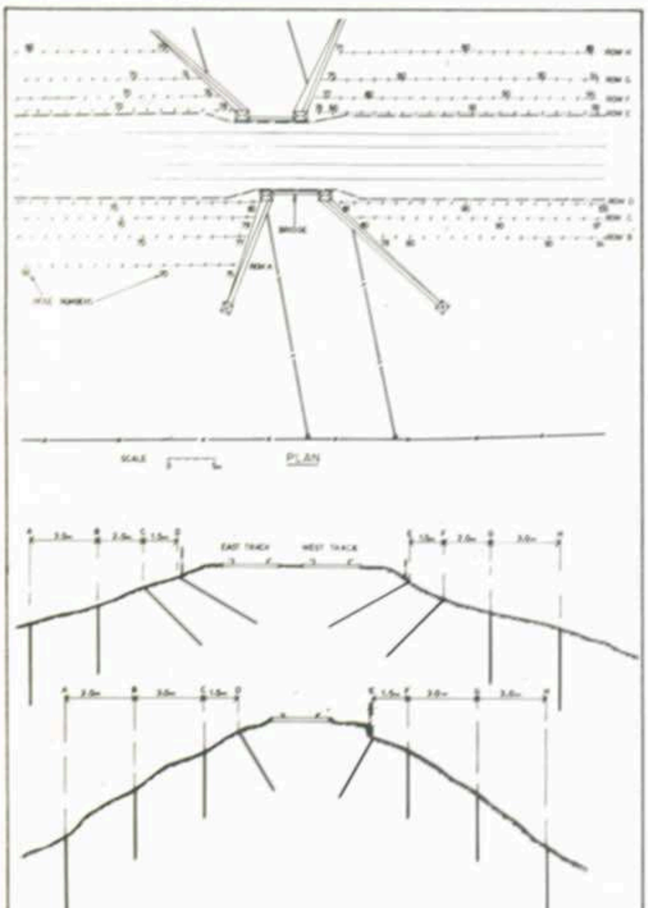


Fig 7. Typical grout hole arrangement, railway embankments, London Underground (Ltd)

Location	Total Loading	Pile Details
Abutment A	1070kN	17 Nr – 13 across abutment + 2 in returns at each side
Pier B	2312kN	36 Nr – 26 across pier + 5 in each side in returns
Pier C	3452kN	38 Nr – 26 across pier + 5m each side in returns + 2 to support cutwater
Abutment D	1988kN	17 Nr – as per Abutment A

Table 2. Arrangement of Minipiles, Pont Glan Gwynedd

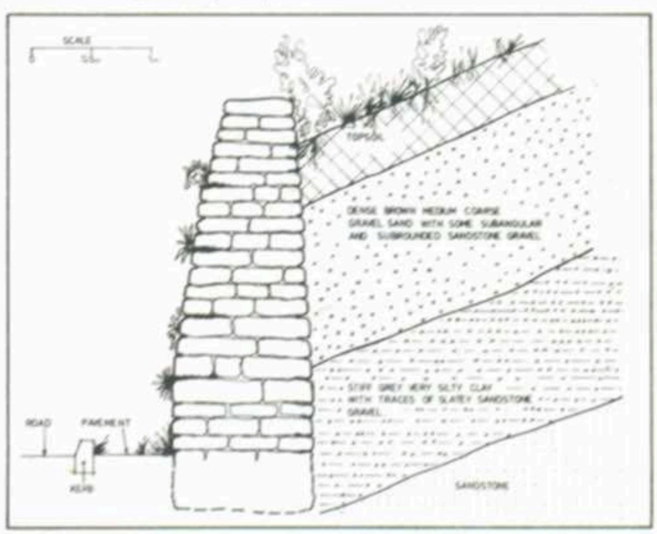


Fig 8. Typical cross section of retaining wall before repair, Denholme Clough, Yorkshire

5mm thick (to BS4360 Grade 43 C) as protection. The main load bearing element was a centrally positioned 25mm steel bar.

- Tie bars: ten tie bars to tie together the two sides of the bridge.
- Stitching: 24 Nr 16mm bars within the parapets and spandrels.
- Pointing: to repair the defective areas, particularly under the arches.
- Grouting: to consolidate the hearting over the two intermediate piers in particular.

The Execution

At an early stage in the drilling of the minipiles from deck level it became clear that the open and variable hearting (plus more recent concrete) necessitated pregrouting to allow progress to be made without excessive delays, or damage by the water flush. 48 Nr 114mm diameter holes, drilled at pile locations, and totalling 290m, accepted 44 tonnes of sand-cement grout of W = 0.6. Thereafter a 220mm rotary duplex drilling system was used to install the piles, involving 660m of drilling and 324m of permanent steel liner. A 2:1 sand cement grout of W = 0.6 was used, giving a target 28 day strength of $30N/mm^2$. All drilling was executed by a diesel powered hydraulic track mounted rig, and grouting by a colloidal mixer/pump unit.

The 10 tie bars (totalling 60m) consisted of 30mm diameter medium tensile bars, sheradised and encased with a PVC sheath, and grouted into the 75mm horizontal boreholes with a W = 0.45 neat cement grout. A frame mounted rotary drill rig was used with diamond drilling bits. The 600mm square pattress plates were also carefully protected against corrosion.

Stitching was conducted generally in the upstream parapet, but with some on the downstream side. The 16mm dia bars were 2m long and a total of 24 were installed to secure the parapet and spandrel wall.

Pointing was conducted in all areas where the original mortar was missing, loose or soft. It involved removal of debris and vegetation to at least 25mm depth before jetting and final cleaning. The $156m^2$ finally executed covered the sides and ends of piers up to and including the arch springing and all joints to arch soffit and spandrel walls.

A total of 70 Nr holes were hand drilled between 500 and 1000mm deep to permit the injection of 19 tonnes of neat cement grout to complete the consolidation treatment. Pressures of up to 2 bars were used.

Finally some minor alterations to the service ducts in the structure were conducted together with regrading of the river bed for flood alleviation purposes.

At the conclusion of the works, the Engineer, Powys County Council accepted the execution of the remedial works and lifted the traffic weight restriction thereby returning the bridge to normal operating standards.

2.4 Railway Embankments, London

The Sites

Many of the embankments in the open running sections of the London Underground Railway system, were constructed during the latter part of the 19th century. Few details are available of the methods of construction, the nature of the fill material, or its degree of compaction, although it may be assumed that cut and fill methods were used, involving clay excavated from the cuttings. In some cases fill material was placed directly onto the pre existing grass surface at formation level without prior preparation. Embankments usually have a clay core with various thicknesses of ash on the shoulders.

The Problem

With time and under modern loading conditions, a gradual spreading and slipping of the embankments occurs, leading to settlement of the line. Much of the remedial work involved (i.e. tipping and placing ballast beneath the sleepers) is conducted by the Authorities' workforces, invariably during short possession times. However there are other occasions when movement is excessive and progressive and so the costs of conventional maintenance are very high.

The Solution

Consolidation of the embankment with suitably arrayed fans of grout holes has been conducted (Figure 7). The holes are designed to extend beyond the shallow slip circle surfaces anticipated.

The Execution

Holes were spaced conventionally at 1.5m centres, with holes in adjacent rows staggered. The distance between rows varied from 1.5 to 3m dependent on ground conditions. The two rows closest to the track were raked at 30-40° to the vertical, depending on the slope of the embankment and were 3.5-4m long. The top row was placed at least 2m clear of the outside running rail for safety reasons and had maximum hole lengths of 5m.

Grouting started on the bottom row of holes, with Primaries lanced and grouted at 3m centres. Upon completion of an appropriate length (say 50m), the work transferred to the Primaries of the next row, and so on.

The "lances" were 12.5mm i.d. steel pipes, sealed by a lost rivet, driven to the required depth by air powered percussive hammer. Progressive extraction of each lance during injection was accomplished either by hand or by hydraulic jack, depending on ground resistance. The typical "mix" was 3:1 PFA (or sand):OPC at 0.35 WSR, mixed in a colloidal mixer and injected by Colmono (pumping distances < 200m) or piston (pumping distances 200-1000m) pumps. Whilst some of the lower Primary holes consumed over $5m^3$ of grout, overall contract averages were about $0.7m^3$ per hole. Where takes were particularly and uniformly low, the pfa or sand content was reduced.

Special efforts were made to use "quiet" plant, given the urban environment of much of the site, whilst the repair proceeded with minimum removal of, or damage to, vegetation on or adjacent to the embankment.

Over the years, many kilometres of embankments have been successfully stabilised by this traditional and simple grouting approach.

2.5 Retaining Wall at Denholme Clough, Yorkshire

The Site

The 125m run of near vertical drystone retaining wall (Photograph 3) at Denholme Clough, near Bradford, is typical of many thousands of kilometres of structures in West Yorkshire and neighbouring counties, built around 100-200 years ago. The wall is up to 3m high, retains soil and fill, and bears on weathered sandstone bedrock (Figure 8).

The Problem

The progressive failure and collapse of such walls represents a major problem to the highway authorities since there is (i) physical danger resulting from a collapse, (ii) disruption to traffic during clearance and rebuilding works and (iii) high maintenance costs associated, often of an ongoing nature.

In this particular example, the wall had actually collapsed in places whilst in other areas

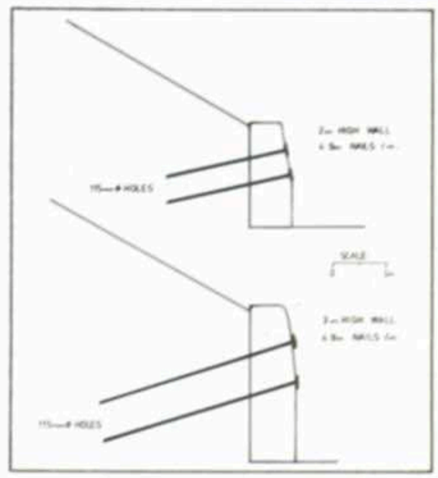


Fig 9. Design Cross Sections for Nails for 2m and 3m High Wall Sections, Denholme Clough

Photograph 3. View of part of the retaining wall, showing deformation and general delapidation. Denholme Clough, Yorkshire

Photograph 4. View of wall after cleaning, partial rebuilding, guniting and nailing, and before construction of new masonry face. Denholme Clough

there was significant bulging indicative of imminent failure.

The Solution

Over the last 15 years or so, soil slopes and excavations have been stabilised throughout Europe by the technique of soil nailing ("Clouage"). Soil nails are basically grouted subhorizontal minipiles (or passive dowells) installed across potential slip planes to increase adequately the resistance to movement and hence ensure the stability of the structure. They mobilise the properties and strength of the existing materials in a manner akin to the reinforced earth method for new structures. The design of such systems in terms of specifying nail intensity, length, orientation and individual capacity is complex and relies on a fundamental analysis of the potential failure mechanisms, as related to the geotechnical and physical parameters of the slope to be supported.

Bearing in mind that the facing of this wall had deformed extensively, failed locally and weathered generally, the following sequence of operations was agreed

- Clear vegetation
- Install drain holes to maintain the natural drainage of excess water from behind the wall
- Repair/rebuild facing of walls
- Install nails, according to design summarised in Figure 9, and conduct individual pullout tests as prudent
- Build new masonry facing

The works were conducted as an engineering joint venture between the Client and the author's former Company, on a section which would otherwise have had to be demolished and rebuilt at greater cost.

The Execution (Photograph 4)

- **Vegetation Clearance.** Vegetation was generally removed, although in places it appeared to be contributing positively to the delicate state of equilibrium of the wall and so the root network was left in situ. A shallow footing trench was also excavated at this time (Figure 10).
- **Drainage.** Firstly a series of 50mm temporary drain holes were drilled 300mm above the road level at 2m horizontal spacings. After nail installation, two rows of permanent 115mm dia drain holes were drilled 300mm and 1500mm above nail level at 3m horizontal spacings.
- **Repairing Wall Facing.** Those sections of wall in a state of collapse were rebuilt in a single skin brickwork with strengthening piers. The voids behind were backfilled with a sand cement grout plus Conbex 653 additive to restrict flow. A layer of light D49 steel mesh was fixed to the wall which was sprayed with a 50mm thickness of gunite to hold the component stones in place during the subsequent nailing. Vertical construction joints were located at 15m centres.
- **Nailing.** A track mounted drilling rig was then used to drill the nail holes according to the designed pattern. The nail reinforcement consisted of a 16mm HY bar with threaded proximal end. The grout was a neat cement mix of W = 0.45, prepared in a colloidal mixer. A corrugated PVC sleeve was placed through the upper section of each hole to restrict lateral travel of grout in the masonry. After setting of the grout, a headplate was fitted and a nominal stress applied.

  Pull out tests were conducted on three test nails. These gave excellent results, confirming that individual ultimate capacities were comfortably in excess of the required working conditions.
- **New Masonry Face.** The new facing was then built and the void between existing and new fully grouted.

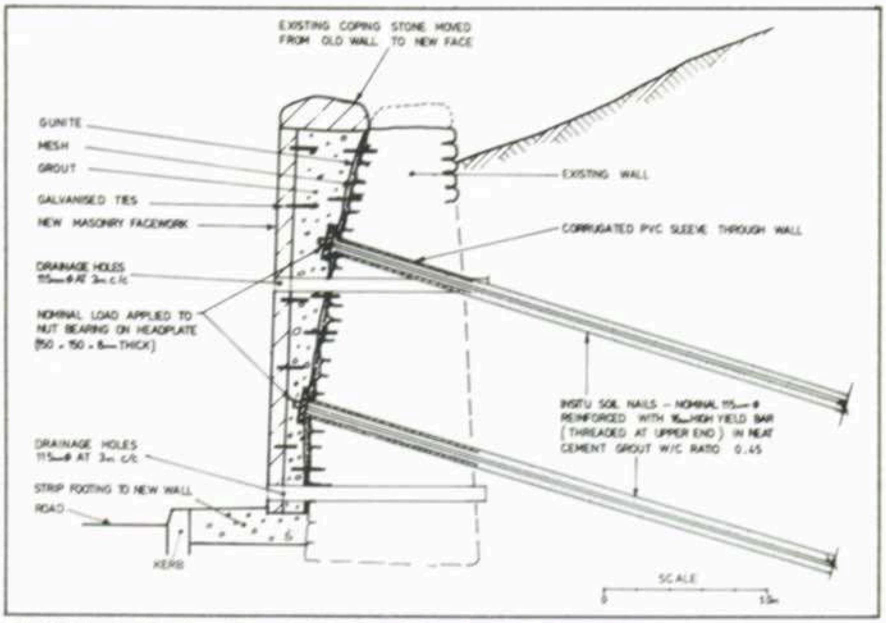


Fig 10. Typical Section of Repaired Wall (including backfill behind rebuilt section), Denholme Clough

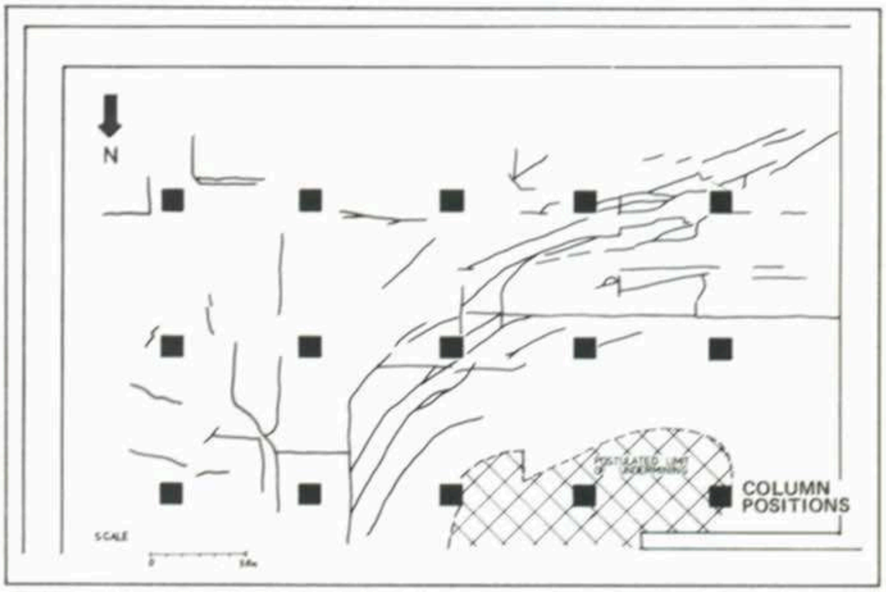


Fig 11. Plan of major fissures in reinforced concrete base slab, underground cistern, Rod El Farag Water Treatment Plant, Cairo

It was concluded that from commercial and technical view points the technique is attractive and viable. The actual details may be varied from case to case e.g. if the old facing must be retained (i.e. without new brickwork or guniting) then it would be possible to conceal the nails behind blocks carefully and temporarily removed.

The nature of the repair is considered to be particularly apposite in applications where the installation of post tensioned anchors could upset very delicate equilibria, or where subvertical reticulate root pile arrays are impractical or very expensive to install. Both artificial slopes/retaining walls and natural slopes are prime potential applications for this very positive and practical technique, as are road and railway embankments threatened by slippage.

3. BASE SLABS

3.1 Rod El Farag Water Plant, Cairo

The Site

As part of a huge programme to renovate and expand the water and sewage infrastructure in Cairo, the Rod El Farag Water Plant is being built under a US Aid Grant for the client, the General Organisation for the Greater Cairo Water Supply. The vast construction is situated on the east bank of the River Nile in the district of Shoubra, to the north of central Cairo.

One of the major elements of the project is the main underground water cistern, the base of which is a heavily reinforced concrete slab, from 1 to 1.8m thick. This is cast on relatively impermeable materials varying from natural silty clay or sand to a compacted backfill of clay, sand and gravel.

The Problem

A portion of the filter foundation slab was found to have experienced undermining and associated cracking (Figure 11). Several weeks later, another smaller area of undermining was discovered west of the first area. It was believed that the damage to both areas occurred at about the same time and both were a direct result of a previously placed granular backfill being removed by an immediately adjacent dewatering system.

Quickly the Main Contractor carried out some preliminary remedial works: the major

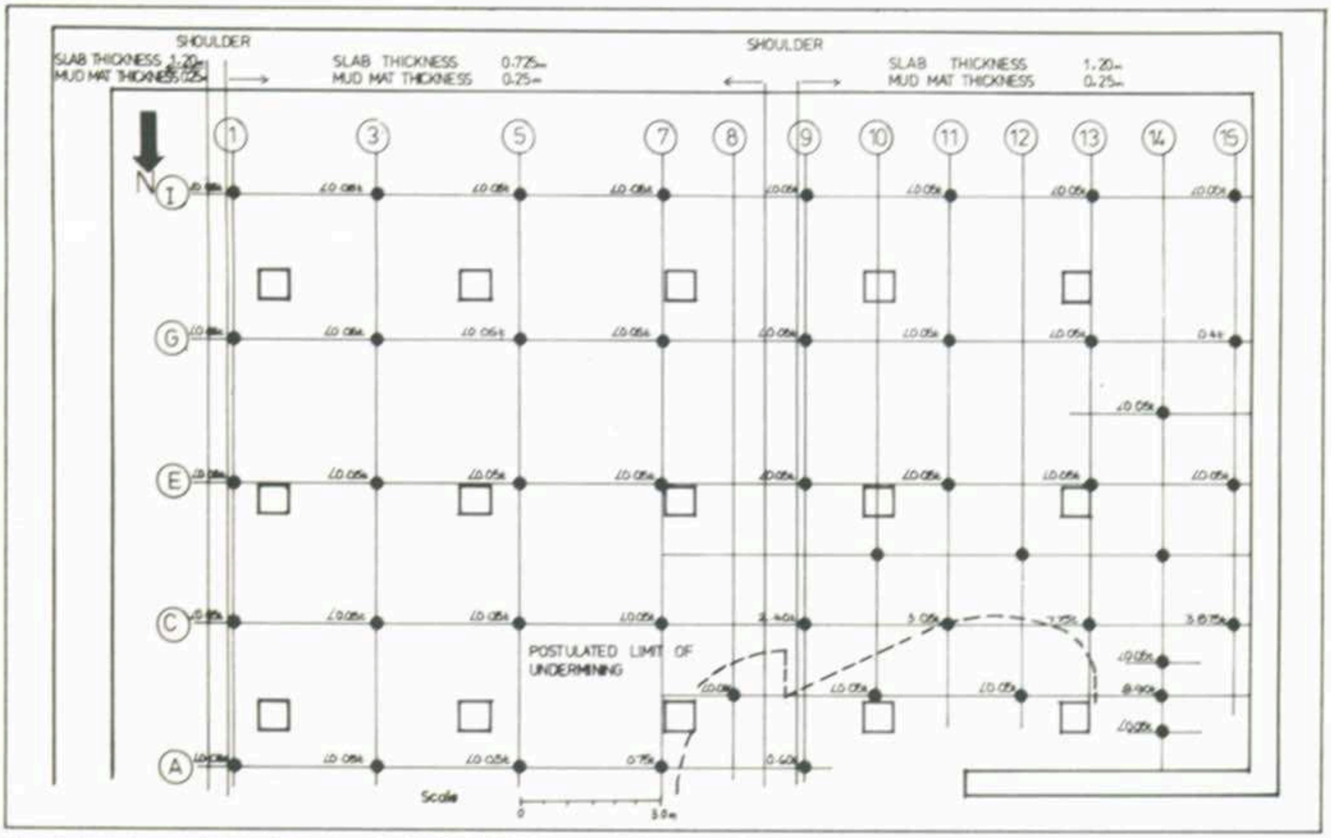


Fig 12. Underbase Drilling and Grouting Results, Rod El Farag Water Treatment Plant

underslab voids were filled from the open (north) side with concrete, and the direct influence of the adjacent dewatering system was eliminated. Thereafter, survey indicated a static structural condition, with a range of settlements in the main area of 25-30mm. However, voids of up to 100mm (typically 25mm) were suspected under the settled areas and so it was necessary to conduct further remedial work to complete the repair.

The Solution

The following procedure was agreed:

- firstly recreate reasonably uniform support to the foundation filter slab by pressure grouting the voids between slab and soil, with a penetrative cement based grout, and
- secondly restore the structural integrity and watertightness of the slab by pressure injection of structural epoxy into the fissures and construction joints.

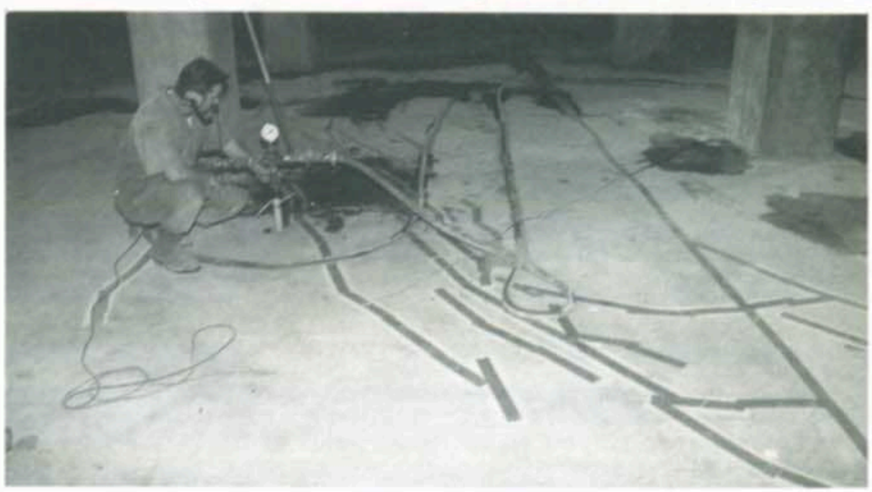
For practical reasons, the work was best conducted in the dry and so the piezometric level was maintained 1m below the slab base throughout the treatment.

It was not necessary to attempt consolidation or compaction of the backfill as the undermining was not caused by the removal of fines. It was also not required to slabjack[9] since the current position of the slab was not detrimental to its proper functioning, and as a result of the varied and complex structural stiffnesses involved, attempting to raise the slab could possibly have been harmful.

The Execution

- **Cleaning and Survey.** Clearing of the surface debris was followed by hosing and scrubbing the slab with water. A high pressure air and water spray was then used to remove all fine particles from the cracks, which proved to be far more intensive and frequent than had been foreseen. The cracks were then protected against further contamination by applying adhesive tape. The slab was then closely surveyed to establish accurately its attitude before grouting.
- **Underbase Grouting.** A primary 3m grid of 41 holes was established (Figure 12). In order to minimise damage to the heavily reinforced base slab, a rebar detection meter was used to adjust the location of each hole to reduce the requirement to drill major reinforcing elements. In addition other holes were slightly repositioned to provide adequate edge distances. Drilling through the concrete was accomplished with electric powered wheel mounted drilling rigs (assembled in situ) using 82mm dia. roller bits on NWY rods with water flush. Upon encountering steel, (usually in the lower reinforcing mat) the system was changed to using thin walled 56mm o.d. diamond core barrels to ease penetration and minimise structural disruption. After completion, each hole was thoroughly washed out and temporarily sealed with a wooden bung to keep out other construction debris. Hole depths varied from 1-1.8m and were occasionally continued for up to 0.9m into the fill material, for investigatory purposes. Drilling commenced in the NW corner of the main slab where undermining was anticipated as being most severe. Only 10 holes, all in this area, confirmed the anticipated voids under the slab. A "down the hole" Borescope was then used to inspect closely the slab/soil interface in key areas. These investigations all confirmed the existence of a significant void in the NW area.

The grouting station, consisting of a colloidal mixer, water measuring tank, agitator holding tank, and a double acting piston pump, was equipped with a return line to allow grout recirculation from the injection head.



Photograph 5. Underslab grouting underway with return line injection system. Note adhesive tapes protecting the cracks in the concrete surface. Rod El Farag, Cairo

The head incorporated three valves and a pressure gauge/diaphragm unit, in order to control precisely the injection pressure. The grout injection crew working on the slab were equipped with radio headsets to enable direct communication with the grout mixing crew on the surface, both to control the works effectively and to determine the quantity of grout injected into each borehole (Photograph 5).

The grout mix employed was a neat cement grout of W = 0.4-0.5, with the addition of Colplus W plasticiser in the proportion of 0.45% by weight of cement. Grouting pressures were generally limited to 1 bar. A hole was considered to be complete when it would hold this injection pressure for 3 minutes.

Injection commenced in the area of known voids, and as boreholes came up to pressure the injection face was extended radially away from the voided area. In every borehole, grout was injected via a tremie pipe placed to the bottom of the borehole and grout was pumped into the open hole until the grout level showed within the borehole. At that point the injection head was installed and pressure grouting commenced.

Close quality assurance was maintained throughout, with periodic checks on the fluid grout properties (Bleed, fluidity and S.G.), supplementing the grout cube data.

A total of $12m^3$ was injected into the 41 boreholes (Figure 12) with the high takes and interconnections in the NW area confirming the scale of the void. Eight secondary holes were therefore drilled and injected in this area but only one (B14) took anything other than theoretical hole volume ($7m^3$ total). To finally seal this particular area, two adjacent Tertiaries were drilled and both yielded continuous grout cores and accepted no grout. The total drilling involved in the 51 holes was 76m.

At this stage it was accepted that the object of restoring effective slab/soil support had been comprehensively achieved.

Crack Sealing. Firstly the 51 underbase grout holes were chiselled out to a depth of 100mm, blown clear, painted with a high strength epoxy mortar adhesive and sealed with a high strength non shrink mortar. The tapes were then removed from the cracks which were again blown clean. 12-15mm dia. holes were then drilled at 150-300mm intervals in the cracks to a depth of 50mm and plastic injection pipes glued in with a rapid epoxy adhesive. Thereafter Permagile Salmon "Perm Inject" epoxy resin grout was injected using a hand pump at pressures of up to 11 bars. Despite its very low viscosity and the high pressures used, approximately 50% of the surface cracks accepted no resin, thereby proving themselves to be very shallow and structurally insignificant. After, the pipes were removed and the holes backfilled with more resin grout. The construction joints which had been injected with resin were channelled out and backfilled with a joint sealant.

Again the works proved entirely satisfactory and were fully accepted following a full scale test involving filling the cistern with water to simulate working conditions.

3.2 Tarbela Dam, Pakistan

The Site

The Tarbela Dam Complex is one of the world's greatest water resource developments and is located 70km northwest of Islamabad, Pakistan (Figure 13). As has been well documented in the world press over the past decade, major remedial works have been conducted across the site, but principally on the Main Embankment Dam, the Service Spillway and the Auxiliary Spillway.

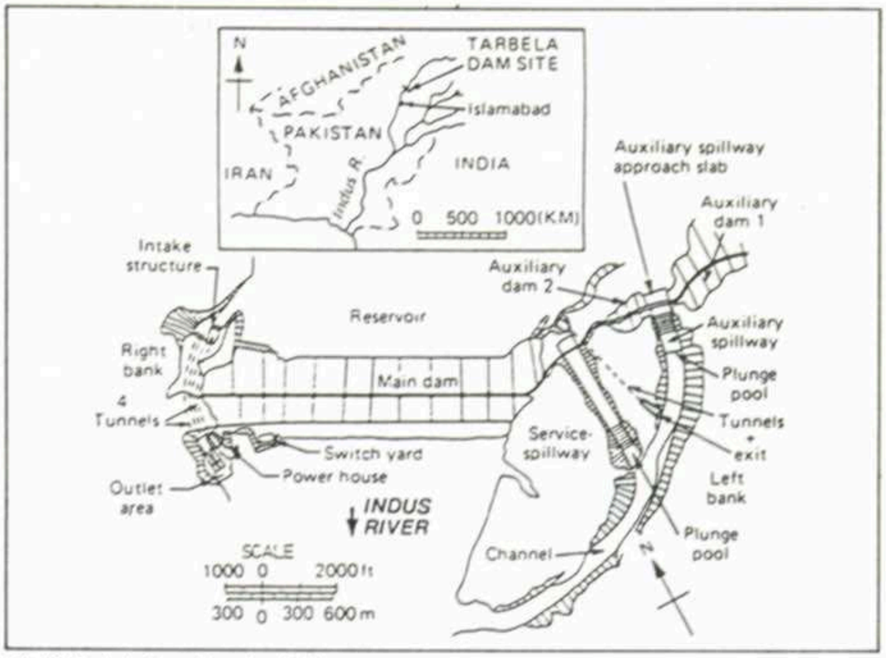


Fig 13. Tarbela Dam Complex, Pakistan

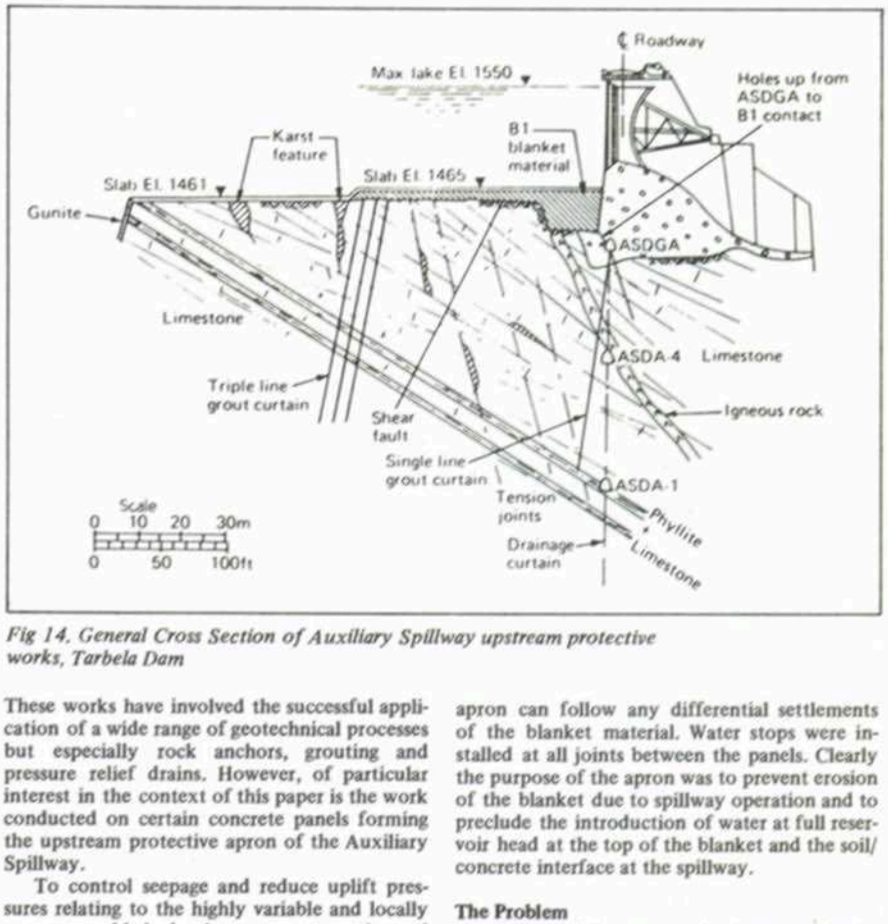


Fig 14. General Cross Section of Auxiliary Spillway upstream protective works, Tarbela Dam

These works have involved the successful application of a wide range of geotechnical processes but especially rock anchors, grouting and pressure relief drains. However, of particular interest in the context of this paper is the work conducted on certain concrete panels forming the upstream protective apron of the Auxiliary Spillway.

To control seepage and reduce uplift pressures relating to the highly variable and locally very permeable bedrock, two grout curtains and an extensive drainage system were constructed[10]. The upstream triple line curtain and the spillway structure itself were then linked by a 68m long blanket of low permeability fill laid on the excavated rock surface (Figure 14). The purpose of the blanket is to lengthen the potential seepage path so as to lower hydraulic gradients (and thus uplift pressures) under the spillway.

The blanket is capped with a 0.3-0.6m thick concrete apron (Photograph 6), constructed in panels typically 15 x 11m in plan except at locations permitting articulation so that the apron can follow any differential settlements of the blanket material. Water stops were installed at all joints between the panels. Clearly the purpose of the apron was to prevent erosion of the blanket due to spillway operation and to preclude the introduction of water at full reservoir head at the top of the blanket and the soil/concrete interface at the spillway.

The Problem

Monitoring of the structure/rock system for the few years after first impounding indicated that the blanket was not acting in an acceptably efficient manner. Further investigations led to the conclusion that a major contributory factor was the concrete apron. It was reasoned that the blanket had settled under reservoir water pressure but that the overlying apron was "hanging up" at the spillway structure, leading to the formation of a void to which reservoir water had access from both sides. This water had washed out areas of fill leading to irregular settlements of certain panels by 30-180mm at

Photograph 6. Packers being placed in holes drilled in concrete apron, adjacent to upstream face of Auxilliary Spillway, Tarbela Dam, Pakistan

distances of 6-18m from the headworks. There was therefore also concern that the differential movements could have given rise to voids beneath "hanging" slabs and could be distressing the rubber waterstops to the point of failure, leading to the complete ineffectiveness of the crucial protective apron.

The Solution

It was decided to inject cement based grouts under the panels, both to fill possible voids, and also, by using slightly higher pressures, to jack up the slabs to an attitude akin to their original. This operation would also automatically check the integrity of the vital waterstops. After preliminary site tests, the following outline method was proposed.

- Drill holes at approximately 2m centres on a square grid through each slab and up to 300mm into the fill, carefully logging the presence of voids or loss of flush.
- Backfill holes encountering voids by pouring in a cement-bentonite mix, adding sand to the grout if the take and grout travel were substantial.
- After at least 24 hours, pressure grout to 2 bars with less viscous cement-bentonite mixes, lifting adjacent slabs in steps limited to 25mm to prevent damage to the waterstops. Careful frequent surface surveys to be conducted to monitor closely the slab movements.
- Make final fine adjustments to levels, and backfill drill holes with a compacted dry mix of cement, water and sand, topped with 50mm of an epoxy cement and sand mix.

Regarding the design of the grout, the prime criterion was that, when set, it should have deformation characteristics compatible with the fill material i.e. E = 70 – 130N/mm². In addition, the mix did not require high strength, but had to be stable (i.e. minimal bleed capacity) to ensure intimate concrete-fill contact after setting. The fluid mix had also to be sufficiently viscous to have restricted, controllable travel during injection.

Earlier grouting operations on site had led to the development of cement, or cement-sand, mixes with relatively large amounts of local bentonite, and this general outline was again adopted initially. Back-fill mixes were thus of the form cement (c): bentonite (b): sand (s): water (w) – 20(c): 40(b): 0 to 20(s): 100(w), and pressure grouting was foreseen with a c:b:w mix of 25:30:100 ("A/2 mix").

The Execution

A total of 1302 holes, each 75mm diameter, was drilled with air flush and pneumatic track rig in the 5130m² area to be treated (average number – 25 per slab: range 12 to 36, depending on slab size). None were drilled within 900mm of a panel edge, to avoid damage to the concrete. Voids were logged beneath the apron primarily under the thin articulation panels above the shallowing of the blanket – the zone of maximum depression. The maximum void depth was 276mm (Panel 68) with other voids of up to 200mm noted towards the spillway. Elsewhere the general limit was 50mm.

Grouts were mixed in high speed mixers and pumped to the mechanical packers (set towards the slab invert) by a circulation line arrangement.

Early grouting results showed that both void fill and contact grouting could be achieved in the same phase of grouting and so backfilling was omitted.

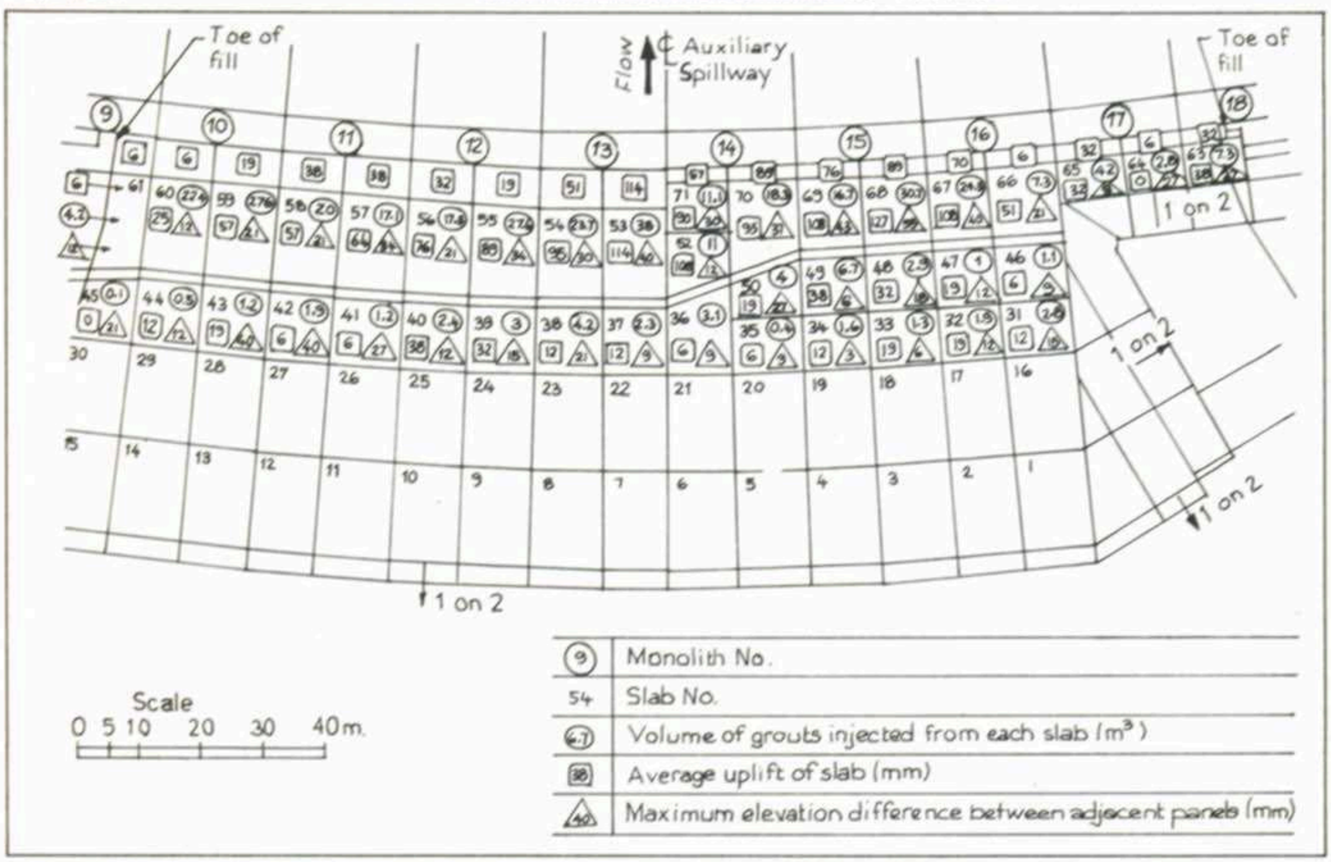


Fig 15. Grout consumptions, average uplift and relative attitudes of panels, after slabjacking, Tarbela Dam

The pressure grouting commenced under Panel 55 (Figure 15), and progressed from left to right and in a general way towards the headworks. The lifting of the lower slabs was initiated from one of the peripheral holes on one edge only, and continued across the slab. As the grout migrated, wooden plugs were driven into the holes from which grout emerged. Practice was to permit travel of up to 6m before further packers were then installed nearer to the travelling face of the grout and injection transferred to them. When approximately two-thirds of the slab had been so treated, the grout line was returned to the first packer to attempt to lift that part of the slab.

An initial phase of lifting was conducted on a two shift per day basis, treating each of the lowermost 32 slabs at least once. During this period, experience was gained which proved of great value in the later stages of "fine adjustments". For example, although the mix with 70 litres of water had proved very penetrative, it was, by the same token, too fluid to yield controlled localised uplifts. Progressively thicker mixes were tried, within the overall concept of the design requirements, until a "52 litre" mix was found to be the most viscous that could be mixed and pumped given the site conditions (more than 100m of 20mm i.d. steel grout line from pump to injection point).

The second phase of pressure grouting, including the levelling, was conducted during the day shift only, in order to maximise control of operations. As familiarity with grout travels and effects grew, lifting was commenced from the lower one-third of panels, often via up to three simultaneous injection points. This phase concluded with the injection of the "70 litre" mix under Panels 31 to 43 to ensure the filling of any interstices left empty by uplifting with the more viscous mixes. No grout entry to the drains beneath the spillway structure was recorded at any time.

The pressures required to initiate slab "floating" varied considerably, with the size of the slab and the joint friction effect being critical factors. Pressures up to 7 bars were required – briefly – in certain cases, although this commonly dropped to well below the limiting 2 bars pressure during continued injection.

Whilst the slabs were all below the required grade, the problem of unintentionally lifting adjacent slabs was not critical. However, as slabs approached level, grout travel and edge friction rendered the achievement of absolute levels extremely difficult. In particular, the intended raising of the narrow panels in the articulation sections proved almost impossible to effect. Accepting that more viscous grouts were not feasible, the best practical compromise was to make the final levelling goal the securing of a smooth profile across the whole area. Caution was also required to forestall slabs from undue bowing, although in a few panels the uneven loading caused by the variable friction effects initiated minor surface cracking.

Joint leakage occurred only between Panels 44 and 45, where the waterstop was subsequently removed and replaced. This verified the continued integrity of all the other waterstops.

The volume of grouts injected under such panel is shown in Figure 15: in total a net volume of 375m³. Tests confirmed that the set properties of the mixes matched the characteristics of the fill.

A final close survey proved that the slab-jacking had reduced the distortion of the individual panels, and had restored a uniformity of elevation.

After completion of the slab-jacking, water and grout injection tests were conducted on the lower contact of the fill and concrete, through 20 drill holes angled up from ASDGA (Figure 14). The majority of the holes were completely water-tight and only the lateral monoliths (5 and 22) had any grout consumptions (58 and 250 litres of "A/2 mix" respectively). No open water paths thus existed along this contact. Furthermore, piezometers installed in ASDGA after the slab-jacking recorded zero pressure under full reservoir conditions, and so confirmed the efficiency of the front seal.

Finally, excavation was conducted during the next low reservoir period of certain of the thin strips forming the upstream articulation and which had proved impossible to accurately slab-jack. Grout was found in intimate contact with the underside of the slab, and no void between B1 and grout could be found.

4. TUNNELS, CULVERTS AND SEWERS

4.1 Pipeline, Alexandria

The Site

As part of the works to upgrade the sewage system of the city of Alexandria, ARE, many kilometres of new pipelines are being laid. Typical of the construction, 3.5m long sections of reinforced concrete sections, 2.75m in diameter (Figure 16) are being laid in trenches, excavated through the surrounding cohesive materials by use of flanking sheet piles (later extracted), in the standard cut and cover system. This gravity sewer is bedded on crushed limestone and gravel, surrounded by a backfill of rounded gravel/coarse sand material almost to the crown, and compacted fine sand above, to ground level. It is designed to operate at a combined flow rate of 953mld, at velocities up to 1.86m/sec.

The Problem

Due to certain operational problems during the construction of a particular 280m long stretch of pipeline, most of the connecting joints were damaged to the extent that major water ingress occurred due to flow of groundwater in through the surrounding highly permeable backfill. Access manholes were flooded to a level of 1m below ground surface.

Less severely damaged joints also gave inflows ranging from minor seepages to steady leakage. It was therefore clear that this situation was totally unacceptable in that it would prevent the structure acting efficiently by permitting escape of sewage. Various schemes were considered to remedy the situation, the most extreme being to exhume the defective section and replace it. However, a proposal based on grouting proved to be far more attractive commercially and was also preferable in terms of programme and logistics – the upheaval of conducting another period of excavation would not have been acceptable in the congested urban area. In addition, the relatively high strength microconcrete into which the backfill should be transformed by grouting would act to reinforce the "strength" of the pipeline structure and so

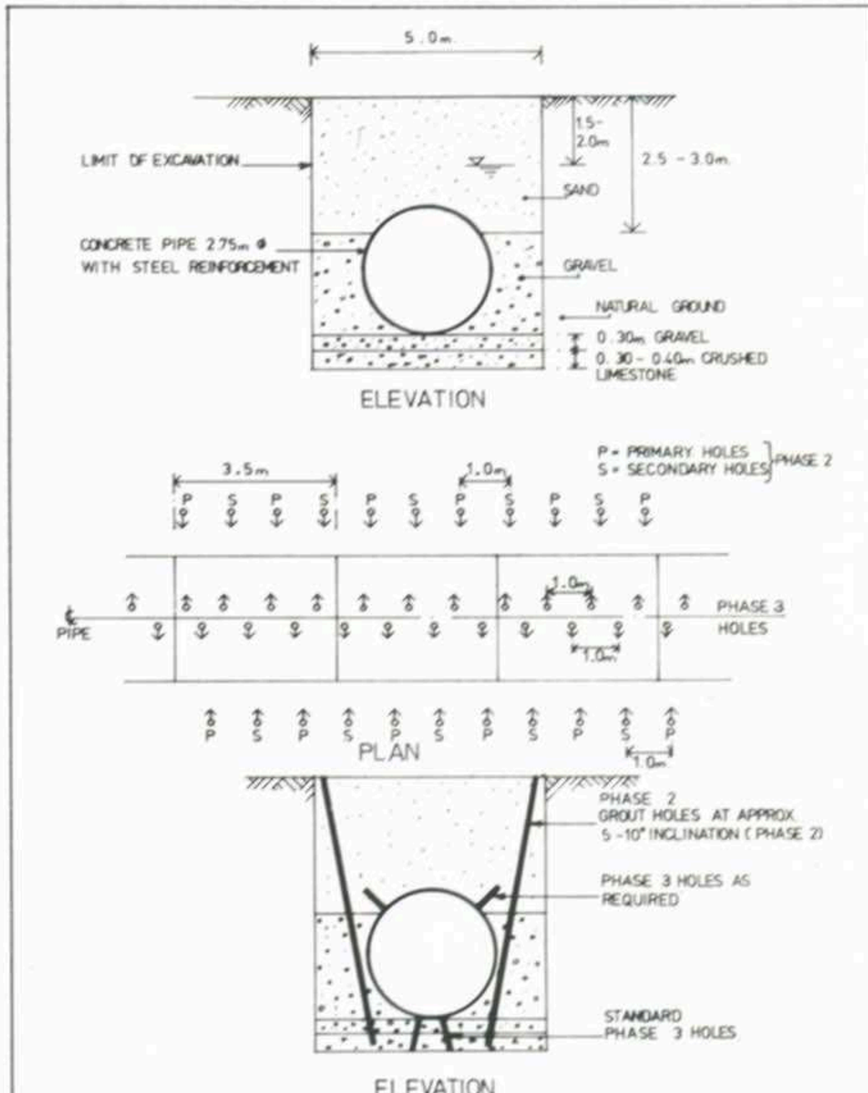


Fig 16. Details of pipelay and grout hole arrangement, sewer, Alexandria pipeline, A.R.E.

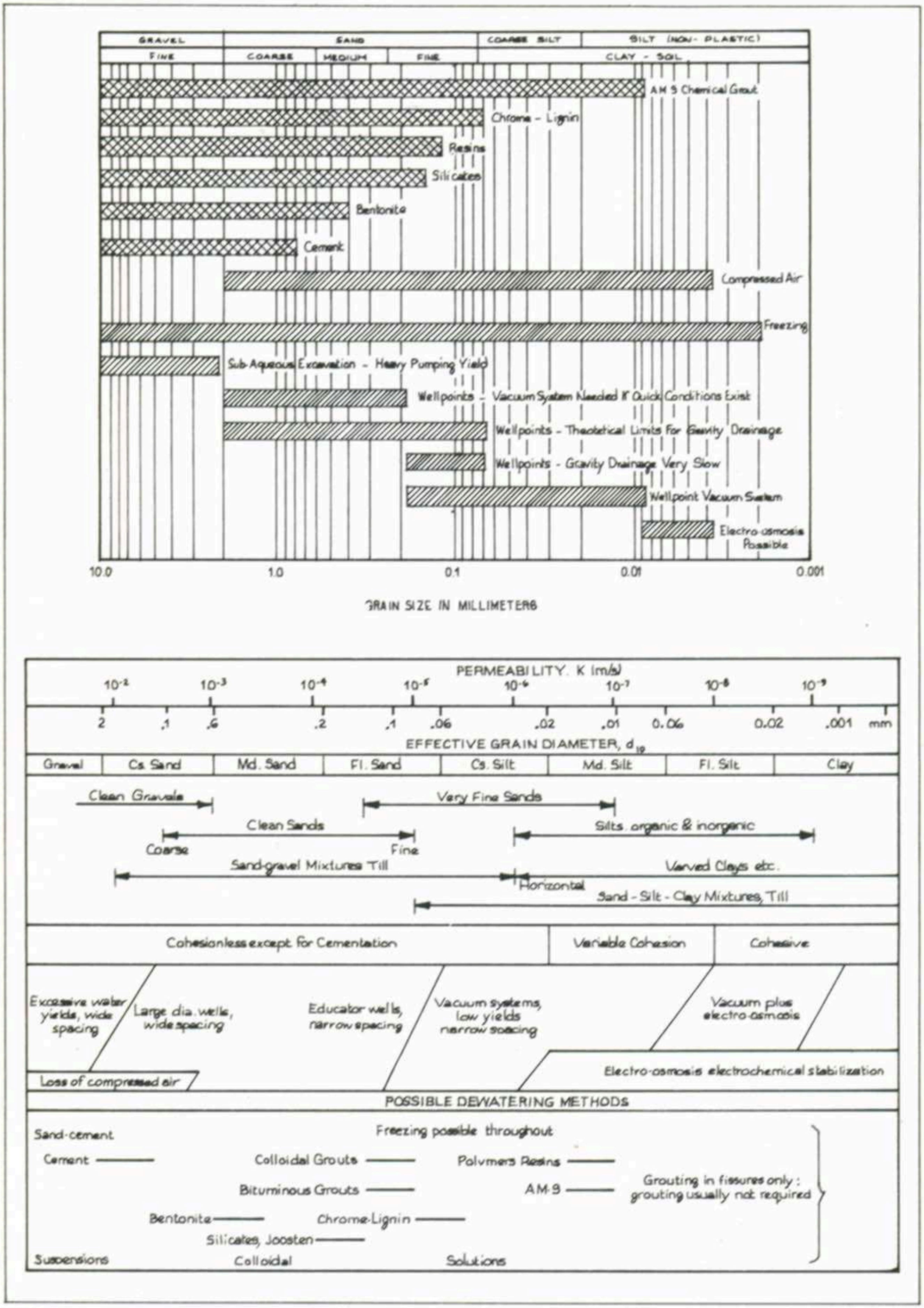


Fig 17. General theoretical limits of grout permeation[11]

help resist stresses imposed during service which would cause tendencies to sag or hog.

The Solution

Classification data on the coarse backfill accounting for most of the surround to the pipeline indicated that it would readily accept a cementitious grout (Figure 17). Although the nature of the upper sand would probably preclude permeation, it was expected that the grout would travel around the whole section of the pipeline and so seal off leaks in the crown also by forming a "skin".

The following sequence was foreseen:

- "Coarse" caulking. Following an internal inspection and assessment of the structure on a joint by joint basis, the major leakage areas were to be blocked using a variety of bungs, rags etc. No attempt was to be made to try to seal all leakages, as 1) the philosophy of the repair was grouting of the encasing fill and 2) observation of the reduction in joint seepage during the grouting would indicate the progress and efficiency of the process.
- Grouting of the Backfill. It is common to have to treat ground surrounding sewers by injection from within the structure[7]. However in this instance, the depth of the sewer was small, the surface access good, and the possible working conditions within the structure prohibitive. Thus it was logical to conduct the work from the surface, as far as possible. In order to ensure uniform penetration bearing in mind the shallowness of the treatment and the practical limits on the grouting pressures which could be mobilised safely, holes were designed at close centres – 1m – along both sides of the pipeline. Adjacent holes were designated Primaries and Secondaries (and executed in that sequence) in order to gauge the progress of the works by reference to pressure – volume – time injection characteristics.
- Check Grouting. As a final check, it was proposed to drill and inject a number of holes (2 at each joint) from within the pipeline. Thereafter, if any remnant seepage remained, it was intended to execute a final sealing with a proprietory sealing mortar – a viable option after the bulk of the inflow and pressure had been eliminated by the grouting.

The Execution

Holes were drilled using a rotary drilling method with 92mm tricone bits with water flush, operating off electro hydraulic skid mounted drill rigs with rotary heads. Neat sulphate resisting cement grout of W = 0.45 was mixed in a high speed mixer (thus eliminating the need to dewater the gravel – the grout, being immiscible in water would displace upwards the interstitial fluids) and injected by air powered piston pumps. Injection was achieved by drilling to full depth (i.e. through the basal blinding layer, and then injecting through the rods during their steady withdrawal. To avoid possible structural damage but to promote grout travel, maximum pressures equivalent to 0.50 bars/m depth were set (to a maximum of 2 bars).

In total, 566 holes were drilled totalling 3396m, and 481m^3 grout injected. This corresponded to a notional voidage of about 18%. The average Primary take of 1.39m^3/hole had reduced to 0.31m^3/hole in the Secondaries, demonstrating clearly the influence of the grouting. More significantly, however, observations made from within the pipeline were extremely positive, with a progressive reduction in the flow through all joints. So successful was the work from the surface that only 13 joints required further grouting from within, and of those only 2 could not be completely sealed. (Seepage through the ungroutable silty sands at the crown was revealed to be the cause, and was remedied by patching.)

The work was to the complete satisfaction of the Engineer who approved the work unreservedly.

4.2 Glenlatterach Dam, Scotland

The Site

The 38m high Glenlatterach Reservoir embankment dam near Elgin in NE Scotland is constructed of local morainic sandy silty clayey gravel with cobbles, supporting a silty clay core (Figure 18). A concrete cut off wall is provided along the embankment centre line and provides a base for the clay core. The draw off culvert (Figure 19) is 1.83m high, 1.68m wide, approximately 167m long, and runs from the base of the 40m high valve tower to exit at the downstream portal in the centre of the natural valley slopes.

The Problem

The periodic reservoir inspection required under the Reservoir (Safety Provisions) Act 1930 showed unacceptable levels of leakage drawing in silt and clay fines in the valve tower and draw off culvert, particularly through the culvert construction joints. Flows through individual joints ranged from slight seepage to about 400 litres/hour. The work could only be conducted under normal service conditions and so the treatment had to be executed from inside the concrete structures.

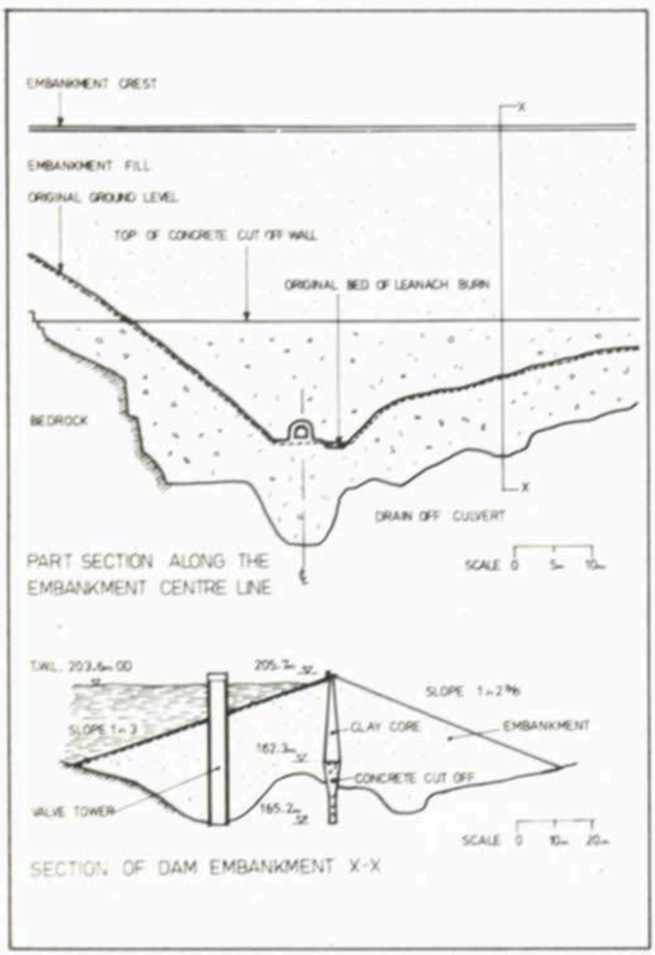


Fig 18. Diagrammatic representation of Glenlatterach Dam, Scotland

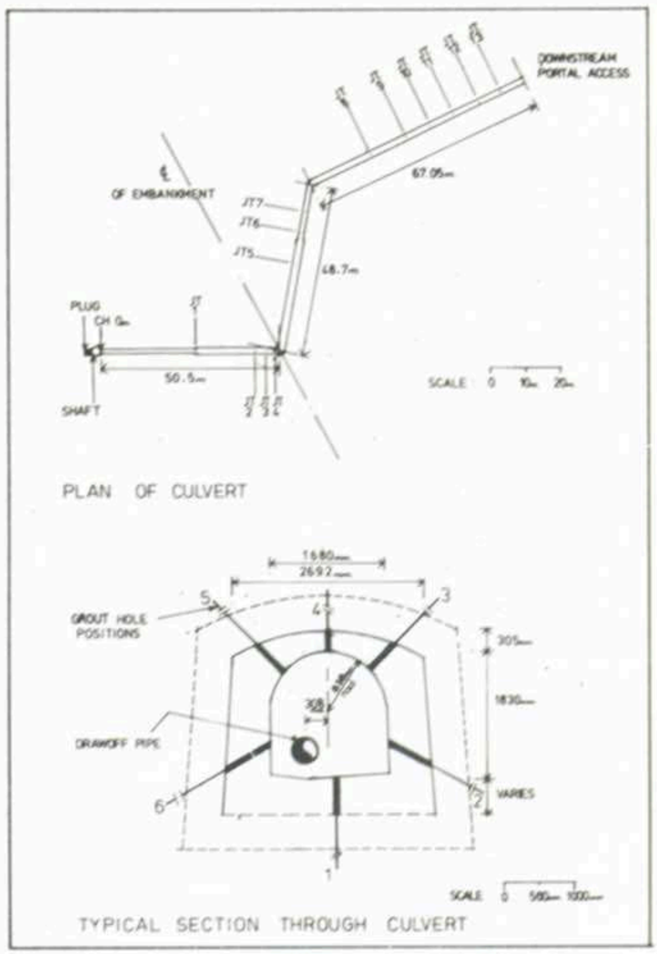


Fig 19. Details of drawoff culvert, Glenlatterach Dam, Scotland

Photograph 7. Hand held drilling equipment being used for grout holes in the culvert of Glenlatterach Dam, Scotland

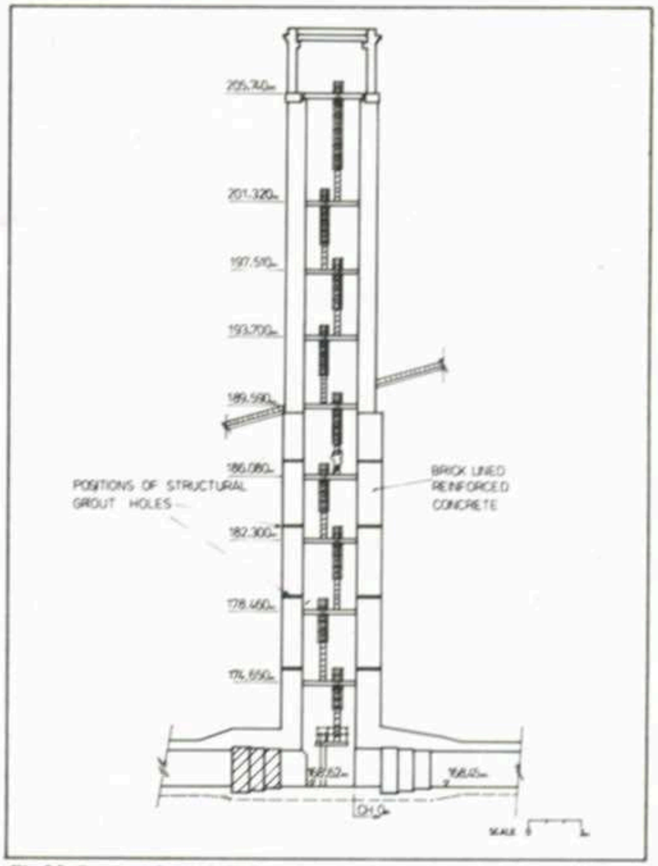


Fig 20. Section through Valve Tower, Glenlatterach Dam, Scotland

The Solution

It was decided to seal the joints, followed by systematic drilling and grouting along the culvert and valve tower to locate and fill any voids which may have been formed by the removal of fines and to consolidate the embankment where found necessary.

The programme incorporated the following phases:

- Joint sealing in the culvert
- Intermediate grouting, comprising
  - a) Cut off grouting
  - b) Contact grouting downstream of the cut off
- Consolidation grouting of embankment upstream of the cut off
- Valve tower works
- Site investigation and instrumentation of the dam embankment (not described herein)

The Execution

Drilling of the 40mm diameter holes was conducted with air powered rotary percussive hand held drills (Photograph 7). Grout, mixed in a colloidal mixer/pump unit, was passed to a holding tank located on a platform at the base of the lower. From here, it was distributed to the injection point with a piston pump.

- **Joint Sealing.** Thirteen joints requiring sealing represented phase 1. After being photographed, they were cleaned, dressed and caulked with OPC and accelerator. Joints with very high inflows were first drilled at the highest point of leakage and fitted with a threaded pipe to direct away the water. Grout holes were inclined to intersect the plane of the joint and were generally 600mm long. Injection was conducted with a W = 0.67 grout, (plus Colplus plasticiser to improve flow properties), at up to 4 bars by hand pump. Second and third phases of injection with a phenolformaldehyde chemical grout were used to tighten up certain areas. Overall, 23 joints were treated successfully involving 177 holes and 187 injections (featuring 7 tonnes OPC and 165 litres of chemicals).
- **Intermediate Grouting**
  - a) *Cut off Grouting.* To locate a suspected seepage path near the cut off/culvert intersection, two rings of holes were drilled downstream (Ch 52 and 54m) with a third upstream (Ch 47m). Each ring had 6 holes drilled through the culvert concrete and 0.5m into the fill. Injections were made with W = 0.67 grout plus 5% bentonite (by weight of water) to minimise bleed and limit grout travel. Injections continued to 4 bars or 500Kg of take. The relatively high takes indicated substantial voidage and 30 additional holes were drilled, including 9 up to 2m into the weathered schistose bedrock. The total of 48 holes accepted over 11.8 tonnes of cement and had evidently located and filled significant voids.
  - b) *Contact Grouting.* It had been postulated that the silt wash in had created a void between the outside of the culvert and the surrounding fill. Downstream of the cut off, for a length of 90m, a pattern of 3 holes was drilled at 4m intervals (Figure 19) positions 1, 3 and 5 alternating with positions 2, 4 and 6 on adjacent rings). The same mix was used, to a 500Kg limit and pressures of up to 4 bars. High average primary grout consumptions (210Kg/hole) necessitated secondary (186Kg/hole) and tertiary (147Kg/hole) treatments with cement whilst final tightening involved the phenol grout. In total 248 holes were drilled and almost 39 tonnes of cement injected in order to complete the treatment.
- **Consolidation Grouting.** An initial phase of permeability testing indicated a generally impermeable fill. Grouting tests were then conducted. Holes were drilled to 6m and grouted in ascending 2m stages with various cement-bentonite mixes of W = 2.0 to 0.67. A typical pattern of takes was

Stage Depth	Cement
0-4m	43Kg
4-2m	113Kg
0-2m	256Kg

The tests also showed that holes up to 3m long in positions 1 and 6 (Figure 19) gave valuable information on the rock profile around the culvert and concluded that grout be confined to the area immediately around the culvert concrete. So, rings of 6 holes were drilled to a depth of 3m, and grouted as in the test. Four phases of treatment were necessary, with 82% of the total of 250 holes being Primaries and absorbing 94% of the total cement injected (62.6 tonnes). Average hole takes ranged from 265Kg (Primary) to 73Kg (Secondary) and 54Kg (Quarternary).

- **Valve Tower** (Figure 20). The 40m high tower of diameter 3m had its lower 20m brick-lined. Grouting to seal leaks was concentrated on areas of salt deposits and points where steel was cast into concrete and brickwork. Ten holes were drilled to the fill at each primary platform level, from the base upwards and averaged 1.6m long. A grout of W = 0.67 with 5% bentonite was used to a pressure equivalent to Hydrostatic head x 0.1 + 0.5 bars. The average grout take was 45Kg (max 150Kg). A ring of 20 holes was drilled at the highest level of treatment (187.6m i.e. still in fill) and injected to form a "collar". These holes averaged 195Kg. Four secondary rings of 10 holes were then drilled at intermediate levels, followed by two rings at the base of the shaft. Overall, 114 holes were drilled and 17.4 tonnes of grout injected in order to achieve the final level of acceptability.

4.3 Summit Tunnel, Yorkshire

The Site

With a length of 2582m, British Rail's Summit Tunnel passes beneath the Pennines and links the towns of Todmorden and Littleborough. The brick lined tunnel, approximately 6.55m high by 7.0m wide, was engineered by George Stephenson and opened in 1841. Most of the tunnel lining is made up of at least 6 separate skins of brickwork totalling 500mm thickness, but round the 14 Nr 3m diameter air shaft openings, the crown of the lining widens to 1m to help support the weight of the shaft lining.

The Problem

Just before Christmas 1984, a fully loaded petroleum tanker train was about half way through the tunnel when one of the 14 wagons became derailed. A severe fire started, which caused flames to leap dramatically over 50m into the air from the top of Shafts 8 (70m high) and 9 (91m high).

After the fire had burned out three days later, and the worst of the wreckage removed, a television camera scan survey of the tunnel confirmed predictably that the worst affected area was Shafts 8 and 9 and the intermediate tunnel section, although generally the tunnel had resisted well. In this region the brick lining had vitrified and the sleepers burned to ash. Temperatures had reached approximately 1200°C. There was also evidence of spalling and separation of some unbonded layers of brick. It was therefore suspected that the lining of this section of the tunnel and the shafts could be in a potentially unstable state. British Rail's Regional Civil Engineer appointed a specialist consultant, James Williamson & Partners, to investigate the structural stability of the damaged tunnel and design the remedial works.

Photograph 8. Air powered trackrig being used for grout and bolt-hole drilling, Summit Tunnel, Yorkshire

The Solution

The Consultant initially considered stabilising the shaft lining by installing a regular grid of bolts and dowels over the full length of the shafts to provide the necessary shear connection between the brickwork lining and the surrounding rock.

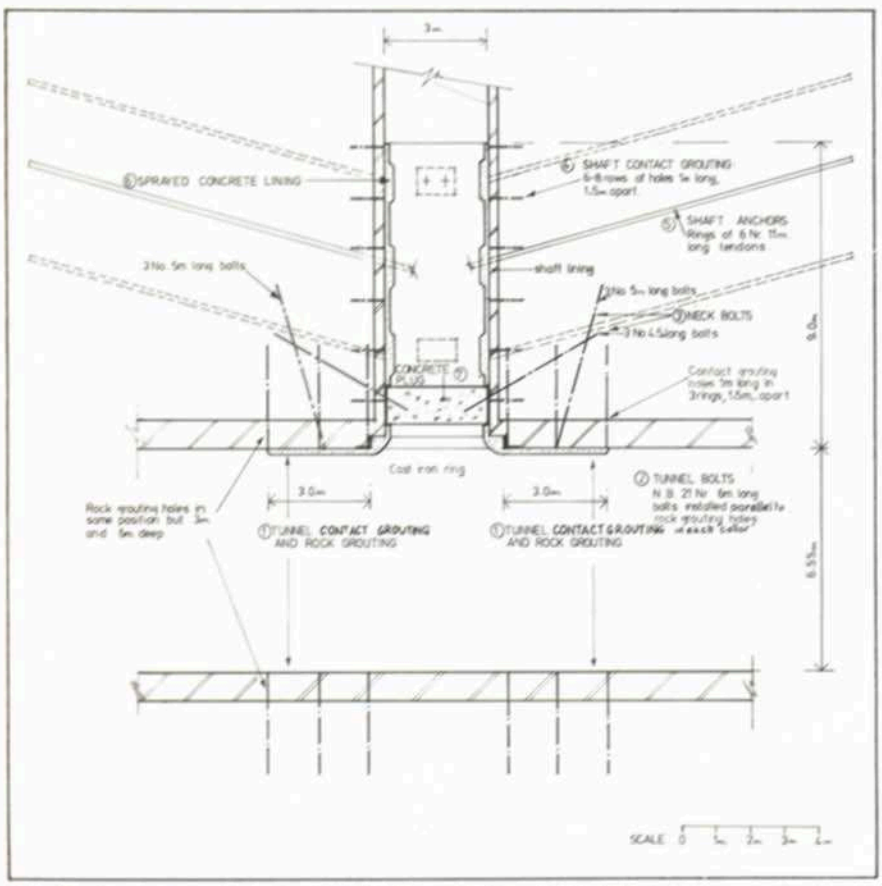


Fig 21. General arrangement of repair, Summit Tunnel, Yorkshire. (Extracted from James Williamson & Partner's Drawing)

However, early exploration holes indicated the presence of voids (some rubble filled) and heavily fractured bedrock behind the shaft linings. Due to the uncertain state of stability of the shaft lining, it was decided that the remedial measures should involve the minimum interference with the lining. The consultant considered using rings of anchored shear keys designed to carry the lining, the rings being located at the base of the shaft and at one and two thirds of the shaft height. Following observation of fissures and bulging of the lining, however, the consultants reviewed the design and decided to concentrate the anchored shear key supports in three rings located over the lower 10m of the shafts and devised the activity sequence for their installation. All work was to be conducted beneath a suspended protective shield.

The following sequence of events was agreed (Figure 21):

- Drill through the lining and grout the brick/rock annulus to create a "plug" – 3m wide – around the tunnel either side of each shaft intersection and for approximately 10m up each shaft in order to fill voids and consolidate debris.
- Install stitching bolts over the same tunnel areas.
- Clean the damaged brickwork, removing and replacing sections, and spraying concrete, wherever necessary to also deal with "perennial damp".
- Install groups of prestressed rock anchors at various levels in the lower 10m of each shaft, to act as shear keys to prevent possible slippage of the lining.
- Form a concrete plug at the base of each shaft and fill the shaft with a light weight foamed polyurethane to prevent the future collapse of the lining.

The Execution

The grouting of the tunnel annulus and surrounding rock was first conducted. On each of the four "collars", three rings 1.5m apart each with 8 Nr 50mm dia. holes 1m long and perpendicular to the brickwork were drilled and grouted. A neat cement grout of W = 0.5 was used, together with the addition of a thixotropic agent to restrict lateral flow. Three holes were slightly deepened for exploratory purposes at Shaft 9.

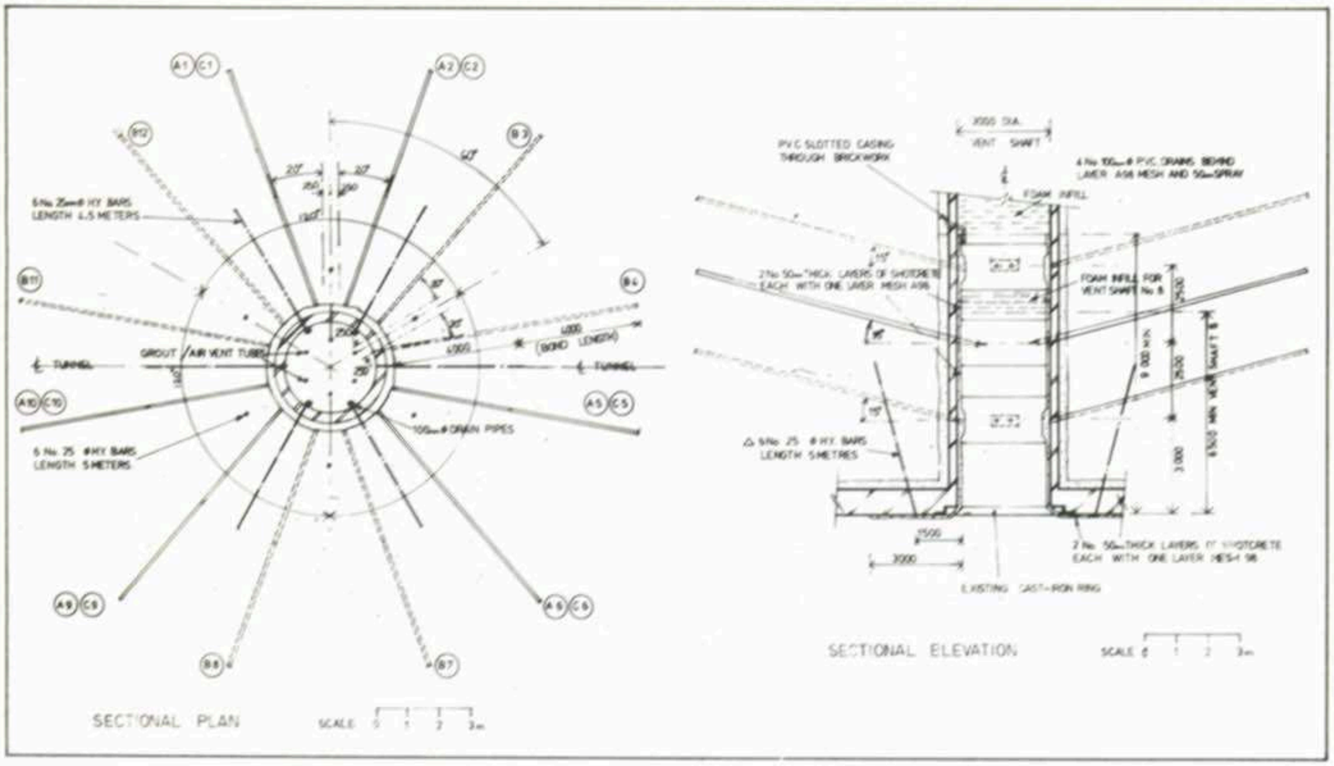


Fig 22. Details of Anchor arrangement, Summit Tunnel, Shaft 9. (Extracted from James Williamson & Partner's Drawing)

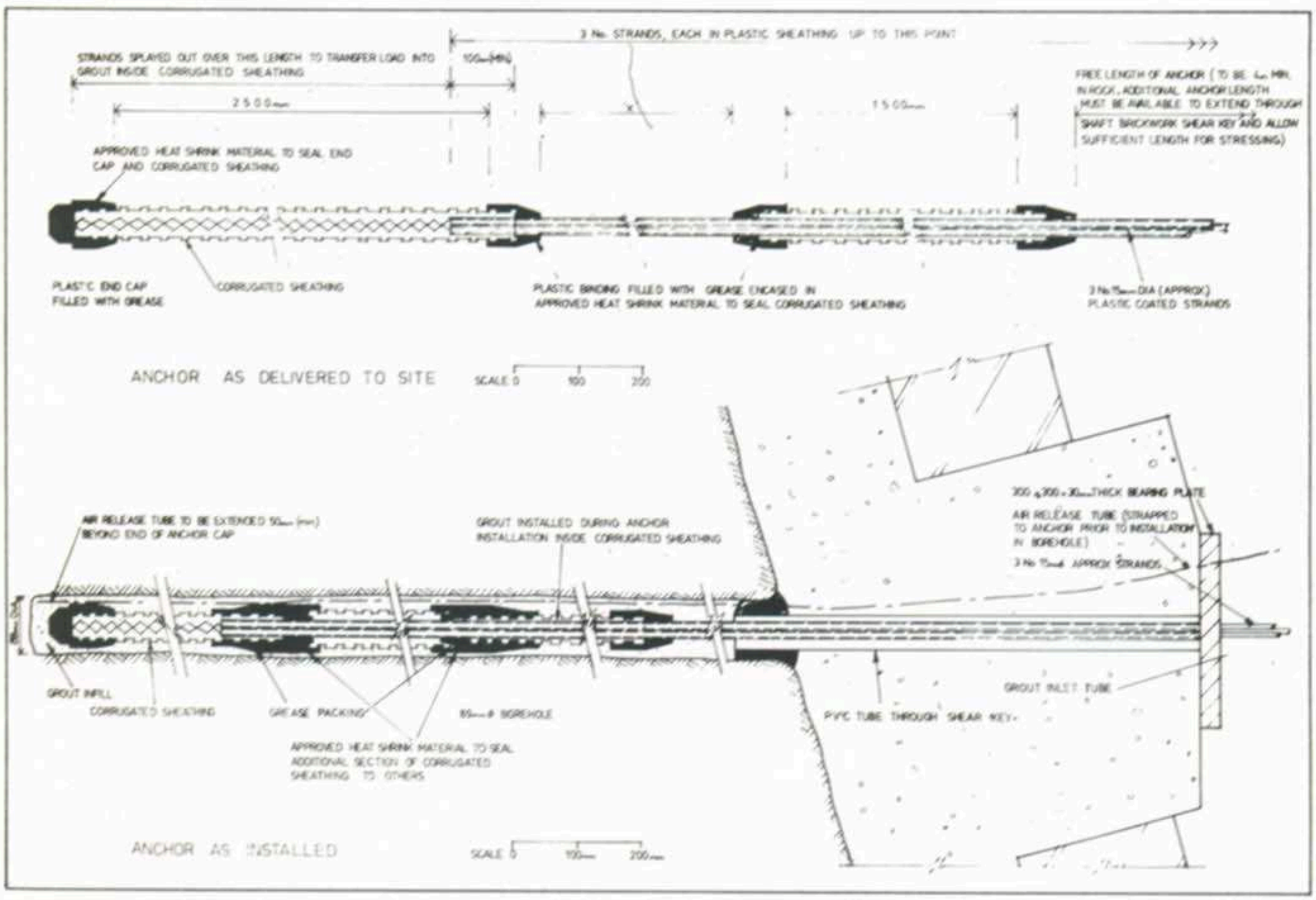


Fig 23. Details of Special "articulated anchor", used in Shafts 8 and 9, Summit Tunnel. (Extracted from James Williamson & Partner's Drawing)

Thereafter these 96 holes were redrilled to 3m or 6m (with track rig as opposed to hand sets) to permit fissure grouting of the immediate rock surround (Photograph 7).

Still concentrating on these collars, 42 Nr 6m long 38mm diameter steel bolts were fixed in 60mm holes by epoxy resin anchorage capsules in each shaft base (i.e. 3 rows each with 7 bolts – all above the horizontal – in each collar). A nominal prestress was applied.

At around this time a total of 14 Nr reflector pins – for survey purposes – were installed.

Twelve Nr 25mm bolts, 5m long were then installed at 15° away from the shaft vertical to further reinforce the next area (i.e. 3 at each intersection).

Attention was then focussed on the lower section of each shaft: 8 Nr staggered rings each of 6 Nr 1m deep 50mm holes were drilled and grouted in the lower 6.5m of Shaft 8, and a further 10 Nr rings were installed in the basal 9m of Shaft 9 (by handset) with an interhole spacing of 1.5m.

Overall, for the contact and rock grouting in these six areas (i.e. 4 collars plus 2 shafts) a total of 220 tonnes of OPC was injected.

Then 6 Nr 25mm, 4.5m long bars were installed radially around the base of each shaft (30° above the horizontal), to act as a direct key to the subsequent concrete plug.

Finally, from the view point of this paper, a total of 30 Nr (12 in Shaft 8 and 18 in Shaft 9) 11m long, 3 strand anchors were installed in 89mm holes. The Consultant's activity sequence required that the walls of the lower 10m of each shaft be meshed and covered with 50mm of shotcrete prior to perforating the brickwork for the shear keys. The lowermost ring of 6 were installed 3m from the bottom of the shaft and were inclined up at 15° (Figure 22). As shown in Figure 23 these double protected anchors had specially fabricated fixed lengths to allow them to be installed in the restricted access available. These were grouted with a low water content "Excem" grout to shorten curing time before stressing. The anchors were stressed to 55% GUTS.

After all the grouting, bolting and anchoring was complete, the Consultant specified that a further layer of shotcrete be sprayed over the working area prior to the placing of the concrete plug and the foam grout.

Despite the major restrictions imposed by the extremely difficult working conditions, the structural repair work was executed speedily and successfully, permitting the safe opening and operation of the tunnel with minimum delay.

5. PORTS AND HARBOURS

5.1 Weymouth Harbour, Dorset

The Site

Weymouth Commercial Quay was completed in 1934 and is a major terminal for the Sealink ferries operating to the Channel Islands. A typical cross section through the 300m long structure is shown in Figure 24.

The Problem

The original construction comprised a counterfort supporting a relieving platform, the backfill beneath being retained by precast concrete sheet piles. The "grout sausage" forming the interlock had deteriorated with time, to such an extent that the fines from the backfill had been washed out. This problem had been accerbated by the poor alignment of the sheet piles, exploited further by the more severe conditions resulting from easterly winter storms. Surface settlement had resulted, as evidenced most dramatically by the differential settlements on the quayside buildings. It was also suspected that the enhanced scouring effect of the directional manoeuvering propellers of the modern ferries had accelerated this problem, and caused over deepening of the sea bed local to the quayside.

The water encountered in site investigation boreholes drilled through the quay was noted to be directly influenced by adjacent tidal levels.

The Solution

As shown in Figure 24 the principle of repair as determined by the Borough Engineer was two-fold:

- Driving of a parallel protective toe wall of Frodingham 4N sheets, followed by the placing of a hydrocrete seal between existing and new piles, and
- Consolidation grouting of the existing granular backfill to fill all voids and safeguard the superstructure against further settlements. This grouting was to be conducted both above and below the horizontal concrete relieving slab in a block about 2m wide and 60m long.

The Execution

An underwater video survey of the existing quay face was carried out and this confirmed the scale of the various deficiencies in the original construction, including open joints between concrete piles, toes located above the harbour bed, and piles driven out of line. Clearly such voids (some sufficiently large to permit diver entry) had to be sealed as a barrier to grout escape. Whilst the smaller fissures (<75mm) could be sealed with hand applied "Quickcrete" mortar and reinforced by sand bags, the larger voids (up to 0.5m wide) needed a different solution. Fabric bags were therefore tailored, to shapes advised by the divers, and placed in the voids. They were then inflated by a cement-sand grout from the surface, thus forming a substantial "balloon" type seal between the piles. Twenty bags were installed in this way, up to 5.5m long and consuming 150m² of fabric.

Such measures were generally successful in minimising subsequent grout leakage, although the flow of this grout was further inhibited in places by the addition to the mix of Conbex 653 Thixotropic stabiliser, as noted below.

The seaward row of 19 Nr 9m deep holes at approximately 3m centres was drilled with the ODEX 76 overburden drilling system, operating off an Atlas Copco ROC 601 track rig. The system was chosen to combat the very variable

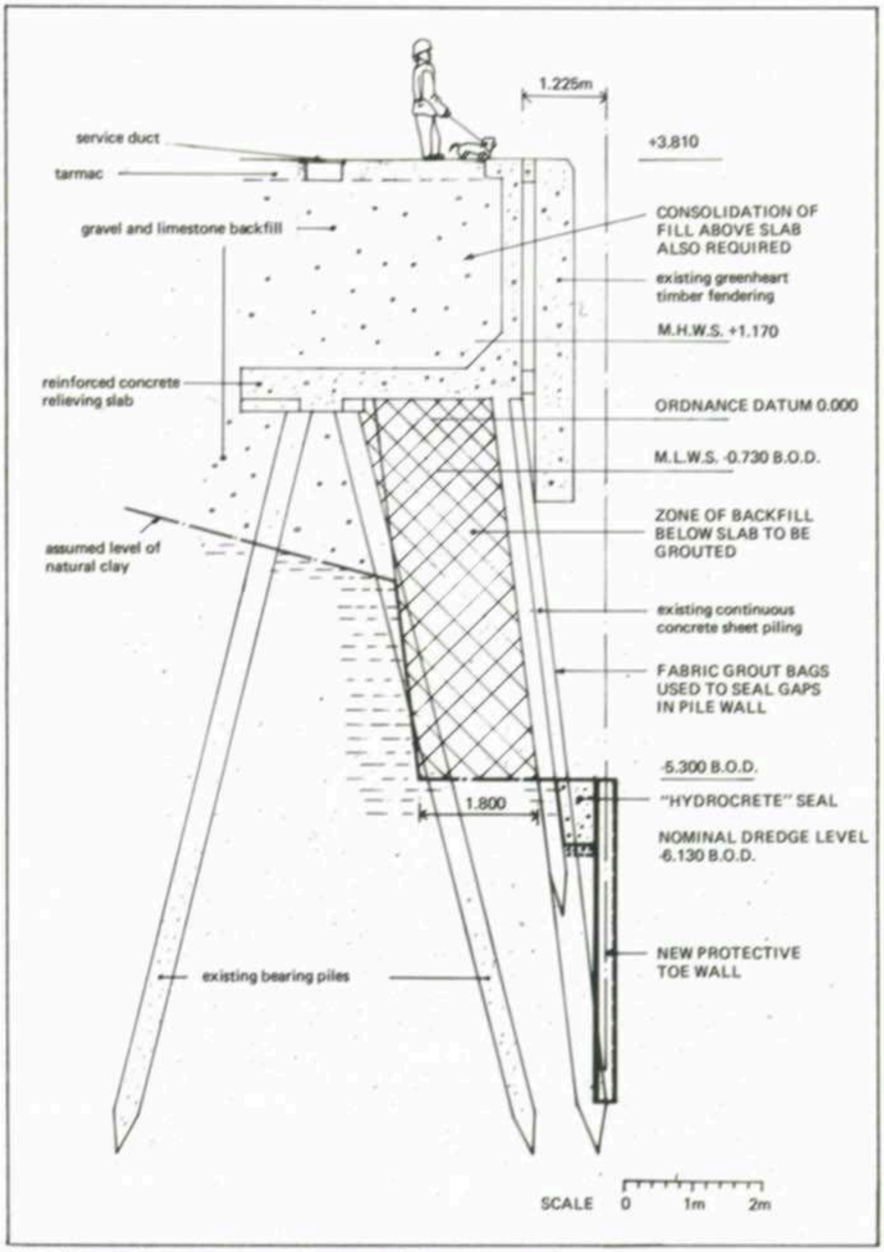


Fig 24. Typical Section through Commercial Quay, Weymouth

ground conditions in which timber and concrete obstructions were frequent. These holes were grouted with a viscous, resistant mix of 2:1 sand SR cement grout of WSR = 0.45, and consumed over 295 tonnes of solids (= 234m³ grout) below the slab. The 21 landward holes were grouted with a weaker, more fluid and economic mix of 10:1 PFA:OPC with WSR = 0.5. A total of 178 tonnes (= 156m³) was injected below the slab. The Primaries were grouted to pressures of 0.25 bars m/depth whilst to encourage travel, the Secondary target was increased to 0.375 bars/m. Individual holes had to be injected up to six times in order to localise the treatment and combat leakage or breakout of the grout. All grouts were mixed in colloidal double drum mixers and injected by Colmono pumps.

Consolidation holes were also drilled under the foundations of a new proposed quayside structure to eliminate the possibility of differential settlements as a consequence of the "drawdown" of fines over the years. 16 Nr holes were installed. The depth to the clay varied from 6-8m, and grout takes ranged from 0.21 to 13.5 tonnes/hole (average 0.84 tonnes/hole).

Generally, interpreted voidages ranged from around 23% for the inner row holes to about 35% for the seaward holes. Altogether almost 446 tonnes of the sand cement grout and 236 tonnes of the pfa cement grout were injected into the 540m of grout holes drilled in the contract.

The adequacy of the main treatment was verified by the attempted injection of 4 test holes in the main block, which took minimal quantities.

5.2 Canning Dock, Liverpool

The Site

In common with the other harbour facilities of the Liverpool South Docks, the Canning Dock was built during the early 1840's. The dock wall itself consists of a sandstone masonry block structure, about 11.5m high, and 4m wide at its base, and supported through approximately 4m of clays, silts and gravels onto the hard fine Bunter sandstone bedrock by timber piles. This wall is about 100m long. After their abandonment in 1972, the Docks became tidal with the result that substantial deposits of river salt had accumulated to a level of about 7.5m above Liverpool Chart Datum. The Docks were then isolated from the direct tidal effect of the Mersey by a series of temporary rubble dams to the seaward side.

As part of the efforts of the Merseyside Development Corporation to revitalise the area, a major refurbishment programme was put in hand, timed to culminate with the arrival of the vessels competing in the Tall Ships Race. Simultaneously, the existing warehouses and adjacent buildings were to be refurbished to accommodate the extended facilities of the Maritime Museum, plus certain commercial interests. Special care was taken to preserve as much of the original local artefacts and materials as possible e.g. bollards, capstans, light standards, stonework, and paving.

The Problem

Throughout the scheme, the typical range of structural repair techniques already described in this paper was used. Thus, dock walls were grouted, stitched and repointed as appropriate whilst the refurbishment of the warehouses and other structures involved the vacuum impregnation process.

However, survey had shown that the structural stability of the Canning Dock wall was highly suspect, and, as a temporary measure, a bund was placed, almost to coping level, in the dock to prevent outwards displacement.

Clearly, however, this bund would have to

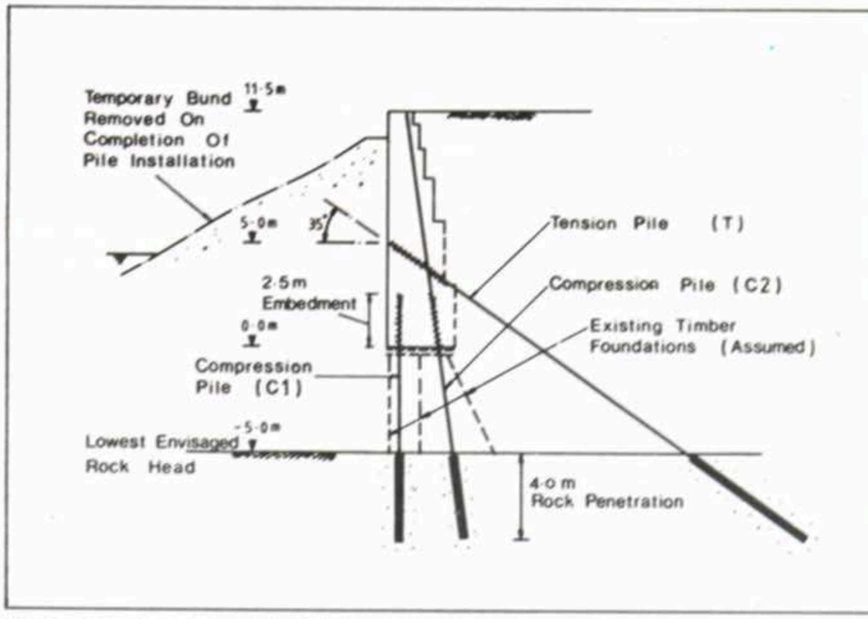


Fig 25. General arrangement of minipiles, Canning Dock, Liverpool

be removed to permit the dock to allow vessels to enter, and tie up against the quay, and so an alternative, permanent, in situ solution was necessary.

The Solution

A complex series of loading conditions had to be satisfied: forces acted from above, below, behind and in front of the wall, varied further by tidal influences.

Forces acting on the wall can be summarised as follows:

- Pa, the resultant active pressure acting behind the wall from fill material and hydrostatic pressure (variable)
- R, the resultant horizontal resistance provided by water in the dock (variable)
- U, the uplift on wall due to hydrostatic pressure (variable)
- Ww, the dead weight of the wall
- Wf, the dead weight of the fill
- Ws, uniformly distributed surcharge loading (variable)
- Wc, line surcharge loading (variable)

An arrangement of minipiles was therefore designed to satisfy all these conditions, in accordance with the provision for permanent structures in the relevant Codes of Practice.

To rationalise the drilling operations, it was advisable to install the pairs of compression piles continuously through the wall before penetrating the underlying strata. The need to maintain a minimum "cover" to each borehole of 400mm to prevent blowout dictated the general pattern shown in Figure 25. Practical reasons also fixed the location and inclination of the tension piles, designed to resist overturning and sliding. It was assumed that the wall was intrinsically stiff, and that overturning could be about the outer row of compression piles (C1). Also, no contribution for frictional resistance at the base of the wall was allowed.

Conventional calculations showed maximum working loads to be:

C_1 = 572.2kN, C_2 = 628.6kN, and T = 703kN, all per metre run of wall.

Pile spacings were calculated based upon the internal load bearing characteristics of the pile selected and the relevant safety factors. Compression piles incorporated 50mm diameter Dividag Gewi bars (ultimate load capacity 980kN) and tension piles used 36mm diameter grade 1080/1280 Dividag bars (ultimate load capacity 1252kN). The system provided minimum factors of safety against overturning of 2.4 and against sliding of 2.0.

Bonded lengths into both the existing wall and the sandstone were designed according to DD81[12] and used working bond stresses of 0.5-0.7N/mm² for the working loads of 47 tonnes (C) and 70 tonnes (T).

Whereas the C1 pile reinforcement was designed to terminate within the base of the wall (i.e. over a length necessary for safe load transfer conditions), the reinforcement of the C2 pile row was continued (as 9m long 30mm bars) to the top of the structure to provide a stitching effect.

The Execution

A number of trackmounted diesel hydraulic rigs was used to drill 140mm holes through the masonry with down the hole hammers, continuing as 127mm cased holes through the subbase deposits (rotary, water flush) and 105mm open holes in the bedrock (rotary, water flush). After the C piles had been completed, the bund level was reduced to just above water level (i.e. about 6m) and the tension piles drilled from this platform.

Each bar was fully protected by a corrugated pvc sheath, with the bar/sheath annulus pregrouted with a very fluid, high strength mortar. These encapsulated piles were fabricated on site.

Grouting was conducted at gravity pressure with Colmono pumps and colloidal mixers, delivering a neat cement grout of W = 0.45. Average grout takes over twice the nominal hole volume were recorded, indicating penetration into the loose or fissured surrounding ground.

Overall a total of 210 Nr compression piles (4197m) and 94 Nr tension piles (1692m) were installed.

Upon completion, the removal of the bund and the dredging progressed. A maximum outwards movement of about 4.5mm of the wall was recorded (this being necessary to cause the passive tension piles to commence working). This deflection was substantially less than could have been expected. The system therefore worked satisfactorily and the whole complex is now functioning as intended, as a major regional tourist and recreation facility.

6. SUMMARY AND CONCLUSIONS

This review of the details of recent successful case histories illustrates the scope of problems posed in structural repair, and confirms the range of grouting related processes which may be exploited to solve them. Thus in certain cases, the emphasis is on manual expertise, where the skill of the operator is paramount. The works done in Snowdonia are prime examples. Equally, the repair of Kincardine Viaduct underlines the value of logistical and organisational qualities, whereas the execution of highly specialised grouting or minipiling contracts demands a high level of technical ability. It would also seem that the range of repair techniques is continually expanding: the potential of soil nailing, for example, remains to be exploited fully in this country as a means of stabilising walls and embankments.

Given the anticipated upsurge in the requirement to repair and maintain structures, it is therefore reassuring to know that there exists already the knowledge and expertise which will surely be required.

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