OBJ3/P3/C1-C2

Transport and Works (Inquiries Procedure) Rules 2004

Proposed London Underground (Victoria Station Upgrade) Order

LAND SECURITIES PLC AND OTHERS (Objector No. 3)

APPENDICES 1 – 2 TO REBUTTAL PROOF OF EVIDENCE of
TIM CHAPMAN of ARUP

OBJ3/P3/C1

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APPENDIX 1

PREVIOUS EXAMPLES OF JET GROUTING

Special Publication 201 London, 2003

Response of buildings to excavation-induced ground movements

Proceedings of the international conference held at Imperial College, London, UK, on 17–18 July 2001

Edited by

F M Jardine

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Summary

The conference "Response of buildings to excavation-induced ground movements" was held under the auspices of CIRIA and the Imperial College of Science, Technology and Medicine. These proceedings include the papers submitted to the conference, all of which were refereed by two or more reviewers. Each paper has been edited usually only for consistency or lightly for clarity of wording and set into a standard format. The Proceedings include two invited lectures, the keynote lecture given by Professor Robert Mair of Cambridge University and the closing address by Professor John Burland of Imperial College. A principal aim for the conference was that it should be based mainly on discussion rather than formal presentation. The six technical sessions were introduced by invited rapporteurs summarising the submitted papers and setting potential discussion topics. Their reports are given here together with the records of the ensuing discussions of the six technical sessions. While the discussions were informal, notes were taken during them from which the records were compiled. A draft of each discussion was made available on the web to all of those taking part for them to check and, if necessary, to amend their contributions.

Response of buildings to excavation-induced ground movements. Proceedings of the international conference held at Imperial College, London, UK, on 17–18 July 2001

Jardine, F M (ed)

Construction Industry Research and Information Association

Special Publication 201 © CIRIA 2003 RP620 ISBN 0 86017 577 4

Keywords		
Underground construction, soil-structure interaction, *in-situ* testing and instrumentation, transport infrastructure, ground engineering, ground improvement		
Reader interest	**Classification**	
Tunnel promoters; tunnelling and geotechnical engineers; structural and civil engineers, building surveyors and insurers	AVAILABILITY	Open
	CONTENT	Collected papers
	STATUS	Committee-guided
	USER	Tunnelling, structural and geotechnical engineers, building professionals, surveyors and insurers

Published by CIRIA, 6 Storey's Gate, Westminster, London SW1P 3AU, UK.

Acknowledgements

As well as being a record of a successful conference, this book is also one of the results of a series of related research and information projects that stemmed from the research project "Subsidence damage to buildings: prediction, protection and repair". That research, sponsored by London Underground Limited, was done by Imperial College together with staff provided by the Jubilee Line Extension Project and the Geotechnical Consulting Group under the DETR-EPSRC Construction Maintenance and Refurbishment LINK Programme (EPSRC reference GR/K34306) and CIRIA's Ground Engineering Programme. As outputs became available, it was decided that an international conference would not only be an appropriate medium by which to make the results more widely known, but it would also provide an opportunity to learn of the results of other researchers and international developments.

The first main output from the research was the publication of the book *Building response to tunnelling* (Thomas Telford, London). Volume 1: *Projects and methods* of this book was made available in advance to conference delegates on CD-ROM, the bound volume being issued to them at the conference. Volume 2: *Case studies* was issued to delegates on its subsequent publication in 2002.

Many people and organisations who had previously contributed to the research supported CIRIA in organising this conference. As the proposal developed, several other organisations joined in as sponsors.

The conference organisers at CIRIA were guided by a conference advisory committee, which took the possibly courageous but key decision that led to worthwhile and enjoyable discussion sessions, which eventually had to be called to a halt.

The committee members were:

Mr J Moriarty (chairman)	London Underground Limited
Mr D A Baker	Balfour Beatty Major Projects
Mr A Bhogal	Institution of Civil Engineers
Professor J B Burland	Imperial College
Mr R Fernie	Cementation Foundations Skanska
Mr P R Glass	AMEC Capital Projects
Dr J E Hellings	Faber Maunsell
Mr F M Jardine	CIRIA
Mr D R Lamont	Health and Safety Executive
Dr R J Mair	Geotechnical Consulting Group
Mr K N Montague	CIRIA
Dr N J O'Riordan	Rail Link Engineering
Mr C Pound	Mott MacDonald
Mr A J Powderham	Mott MacDonald.

CIRIA gratefully acknowledges the following organisations that supported the conference.

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Mott MacDonald Group
Ove Arup and Partners International Ltd

The conference was formally opened by the Chief Engineer of London Underground Limited, Mr K H Beattie, whose support over many years for the research and its publication and for the conference is gratefully acknowledged.

The session chairmen at the conference were Professors E J Cording, H G Poulos, M B Jamiolkowski and S Thorburn, and Messrs J Moriarty, I Chudleigh, and A Bhogal and Dr P L Bransby. Particularly valuable features of the technical sessions were the introductory reports and discussion leadership given by the rapporteurs, Messrs C Pound, R Fernie, J Dunnicliff, K G Higgins and Drs J E Hellings and G M B Viggiani. Mr D I Harris and Dr J R Standing described some of their work on two of the important case studies from the Jubilee Line Extension Project, the Big Ben Clock Tower and Elizabeth House, respectively. The keynote lecture was given by Professor R J Mair. Professor J B Burland in his closing address drew attention to the conference's lessons and highlights. Professor P R Vaughan hugely entertained delegates with his after-dinner speech. The effort, care and thought given by all these people were very much appreciated not only by CIRIA as conference organiser but also, and more importantly, by the delegates.

CIRIA's conference organisers, whose hard work and careful planning helped to make it so enjoyable an event, were Nipa Patel, Charles Perkin, Louise Denniff and Caroline Lillywhite.

EDITOR'S NOTE

The preparation of these proceedings took longer than had been intended for three reasons: the priority of completing Volume 2 of *Building response to tunnelling*, a strained muscle – a rather critical one – precluded editorial or any other work for a time and, in some exculpation, some important contributions from busy people were received many months later than we expected.

All the submitted draft papers were refereed by at least two reviewers and revised as appropriate. In addition, the papers have been – generally lightly – edited for English and consistency of spelling and usage. In that the papers have been put into a standard CIRIA format, it has sometimes been necessary to change the size of figures and illustrations. In a few cases, figures have been redrawn or edited (eg where graph lines in colour would not have been distinguishable when printed in black and white). We have not included the presentations on the Big Ben Clock Tower and Elizabeth House given by Mr D I Harris and Dr J R Standing as these are fully written up in *Building response to tunnelling*, Volume 2, *Case studies*, Chapters 28 and 30, respectively. There are, however, two additional papers in these proceedings that describe case studies of the Jubilee Line Extension– at Park Place and Lloyds Bank – which were not ready in time either for inclusion in *Building response to tunnelling* or the conference, but which are included here to keep them within the main body of the Imperial College-CIRIA work.

An explanation is due of the method used for reporting the discussions. The editor had used a similar method for an earlier CIRIA conference *Engineering and health in compressed air work* – with some trepidation then because the delegates included medical doctors and researchers, occupational health and safety professionals as well as engineers. The method seemed to work well and was used here. Some four or five reporters provided with pens and notebooks were assigned to each technical session to note the names of everyone contributing to the discussion and to record so far as they could what was said. Although some *lacunae* in these notebooks were inevitable, it was possible to match sequences and subjects. The draft record prepared by the editor was placed on a website made available to all who had contributed to the discussions for their comment and amendment as several, very helpfully, did. Perhaps the delegates should have been given the following warning:

> Hear, Land o' Cakes and brither Scots
> Frae Maidenkirk to Johnny Groats,
> If there's a hole in a' your coats,
> I'll rede ye tent it:
> A chiel's among you *takin' notes*;
> And faith he'll print it.
> (Robert Burns, on Grose the Antiquary).

But the inadequacies of these printed records should be laid at the door of this "chiel" – the editor.

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MAINTAINING THE INTEGRITY OF THE READING ROOM DURING BASEMENT EXCAVATION AT THE BRITISH MUSEUM

P Scott[1] and R Essler[2]

[1] Buro Happold Limited, Bath, UK
[2] Keller Ground Engineering, Wetherby, UK

ABSTRACT

The courtyard of the British Museum has been redeveloped following the relocation of the British Library. A key element of this work has involved the construction of a basement adjacent to the Reading Room, which has an inherently fragile interior and is an important element of London's architectural heritage. This paper addresses the evolution of the design solution, the issues arising during construction and the way in which the process was monitored to ensure the integrity of the Reading Room was not compromised. To obtain confidence in the choice of design parameters specialist laboratory and fieldwork was undertaken. Resulting from the design development jet grouting was adopted to underpin the weak Reading Room foundations. During the works a real-time monitoring regime was set up and its value proven at an early stage when excessive ground movements occurred. This resulted in modifications being made to the jet grouting process. Latterly modifications to basement construction were made to accommodate the partial formation of jet grout columns in the London Clay.

Keywords: Case study, London Clay, small strain triaxial, self-boring pressuremeter, finite element analysis, jet grouting, monitoring, ground measurements

1 INTRODUCTION TO THE GREAT COURT DEVELOPMENT

The British Museum was designed in 1823 by Robert Smirke as a quadrangular structure surrounding a central courtyard. Within his courtyard the world-famous Reading Room was constructed (Figure 1). The Reading Room, which is around 40 m in diameter, has a cast-iron frame clad to a height of about 19 m with brickwork. Above this is a hemispherical copper dome, supported on timber struts, from which hangs a fragile *papier maché* ceiling. The cast-iron columns stand on masonry blocks, which sit on brick corbel foundations. Beneath these foundations the structure is underpinned by a weak mass concrete ring beam.

Until recently the Reading Room has been surrounded by bookstacks that were used to house the British Library. With the relocation of the library to a new building in St Pancras, the Museum Trustees were able to consider the redevelopment of the central area of the museum. This resulted in the appointment in 1994 of Foster and Partners as architects, and Buro Happold as engineers, to design the Great Court Project. The project has resulted in the creation of a large public square within the museum, with an elaborate panelled roof spanning over the entire courtyard and exposing the Reading Room to public view. In conjunction with this a basement area has been formed to house a large Centre for Education, exhibition gallery and plant rooms. This resulted in a significant excavation adjacent to, and partially beneath, the weak mass concrete footing that supports the Reading Room. This paper focuses on the evolution of the design that was adopted to allow the basement to be constructed, and the construction procedures and monitoring used to ensure the integrity of the Reading Room was maintained throughout the project period.

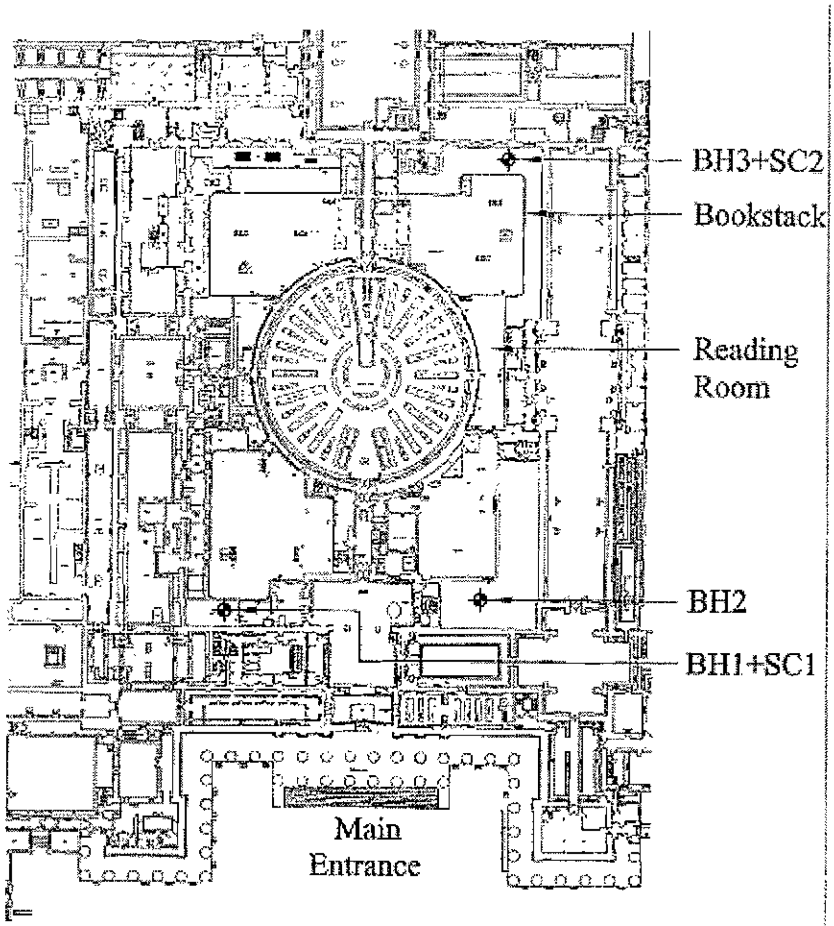

Figure 1 *Plan of the British Museum*

2 CONCEPT DESIGN OF THE BASEMENT

At the design concept stage of this project it was apparent that, due to the restricted working area within the courtyard and access problems, only small manoeuvrable plant could be used to install groundworks to support the Reading Room. This focused attention on the following methods of working.

1. The use of a contiguous piled wall with permeation grouting where necessary to aid water cut-off.
2. The use of jet grouting to support the Reading Room footings. This would act both as a gravity structure to resist lateral displacement and prevent water ingress into the excavation.

Each of the systems had potential problems. Due to the size of the continuous flight auger rig it would probably be precluded for use on the site and a tripod rig would have to be used instead. This method would be slow and was also limited by how close the piles could be installed to the Reading Room footings. The jet grouting option was proven within gravel and sand but effective performance in the London Clay seemed unlikely. A supplementary system to restrain the clay beneath the jet grouting appeared necessary, and the concept of using grouted steel dowels or mini-piles was preferred at this early stage.

With the intention of validating the design concepts, and providing information to allow costing of the project to be undertaken, simplified finite element models were developed and analysis undertaken. The soil parameters were derived empirically by comparing available basic soils information with published case histories of basement construction in London Clay. As undrained shear strengths measured in and around the British Museum were generally higher than those pertaining to the case histories reviewed (Cole and Burland, 1972; Stevens *et al.* 1977; Burland and Hancock, 1977; Burland and Kalra, 1986) the House of Commons Car Park modulus profile, being the most optimistic, was adopted (Table 1).

Results from this early work indicated that construction would not be able to proceed without recourse to some form of restraining system to limit lateral movement during excavation. As, at this stage, the basement was envisaged to entirely surround the Reading Room consideration was given to using ties running right across the underside of the structure, or installing a pre-stressing belt around the entire circular footing. Subsequently, when costs became prohibitive, the basement was restricted to the south side of the Reading Room and ground anchors were considered to provide restraint.

3 DERIVATION OF DESIGN PARAMETERS

In a previous paper (Scott and Seymour, 1999a) a detailed discussion is presented on the use of small-strain triaxial and self-boring pressuremeter testing in an endeavour to obtain good quality design parameters for this project. Problems were encountered particularly in relation to obtaining low suction pressures in undisturbed London Clay samples. This was addressed by comparing data from work carried out at Queensberry House (Scott *et al*, 1999) and subsequently deciding to derive design parameters primarily from the pressuremeter testing.

The adopted parameters derived from the results of the site investigation are summarised in Table 1.

Table 1 *Material parameters*

Material	Depth (m OD)	K_0 (at rest pressure)	E (Modulus) (MPa)
Taplow Gravels	25.0–19.4	0.5	122* (50[1])
London Clay	19.4–18.0	2.0	72* (36[2])
London Clay	18.0–14.0	2.5	79* (52[2])
London Clay	14.0–7.0	2.0	174+ (92[2])
London Clay	7.0–0.0	2.0	278+ (140[2])
Jet grout in gravel			5000
Jet grout in clay			250

* 0.1% strain
+ 0.05% strain
[1] E = SPT N
[2] House of Commons Car Park (Burland and Hancock, 1977)

The consequence of having endeavoured to obtain as best we could, using current best-practice procedures, the appropriate soil parameters was that significant changes to the basement construction could be employed. Our final model incorporated the parameters given in Table 1 and proved that using jet grouting alone, beneath the weak mass concrete footings of the Reading Room, would restrict movements to tolerable levels. The improved stiffness of the soils due to the process of jet grouting, used in the modelling, were based on information provided by a specialist contractor and our assessment of data available from published papers (Shibazaki *et al*, 1982; Greenwood, 1987; Newman *et al*, 1992).

A cross-section of the design solution is given in Figure 2. An element adopted at this stage, which had provoked considerable discussion within Buro Happold, was the replacement of grouted dowels within the London Clay with a jet-grouted block. Although there were questions raised as to its viability considerable programme and cost advantage was identified if the procedure was successful. The groundworks element of the project went out to tender with the preferred design option of jet grouting, even within the London Clay, but with the contingency to incorporate grouted dowels or similar if ground, and subsequently footing, movements exceeded prescribed limits.

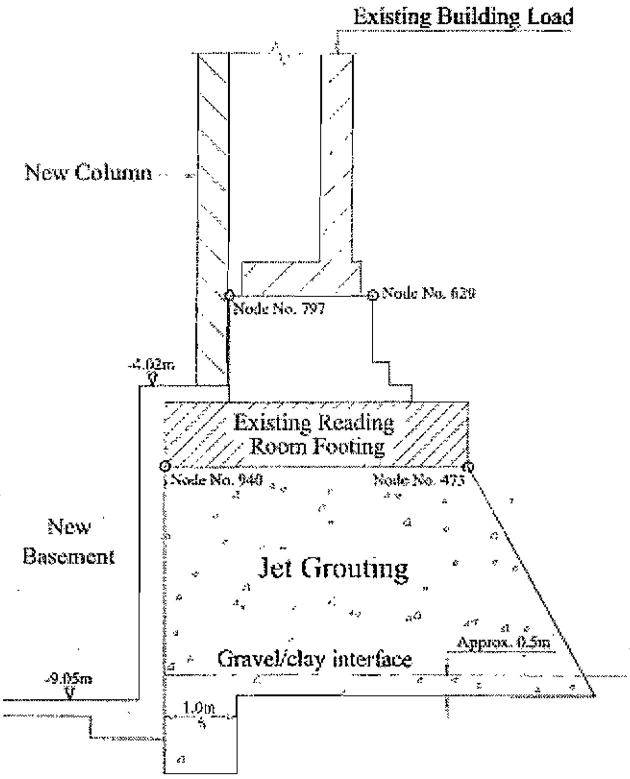

Figure 2 *Jet grouting beneath the Reading Room footing*

4 INSTRUMENTATION AND MONITORING

In order to take account of the potential sensitivity of the Reading Room, and in particular the *papier mâché* ceiling and isolated cast-iron columns, the maximum acceptable limits for the Contract were as given in Table 2. These were set on the basis that when structural analysis was undertaken using the prescribed displacements on two adjacent columns the building would not experience significant visual distress.

Table 2 *Predicted and actual movements for the South Basement*

	Contract limits	Actual movements	Predicted
Horizontal displacement	15 mm	11 mm	12 mm
Vertical displacement	10 mm	6 mm	5 mm
Rotation of footing	1:800	1:1400	1:1700

The instrumentation used to monitor the works against the predicted and prescribed (contract limits) movements was concentrated around the main southern basement area (Figure 3).

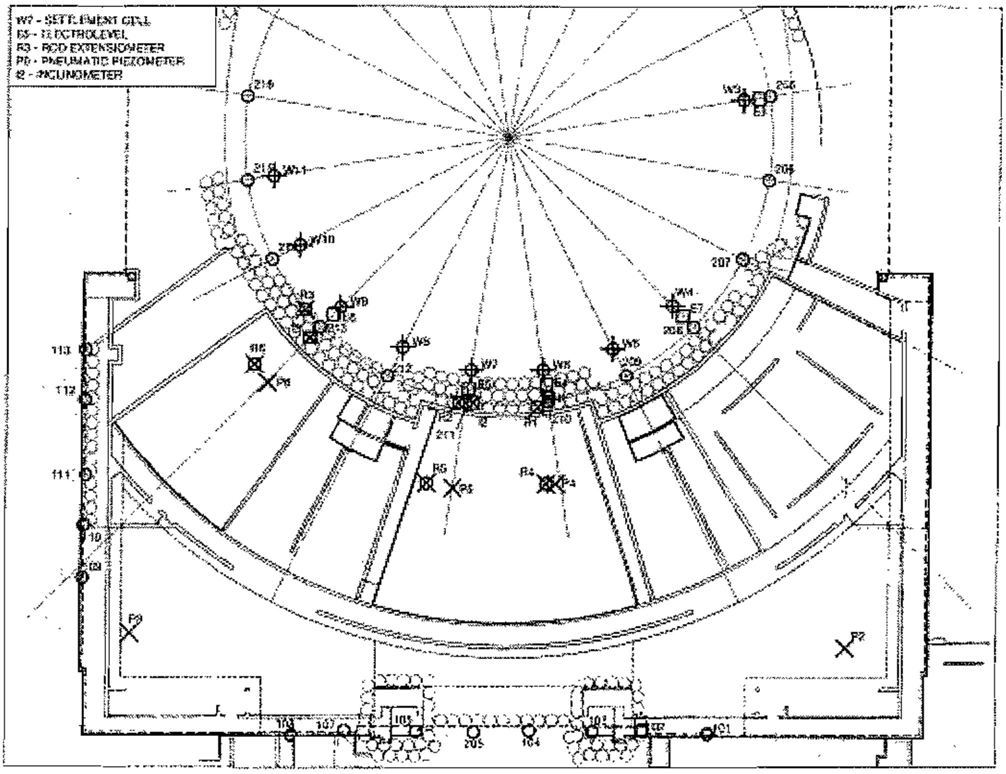

Figure 3 *Layout of instrumentation in the south basement area*

For monitoring footing movement a number of settlement (water) cells and electrolevels were installed, and to duplicate these measurements geodetic surveying and precise levelling was specified.

For measuring movement at depth inclinometers were installed through the mass concrete footing and rod extensometers positioned both in the basement excavation and around the perimeter of the Reading Room. Pneumatic and Casagrande piezometers were also positioned to monitor water level changes in the gravel, and pore pressure changes in the clay due to the excavation.

Beneath the Reading Room is a series of passageways that radiate from the centre of the building. These channels were an ideal network in which instrument cabling could be laid, keeping them remote from construction activity and limiting the possibility of damage. The data were centrally logged, generally at five-minute intervals, and immediately displayed on computer terminals in the engineer's and contractor's offices. An exception to this were the inclinometers, which were manually read and results presented to the Engineer the following day.

A typical form of presentation is given in Figures 4 and 5. In Figure 4 a schematic layout showing the Reading Room south basement and northern lift shaft is overlain with water cell locations. Each water cell was given a unique reference number (eg WL-8) and presented beneath it was the latest settlement reading. On the computer screen the box in which the reading was given was coloured green, amber or red:

Green	settlement less than or equal to the finite element prediction.
Amber	settlement over the predicted value but not exceeding the actual limit.
Red	exceeds the actual limit for that given construction stage.

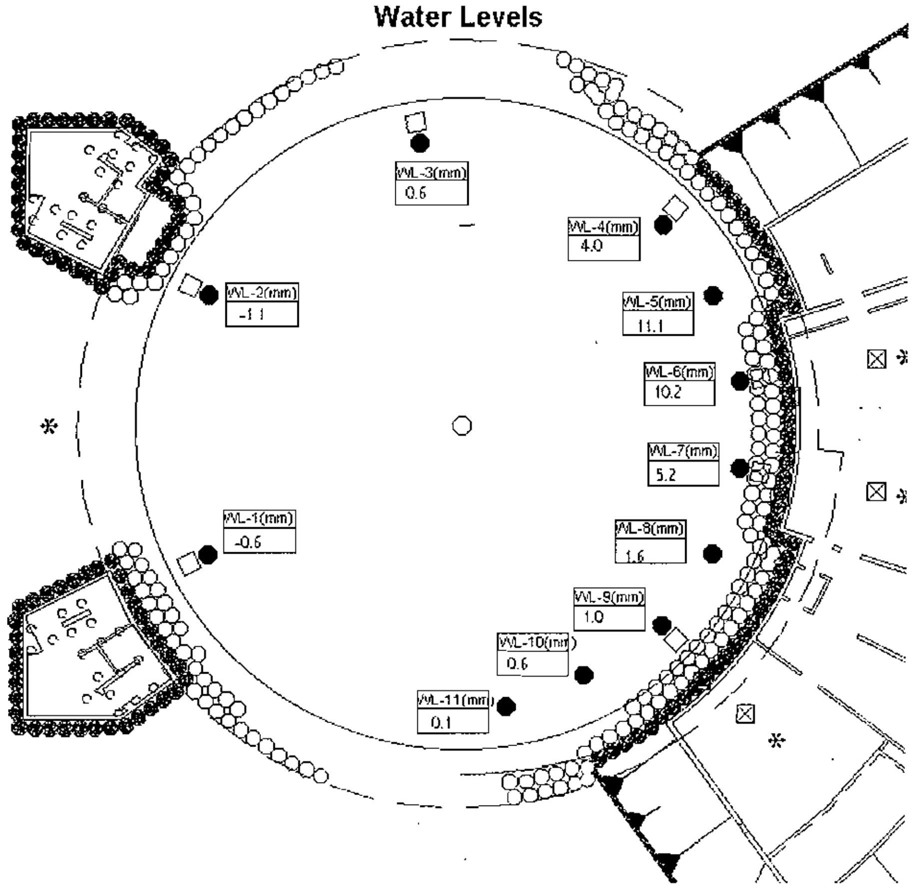

Figure 4 *Typical computer presentation showing water cells*

By "clicking" with the mouse on any one of the water cell points a second presentation was given showing settlement changes over the previous five days for the individual water cell. Figure 5 shows a "five day" water cell presentation for all the cells on site.

5 JET GROUTING

Keller were awarded the jet grouting contract and commenced work on site in August 1998. The jet grout underpin consisted of four rows of jet grout columns within the Taplow Gravels, which were interlocked to form a gravity block beneath the Reading Room Foundations, as illustrated in Figure 6. The column diameter was set at 1200 mm and the front row of columns were positioned at 1000 mm centres. Due to curvature of the foundation the rear columns were centred at less than 1000 mm. The front line of columns was also projected to extend 1 m into the London Clay.

Due to extreme access constraints the se- up area for the contract was created outside the Courtyard, adjacent to the main museum entrance. The grout, jetting water and air were conveyed into the working area by means of lines laid in ducts that passed through the museum basement. Radios were used to communicate between the working area and the set-up area.

The jet grout process adopted to construct the 1200 mm-diameter columns utilised the operating parameters given in Table 3.

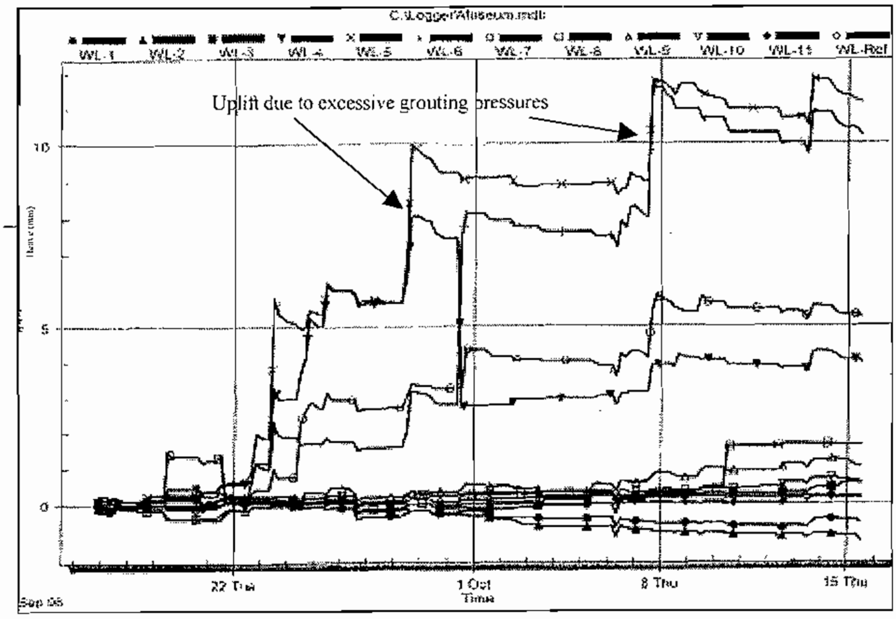

Figure 5 *Display of water cell movements*

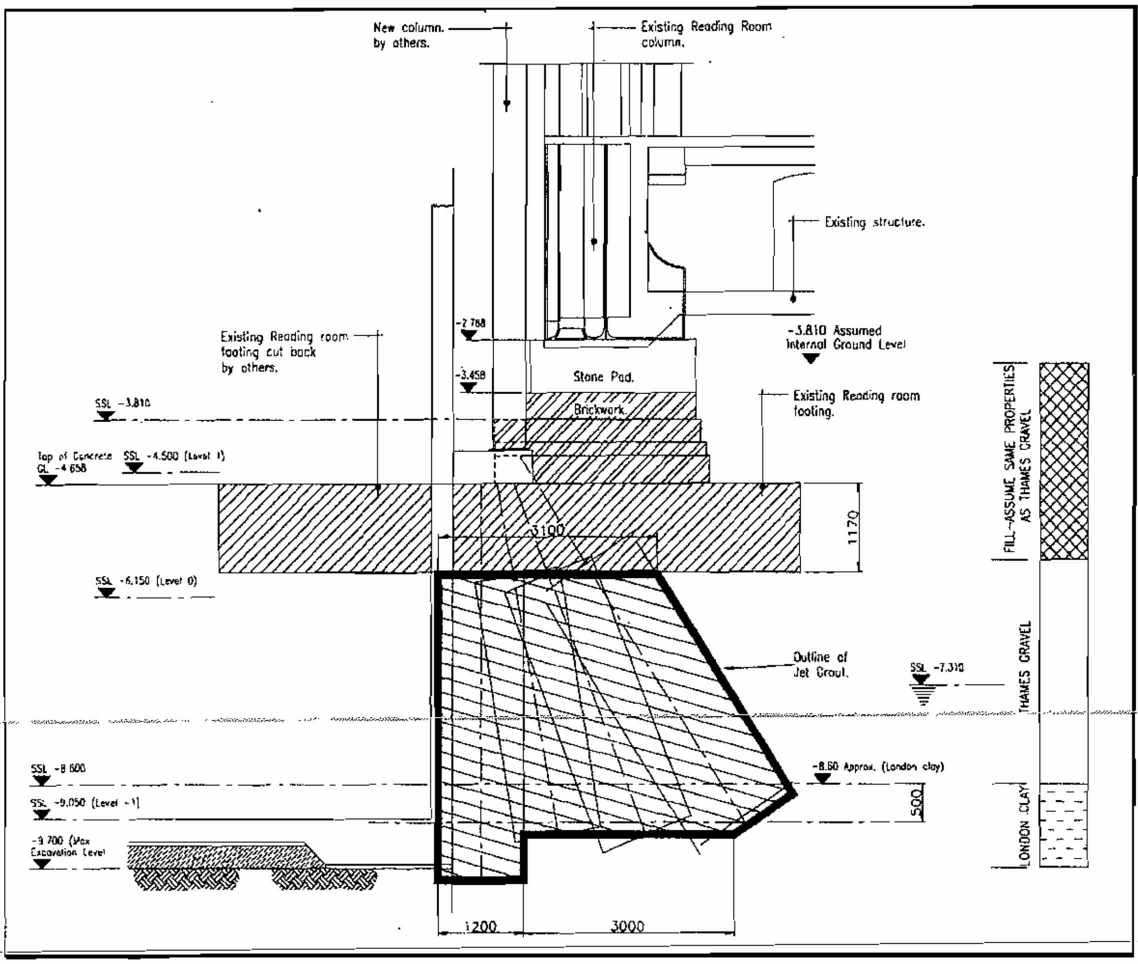

Figure 6 *Configuration of jet grout columns*

6 MODIFICATIONS TO PROCEDURES DURING THE BASEMENT WORKS

During the course of this contract two significant incidents occurred that gave rise to changes in the method of working. They also realised the benefits of real-time monitoring and of having predictive models that compared well with actual field performance.

During jet grouting beneath the Reading Room footing a significant ground heave was observed. The contractor had predicted that movements during the jet grouting operation would only be ± 2 mm. On this particular day an instantaneous rise in the water cells readings of 6 mm occurred (see Figure 5). This was observed on the monitoring screens and resulted in an immediate cessation in grouting work; at this point the heave had reached 10 mm, the maximum allowable vertical movement for the entire works. Subsequently it was discovered that the driller, when forming grout columns in the clay, had applied excessive pressures to unblock the grouting drill bit. Trials were implemented to identify a better means of control, which resulted in grouting all subsequent columns with a larger-diameter drill bit, and in critical areas pre-drilling. The pre-drilling was undertaken using a 300 mm-diameter drill bit, which ensured sufficient annulus during the jetting of the clay to allow the thick viscous spoil to be ejected without invoking undue pressure. With this procedure implemented no further significant movement occurred.

Trials for the jet grouting had been implemented prior to commencing the grouting beneath the Reading Room. Unfortunately, cores of the grouted London Clay had not been obtained at that time to examine the effectiveness of the procedure. Consequently prior to excavation of the main south basement one of the northern lift pits was dug out to expose the jet grout columns that had been formed in the London Clay. Whereas in the gravels above a consistent matrix of grout was seen, in the clay only discrete columns were evident with the majority of the clay unaffected. The clay was also found to be very wet and spalling off the face.

A re-analysis of the southern basement by Buro Happold, taking account of the observations made on site, indicated lateral movements would reach their prescribed limit. When this was considered in conjunction with the poor quality of the exposed clay and the unexpected water flow, Buro Happold implemented some contingency measures. These broadly consisted on leaving a berm in place in front of the clay and then replacing the berm, in a "hit-and-miss" manner, with a 1 m-high L-shaped wall keyed into the competent jet grout above. These procedures proved successful, and worked better than expected as the clay was found to be in a much better state in the southern basement than in the northern lift pits.

7 OBSERVED GROUND MOVEMENTS

A previous paper (Scott and Seymour 1996b) identified problems with some of the instrumentation and readings that were undertaken during the groundworks contract. However, having back-up monitoring systems, many of which were read and displayed to computers in the site offices at five-minute intervals, ensured the integrity of the Reading Room was never compromised. In fact, when heave resulted due to jet grouting the movement was almost instantaneously noted by the engineer (see Figure 5) and the works halted before any obvious damage occurred.

Comparison between predicted lateral movements during basement excavation and observed performance of a number of surveying points is given in Figure 7. It will be noted that movement continued subsequent to the full depth of excavation having been reached, due to continuing "elastic" displacement and the removal of the berm (see Section 6) during wall construction. Inclinometer displacements are given in Figure 8 for the period when the full depth of excavation had been reached but before lateral movements had stopped; unfortunately a review of subsequent data showed inconsistency in the readings. Both cases show good agreement with predicted values with a variance of about 1 mm for both the lateral and vertical movements (See Table 2).

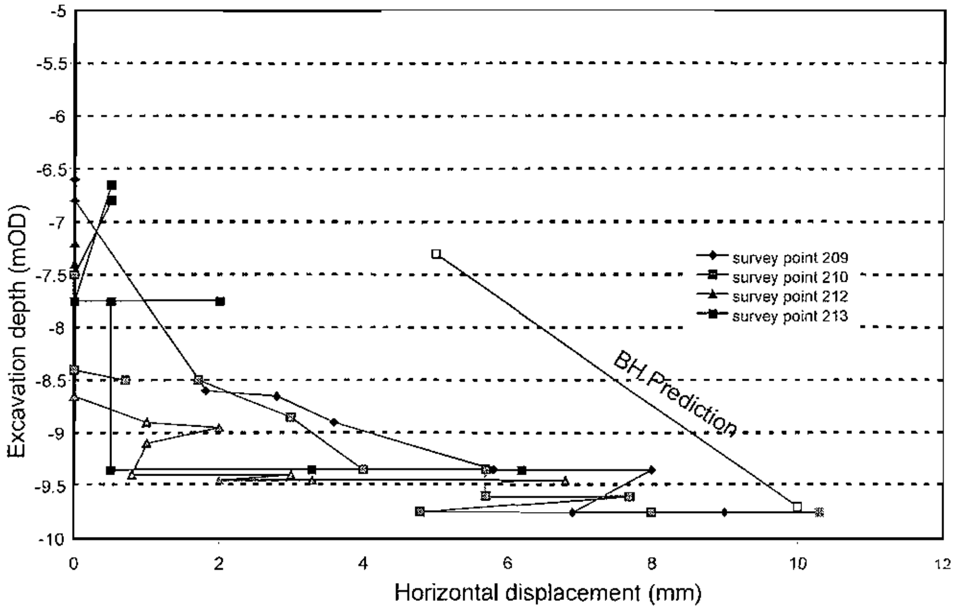

Figure 7 *Precise surveying – lateral movements*

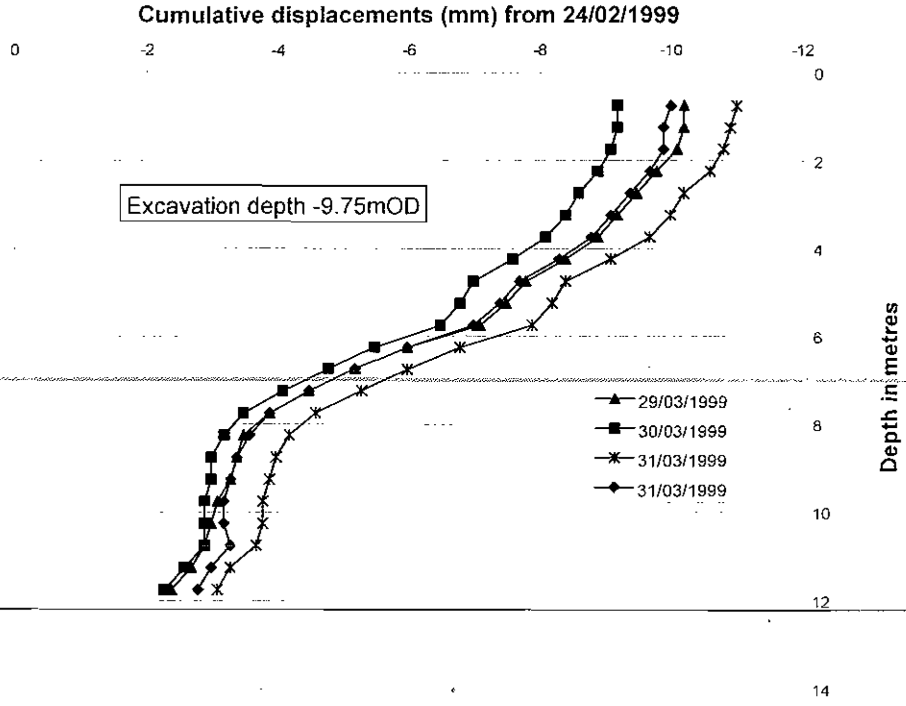

Figure 8 *Inclinometer readings*

8 CONCLUSIONS

The use of relatively complex site investigation and analytical techniques, in conjunction with careful consideration over the choice of design parameters, enabled a simple and cost-effective foundation design solution to be adopted for the Reading Room.

During the course of the basement works, however, issues arose that although successfully addressed at the time need to be borne in mind when similar projects of this type are undertaken.

1. Jet grouting, as with many ground improvement processes, is an operation which can have distinct variations in procedure and is prone to operator error. The value of continuous monitoring was proven when excessive grouting pressures led to unacceptable heave. If these observations had not been made damage to the Reading Room may well have occurred.
2. Jet grouting in stiff clay soils is difficult to achieve. Whereas jet grout columns in the gravels were of the order of 1.2 m in diameter, columns in the clay were about 300 mm across.
3. When the issue of incomplete jet grouting of the clay arose the value of being able to carry out finite element analysis, modified by field observations, enabled alternative changes to construction procedure to be sensibly evaluated. The use of a soil berm, subsequently replaced in a "hit-and-miss" manner by a 1 m-high wall, enabled the works to be completed on schedule and within the allowable movement limits.
4. Predicted and actual movements showed excellent agreement, with the provision of alternative means of monitoring ground movements ensuring that despite instrument failures and inconsistencies in some readings continuous control of the works was maintained.

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55 GROUTED STRUT DESIGN FOR DEEP STATION BOXES, NORTH-SOUTH LINE AMSTERDAM

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SUMMARY: The construction of three deep station boxes in the old city centre of Amsterdam introduces a settlement risk for the surrounding historic buildings. In order to reduce this risk the deformations of the walls of the station boxes should be limited. One of the essential elements in the design of the station boxes to limit deformations is a grout strut at a depth of approximately 30 m below surface level, constructed before excavation. This paper describes the design of the grout strut.
Keywords: Amsterdam, deformations, diaphragm wall, grout strut, historic city centre, metro tunnels, settlement, stiffness.

INTRODUCTION

For many years the city of Amsterdam has discussed the possibility of a metro line from the north to the south of the city. In the seventies the so called East Line was constructed. For this line some parts of the city centre of Amsterdam had to be demolished and there was a lot of political resistance against the removal of the buildings. Since the planned North-South Line had to cross the old city centre, the execution of this project was postponed till new techniques were developed to build an underground metro line without harming the existing city. In the nineties several bored tunnels in the soft Dutch soil were successfully constructed. The experience that was gained in these projects made

Foundations: Proceedings of the Second BGA International Conference on Foundations, ICOF2008. Brown M. J., Bransby M. F., Brennan A. J. and Knappett J. A. (Editors). IHS BRE Press, 2008. EP93, ISBN 978-1-84806-044-9. *www.ihsbrepress.com*

it possible to realise the wish of the city of Amsterdam to construct a metro line in the old city centre without removing buildings in the proposed alignment. By the end of the century the design of the line was finished and in the period of 2000 to 2002 several contracts were signed with different contracting companies to start building the metro tunnel and its stations. Figure 1 shows the alignment of the North-South Line.

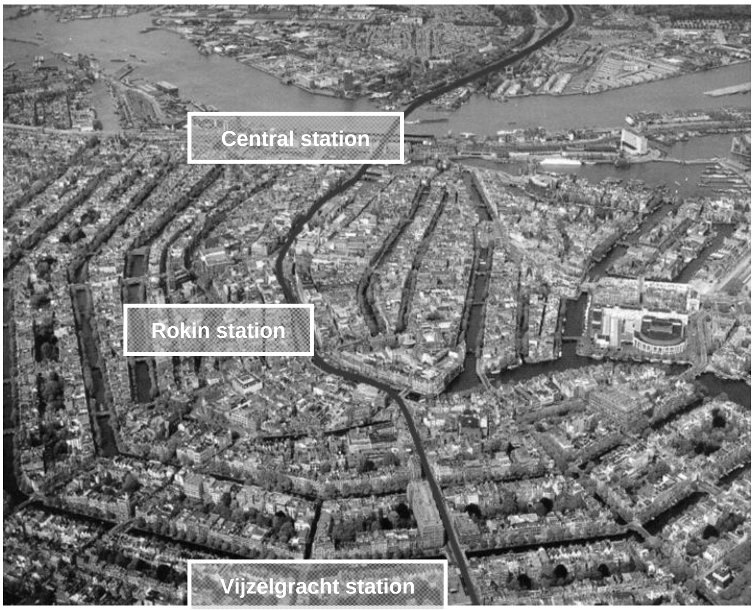

Fig. 1: *Arial view of the North-South Line in Amsterdam.*

As shown on the photograph, the line starts in the north part of the city, then crossing the river Ij, the Central station, followed by the old city centre. At the end the North-South Line is connected to the existing metro line at Amsterdam South – World Trade Centre. Figures 2 and 3 show maps of Amsterdam with the metro line and the stations.

As shown on the maps and the accompanying longitudinal section there are two stations in the northern part of the city which are at surface level. The river IJ will be crossed by an immersed tunnel connected to an underground station to be built underneath the existing Central Station in Amsterdam. In front of the Central Station the entry shaft for the bored tunnel is planned. From this entry shaft the bored tunnel will continue to the southern part of the city where the exit shaft is planned. The exit shaft will also be used as the newly built underground station Europaplein. In between the entry shaft and the exit shaft three new deep station boxes will be constructed, from north to south, Rokin, Vijzelgracht and Ceintuurbaan. The stations will be constructed using diaphragm walls and a reinforced concrete structure inside. Excavation of the station boxes will be done after constructing a deck thus avoiding long time interference with the city's traffic and facilitating a strut construction at the surface prior to the excavation.

Since the station boxes have to accommodate entry and exit of the bored tunnel, the stations are relatively deep with a platform level of approximately 21 m below surface level. The stations as well as the bored tunnel follow the pattern of the streets to avoid as much as possible interference with existing buildings. Because the area available is limited the station boxes are constructed close to the buildings. These buildings are in general founded on wooden piles which are driven to the first sand layer, a sandy stratum

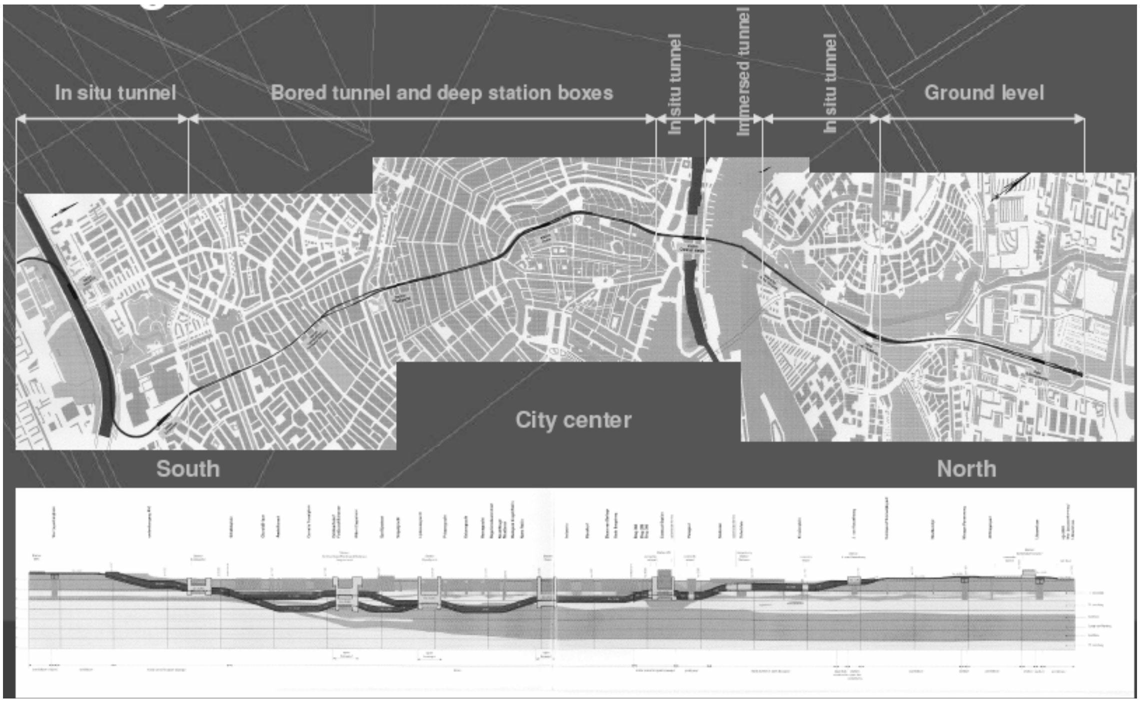

Fig. 2: *Alignment of the North-South Line in Amsterdam.*

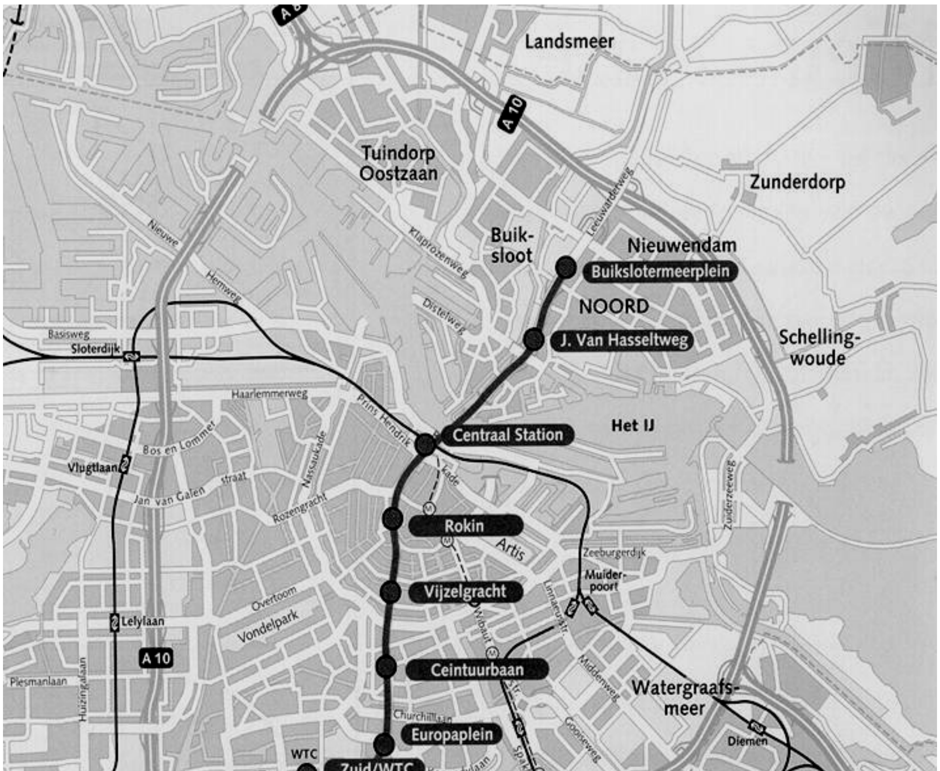

Fig. 3: *Alignment of the North-South Line in Amsterdam with stations.*

some 13 m below surface level. The bottom level of the station boxes will be significantly deeper than the foundation level of these piles. To limit settlement of the first sand layer, and thereby of the buildings, deformation of the diaphragm wall must be limited. To minimize the deformations of the diaphragm walls, the excavation of the station boxes will be done after constructing the station's deck and using steel temporary struts every 5 m of depth. This method is sufficient until the last phase of the excavation. To construct the base slab of the station, excavations need to continue to a level of approximately 26 m below surface level. However a strut cannot be placed due to the construction of the base slab. Further more the soil stratum just underneath the base slab, called the Eem clay is not stiff enough to reduce the movements of the diaphragm wall sufficiently in order to keep the settlement of the first sand layer below 25 mm. The latter being the boundary condition for the design of the tunnel and the station boxes. For this reason reinforcement of the clay layer at a level of approximately 30 m below surface level is required. This reinforcement is put in to practice by creating a strut in the clay layer made of grout. Figure 4 shows the soil profile which is representative for the three station boxes. In the following chapters the method statement used to design this grout strut and the modifications made to the design during execution of the works are described.

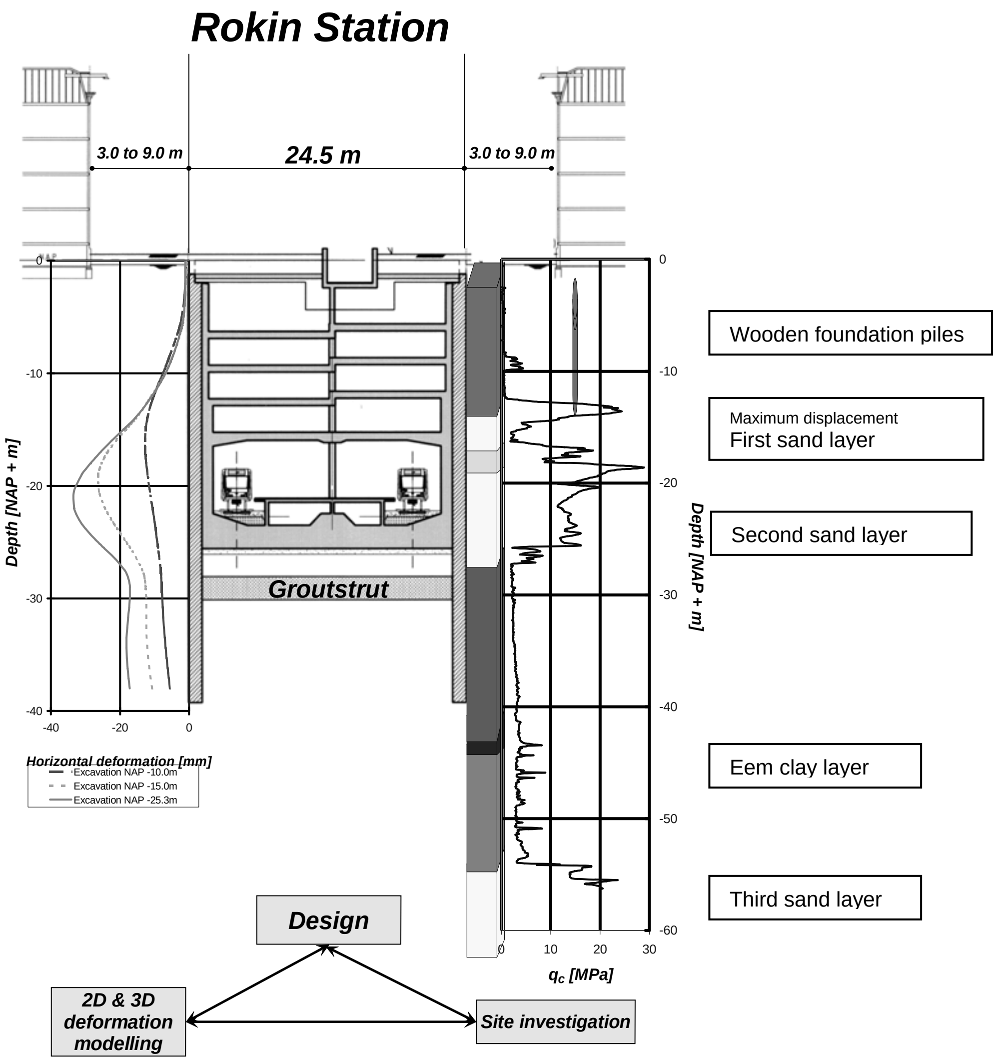

Fig. 4: *Representative soil profile of station boxes.*

INITIAL DESIGN

Initially the design of the grout strut was based on the following boundary conditions:

- The grout strut should achieve a certain stiffness in order to minimize the settlement of the first sand layer to 25 mm. This design stiffness was defined as 1800–2500 MPa.
- The strength of the grout strut was defined as 5.5 MPa.
- The grout strut should consist of a waling beam construction attached to the diaphragm walls with discreet struts at an horizontal interval of approximately 10.80 m.
- The diameter of the grout columns should be 1.10 m. This diameter was based on a trial carried out in 1999 in the northern part of the city.
- The grout strut should be constructed after finalizing the deck construction and the first two excavation stages at 12 m below surface level.
- The height of the grout columns (thickness of the grout construction) should be 1.50 to 3.25 m depending if the columns were part of the waling beam or part of the strut.

Based on these assumptions the contractual design was made. The constraint of a maximum settlement of 25 mm is based on the assumption that settlements within this limit should not lead to major damage to the surrounding buildings. FEM calculations using Plaxis showed that with the design parameters described above the settlement of the first sand layer stayed within this limit. See Figure 5 with the results of the FEM calculations.

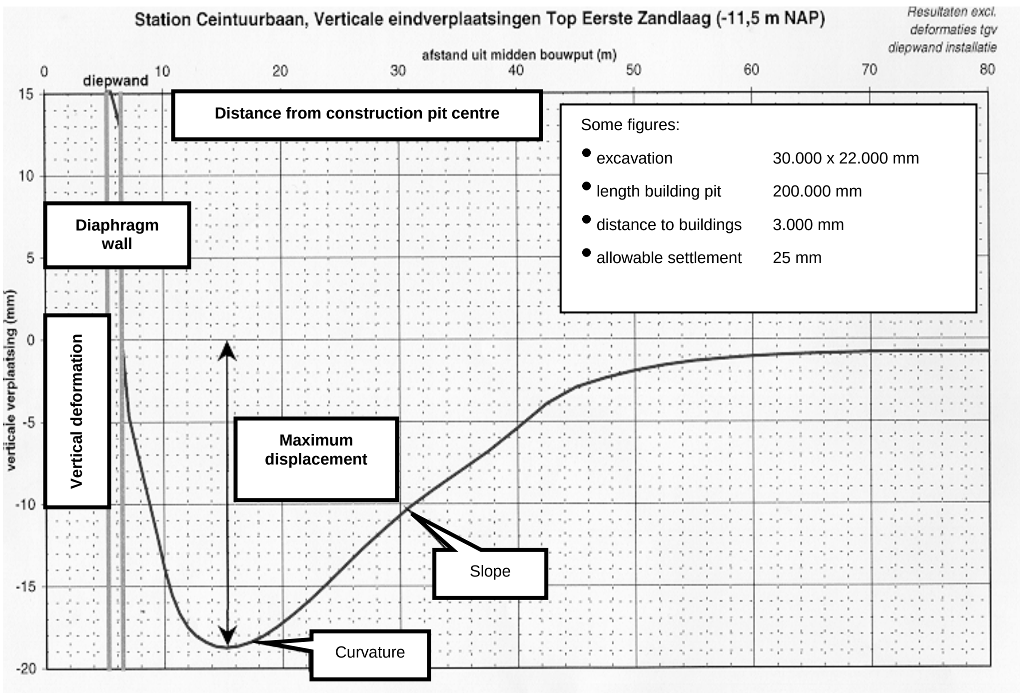

Fig. 5: *Representative prediction of settlement of the first sand layer.*

The original structure with a waling beam and struts every 10.80 metres can be seen on the drawing of Figure 6. The stiffness of this structure, with a Young's modulus defined as 1800–2500 MPa, was inputted in Plaxis to compute the graph as shown in Figure 5. The compressive strength necessary to carry the load imposed on the structure by the diaphragm walls was defined as approximately 1.5–2.5 MPa. Apart from this load from the diaphragm wall, a prediction was made for the phenomenon of swell for the Eem clay layer. Because of the excavation, the surcharge on the Eem clay will subsequently be reduced. This will cause a swelling of this layer and will thereby introduce a curvature in the grout strut. This curvature is an imposed deformation introducing additional stresses and an additional buckling problem. To cope with these additional stresses and buckling the minimum compression capacity of the grout structure was defined as 5.50 MPa. The defined stiffness and compressive strength was incorporated in different clauses of the contract together with requirements for tolerances on the as-built locations of the columns. In principle the idea was that the contractor should supply a grout body with certain dimensions, the method to achieve this was not detailed in the contract.

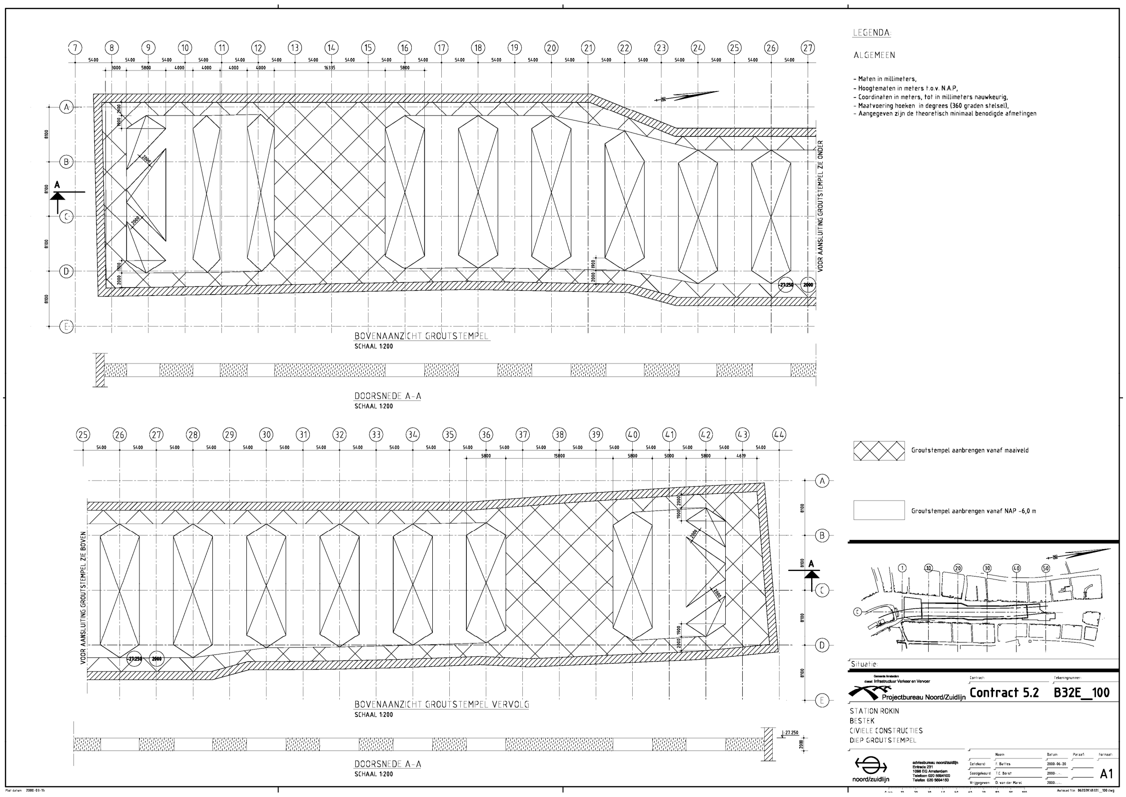

Fig. 6: *Initial design of the grout strut for station box Rokin.*

FIRST AMENDMENT

During the procurement process of the grout works several suggestions for amendments were raised. One of the suggestions was to achieve a larger diameter of the columns by the use of a super-midi jet grout system. To investigate this possibility and to find the maximum achievable diameter in the Eem clay layer, a second trial was carried out in the summer of 2004 on the Rokin station box site, see Figure 7. The reason for this suggestion was that a bigger diameter should lead to minimization of the number of grout columns necessary to create the grout body as defined on the contract drawings. The result of this second trial was that a column diameter of 2.20 m was feasible by using a conventional double system and a diameter of 2.60 m was feasible by the use of the super midi system.

Furthermore there was the phenomenon of the solid grout body as required in the contract. To achieve this grout body, the grout columns should overlap by approximately 30%.

This overlap is inefficient and the idea was raised to abandon the pattern of waling beam and struts and to use the available number of columns to create a continuous slab over the complete area of the station box. This change was made possible by increasing the diameter of the columns to 2.60 m and reducing the overlap from 30 to approximately 10%. In this way a continuous slab was created that only had 90% of the number of columns estimated in the contract. This change however increases the risk of gaps between the grout columns, due to deviations in the inclination and diameter of the columns. Through a statistical analysis a prediction can be made of the percentage of gaps in the grout strut, see Figure 8 and [1].

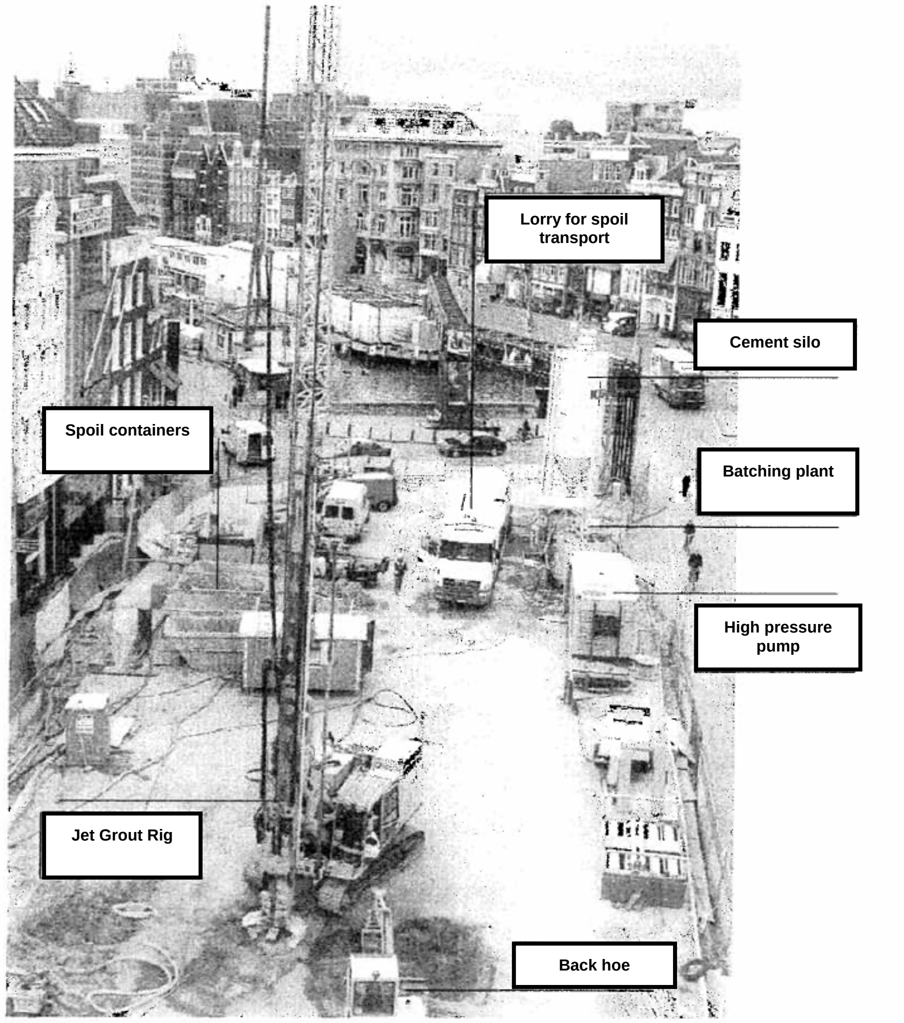

Fig. 7: *Trial at Rokin station box site.*

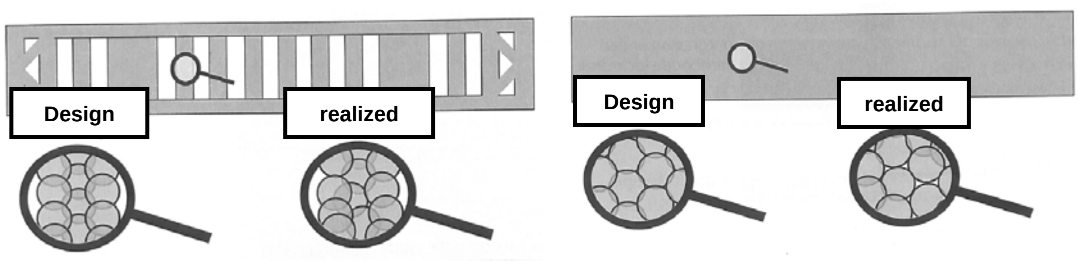

Fig. 8: *From a strut and waling to a continuous grout body.*

In a spreadsheet a part of the grout strut was inputted with the specific tolerances given in the contract, i.e. position of the columns due to inclination of the boring, diameter of the column etc. This sheet, the so-called gap generator, places the columns in the grout strut at random (Monte Carlo method). The result of a single run of the gap generator was then transported to the FEM package Diana, in which the deformations of the grout strut and diaphragm wall were simulated. By repeating this process a thousand times it was possible to find the average, and the upper and lower boundary for the deformation of the grout strut and the horizontal displacement of the diaphragm wall, based on the possible displacements and tolerances of the grout columns, see Figures 9 and 10 and [2].

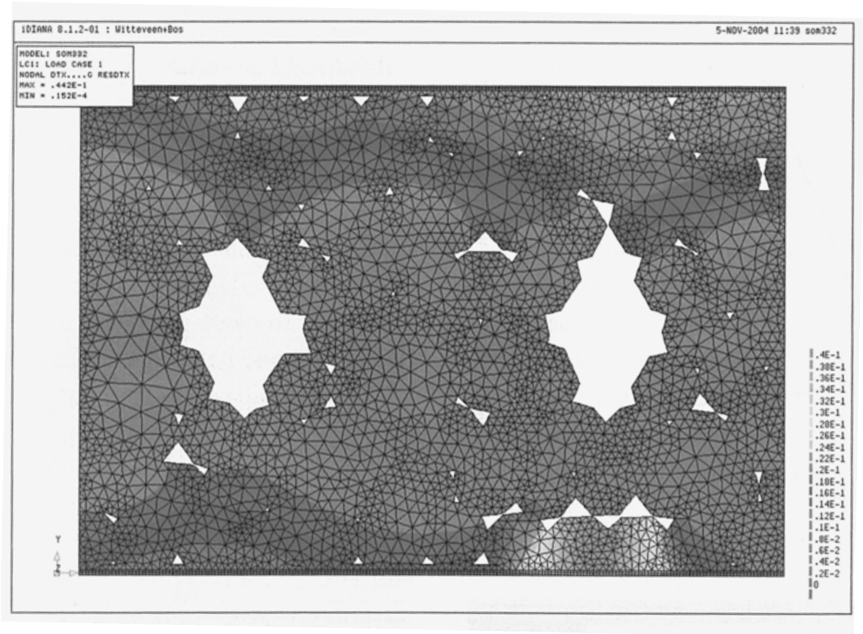

Fig. 9: *The results of the gap generator in the FEM package Diana.*

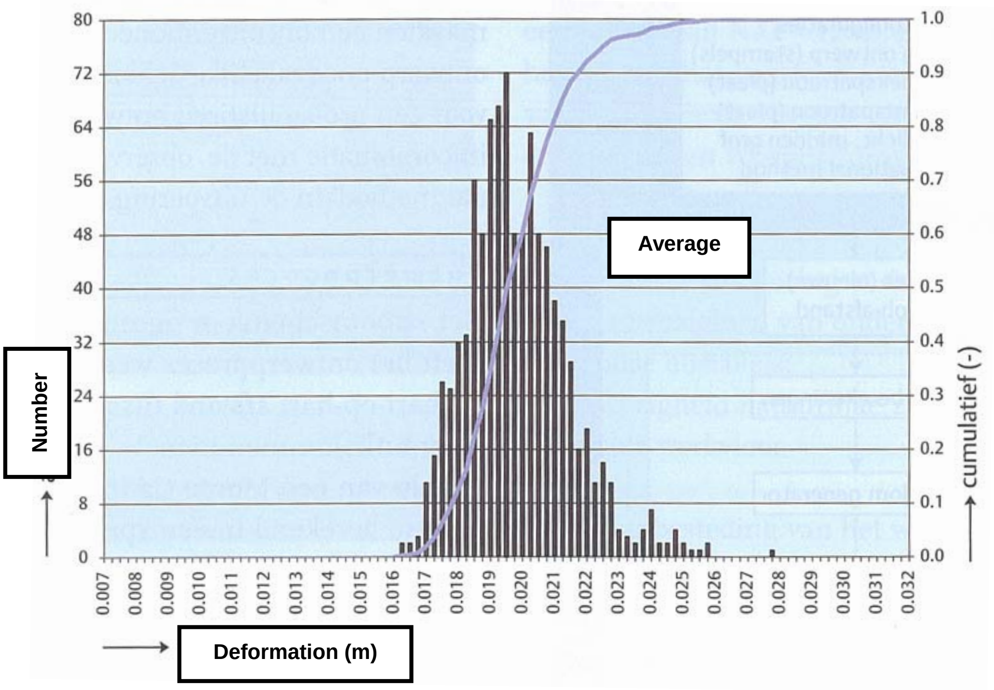

Fig. 10: *Graph of the distribution of deformations of the diaphragm wall.*

With the use of Plaxis the relation between the stiffness of the grout strut, the deformation of the diaphragm wall and the settlement of the first sand layer were determined. Together with the results of the Diana analysis this relation was used to define the upper boundary for the settlement of the first sand layer, and therefore of the historic buildings, see Figure 11.

On this last graph it is shown that the upper boundary of the settlement of the first sand layer is around 20 mm, which is within the boundaries defined in the contractual design.

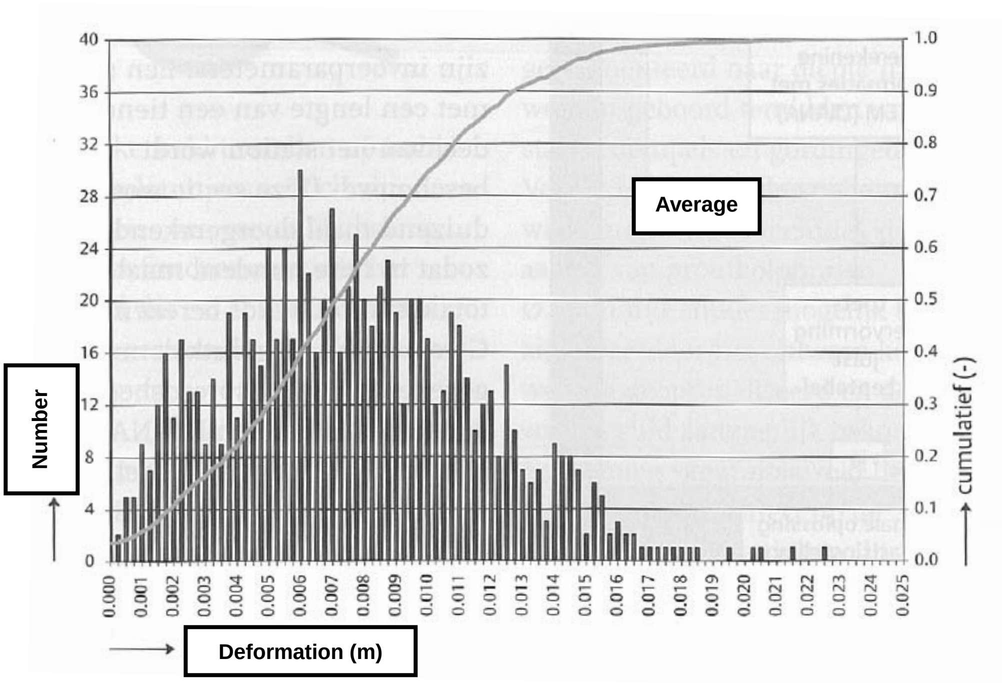

Fig. 11: *Graph of the distribution of deformations of the first sand layer.*

SECOND AMENDMENT

The second amendment was introduced during the procurement process of the jet grout works. In the contractual design it was specified that the major part of the jet grout works should be performed underneath the deck construction. The original sequence of the works was to first construct the deck structure, then to excavate to the level of the first sand layer (approximately 12 m below surface level), and then to make the grout strut. The contractor however considered an alternative sequence in order to speed up the jet grout works. The contractor proposed to perform the jet grout works from surface level prior to the construction of the deck structure. In this way the duration of the jet grout works could be optimized, see Figure 12.

A disadvantage of this proposal however was an increase of the axial force (horizontal reaction force) in the grout strut. This increase in the axial force also leads to an increase in the bending moments and shear forces in the diaphragm walls that were already constructed. This change complicated the design because now there was not only a restriction to the lower boundary of the stiffness, because of the deformations, but also to the upper boundary of the stiffness, due to the strength of the diaphragm wall. The challenge was to keep the stiffness between these two boundaries.

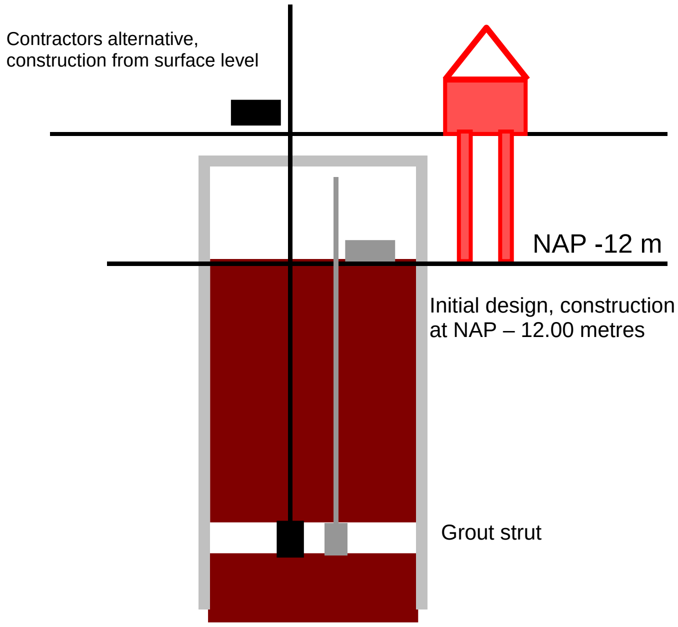

Fig. 12: *Difference between contractual design and the contractor's proposal.*

The initial stiffness of the individual grout columns, and therefore of the complete grout strut, however remained an uncertain parameter in the design process. This was further complicated by the uncertainty about the creep of the material. To control the upper boundary of the stiffness the idea was raised to introduce holes in the middle of the grout strut. See Figure 9 where a part of the grout strut applied with these holes is inputted in an FEM program. A new design for the grout strut was made by introducing a difference in column types, the so-called red and green columns. Green columns were columns to be made without restriction. The red columns were optional. If the stiffness of the grout columns to be produced should prove to be low the red columns had to be made. Red columns could be left out if the stiffness of the grout columns should prove to be high. Through this observational method a controlling tool was created to keep the cumulative stiffness of the grout strut within the narrow allowable boundaries as shown in Figure 13.

This observational method obliged the contractor as well as the designers to have detailed information on the stiffness and strength of the grout columns in an early stage of the project. For this reason the contractor made core borings and took samples from the first columns to be tested in a laboratory on strength, stiffness (including creep) and cement content. On a weekly basis the contractor was informed by the supervisor if red columns had to be made or could be left out. Although the progress of the works was at risk by this method the works were not hampered. Since there were some differences in the design stiffness of the different station boxes the red columns were omitted at Ceintuurbaan and Vijzelgracht but the major part of the red columns were made at Rokin, see Figures 14 and 15.

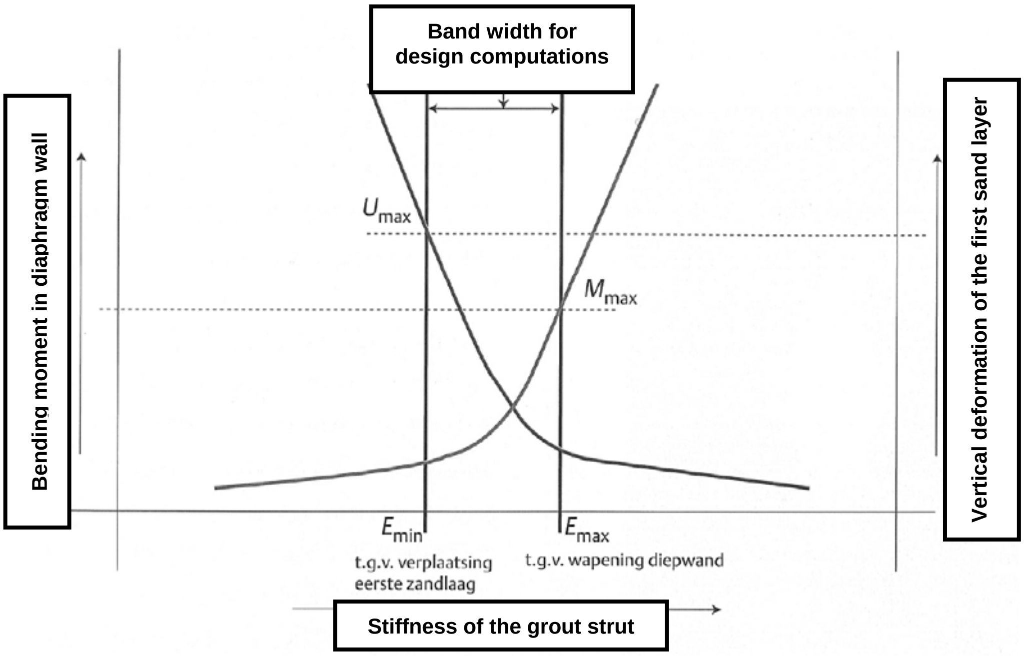

Fig. 13: *Band width for the stiffness of the grout strut.*

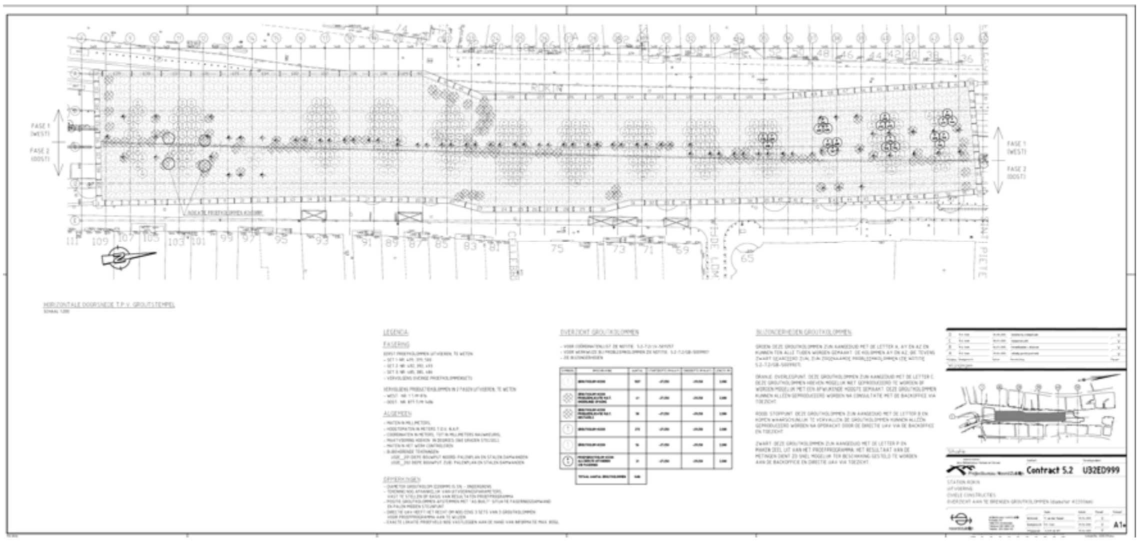

Fig. 14: *Amended lay-out of the grout strut at Rokin with green and red columns.*

THIRD AMENDMENT

Due to a contractual dispute the initial subcontractor for the jet grout works only completed the east part of the jet-grout strut at Vijzelgracht station box. The new subcontractor, which had to build the grout struts at Ceintuurbaan, Vijzelgracht west part and Rokin west and east part, did not have the accessibility to super-midi equipment but only to conventional equipment. For this reason it was no longer possible to proceed with columns with a diameter of 2.60 m. The maximum achievable diameter was defined at 2.20 m. This involved an increase in the number of columns and therefore an increase in production period of the grout struts. Although the works encountered quite some problems during the start-up phase the final production speed satisfied the contractual requirements.

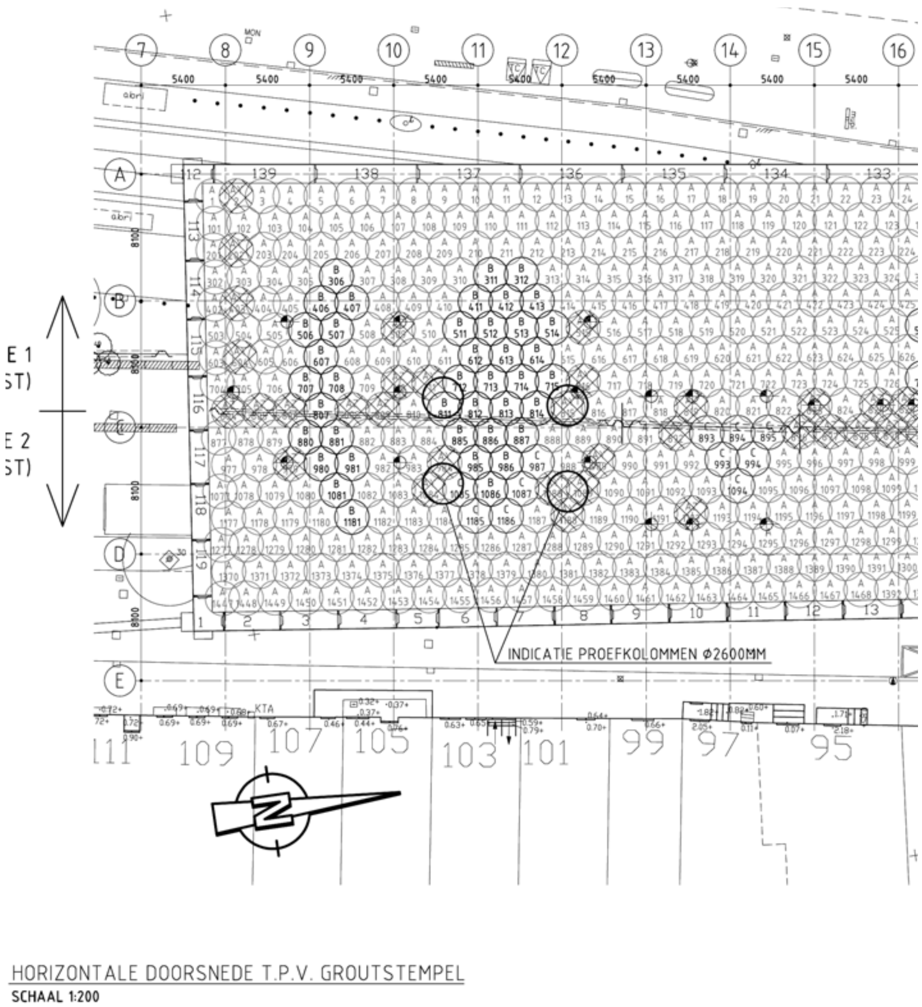

Fig. 15: *Detail of the grout strut at Rokin with green, red and trial columns.*

CONCLUSION

The grout struts were completed in the summer of 2007. During the course of the works several amendments were made to the design. These amendments were mainly introduced because of optimizations in the execution of the grout strut. The changes, together with the uncertainties about the placing of the columns and the parameters of the grout material, made it necessary to choose a somewhat different design approach. In this approach a statistical analysis was used, in combination with an observational method and the possibility to change the lay-out of the strut during the works. With this approach a grout strut was constructed that both met the requirements for the allowable deformations and the allowable forces in the diaphragm wall.

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56 QUALITY CONTROL AND EXECUTION FOR DEEP JET GROUT STRUTS

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SUMMARY: The construction of three deep station boxes in the old city centre of Amsterdam introduces a settlement risk for the surrounding historic buildings. In order to reduce this settlement, risk deformations of the walls of the station boxes should be limited. Limitation of the deformations of the walls can be achieved by constructing struts before excavation of the boxes. These struts have been designed as a deck construction at the surface and as an inner grout strut at a depth of approximately 26 m below surface level. This paper describes the execution of the grout-struts and the applied quality control in order to keep the axial stiffness of the grout structure between certain limits.
Keywords: Amsterdam, deformations, diaphragm wall, jet grout strut, Metro tunnel, settlement, stiffness, tolerances.

INTRODUCTION

For many years the city of Amsterdam has considered the possibility of a metro line from the north to the south of the city. In the seventies the so called East Line was constructed. For this line some parts of the city centre of Amsterdam had to be demolished and there was a lot of political resistance to this removal of buildings. Since the north south line had to cross the old city centre, the local government faced the problem of removal or reconstruction of a lot of historic buildings when constructing this line. For this reason the execution of the project was postponed until new techniques were developed to build an underground metro line without harming the existing city. In the nineties several tunnel boring project in the weak Dutch soil were constructed thus enabling the wish of the city of Amsterdam to construct a metro line in the old city centre without removing buildings in the proposed alignment. By the end of the century the design of the line was

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finished and in the period of 2000 to 2002 several contracts were made with different contracting companies to start building the metro tunnel and the metro lines stations.

Figure 1 shows the alignment of the metro line from the north to the south. As shown on the photograph, the line starts in the northern part of the city, then crosses the river Ij, crossing the central station, followed by the old city centre. After the old city centre the new North-South Line is attached to the existing metro line at Amsterdam South – World Trade Centre. Figure 2 shows the map of Amsterdam with the metro line and the stations.

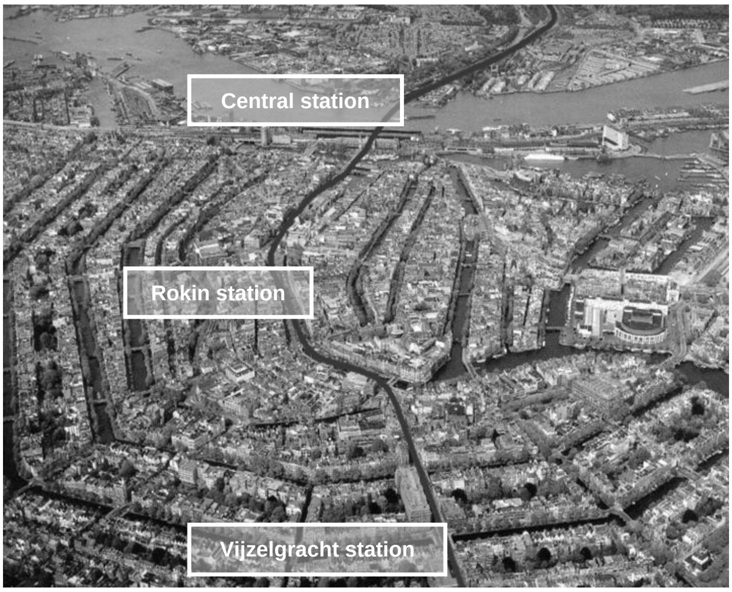


Fig. 1: *Aerial view of the North-South Line in Amsterdam.*

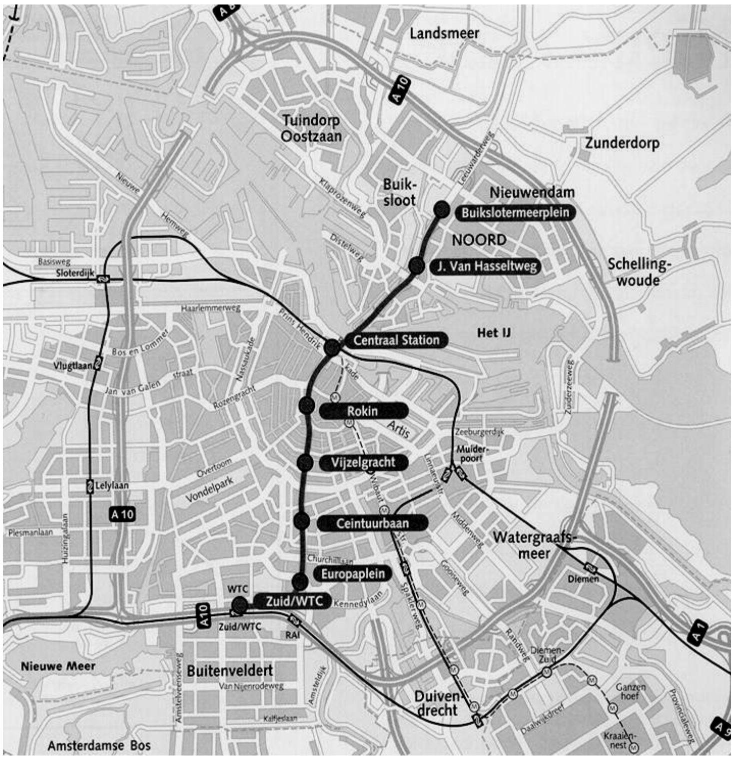


Fig. 2: *Alignment of the North-South Line in Amsterdam with stations.*

As shown on the maps and the accompanying longitudinal section there are two stations in the northern part of the city which are surface stations. The river Ij will be crossed by an immersed tunnel connected to the first underground station to be built underneath the existing Central Station in Amsterdam. In front of the Central Station the entry shaft for the bored tunnel is planned. From this entry shaft the bored tunnel will continue to the southern part of the city where the exit shaft is planned, the exit shaft will also be used as the newly built underground station Europaplein. In between the entry shaft and the exit shaft three new deep station boxes will be constructed, from north to south, Rokin, Vijzelgracht and Ceintuurbaan. The stations will be constructed using diaphragm walls with a reinforced concrete structure inside. Excavation of the station boxes will be done after constructing a deck thus avoiding long time interference with the cities traffic and facilitating a strut construction at the surface prior to the excavation.

Since the station boxes have to accommodate entry and exit of the bored tunnel, the stations are made relatively deep with a platform level of approximately NAP – 21.00 m (Dutch National Datum, Normaal Amsterdams Peil). The stations as well as the bored tunnel follow the pattern of the city streets to avoid as much as possible interference with existing buildings. Because the area available is limited in most cases the station boxes are constructed close to the buildings. These buildings are in general founded on wooden piles which are driven to the first sand layer, a sandy stratum some 13 m below surface level. The bottom level of the station boxes will be significantly deeper than the foundation level of these piles. To prevent settlement of the first sand layer and thus the buildings, deformation of the diaphragm wall must be limited to a minimum. To minimize the deformations of the diaphragm walls the excavation of the station boxes will be made after constructing the stations deck and using steel temporary struts every 5 m of depth. This method is sufficient until the last phase of the excavation. To construct the base slab of the station, the excavation needs to continue to a level of approximately NAP – 26.00 m. However a strut can not be placed due to the construction of the base slab. Further more the soil stratum just underneath the base slab is called the Eem clay, a stiff over consolidated clay layer. This clay layer is not stiff enough to restrain the movements of the diaphragm wall sufficiently in order to prevent settlements of the first sand layer to less than 25 mm. The latter being a boundary condition for the design of the tunnel and the station boxes. For this reason, reinforcement of the clay layer at a level of approximately NAP – 30.00 m is required. This reinforcement is created through forming a strut at the level of the clay layer made of grout. Figure 3 shows the soil and CPT profile which is representative for the three station boxes.

Due to several amendments the axial stiffness of the grout struts became extremely critical, and the only way to finalize the grout-works in a sufficient secure way was by an observational construction method. For this reason an intensive quality control on cement content, stiffness, diameter, tolerances and strength was required. In the following chapters the aforementioned items will be described in detail. This paper should be read in combination with the paper describing the design process of the grout-struts[1].

TRIAL WORKS

The design of the grout strut is described in detail in the sister paper[1]. An initial trial was carried out at Rokin station by Keller Gmbh under a separate advanced contract. The aim of this trial was to confirm the maximum achievable design diameter and the associated

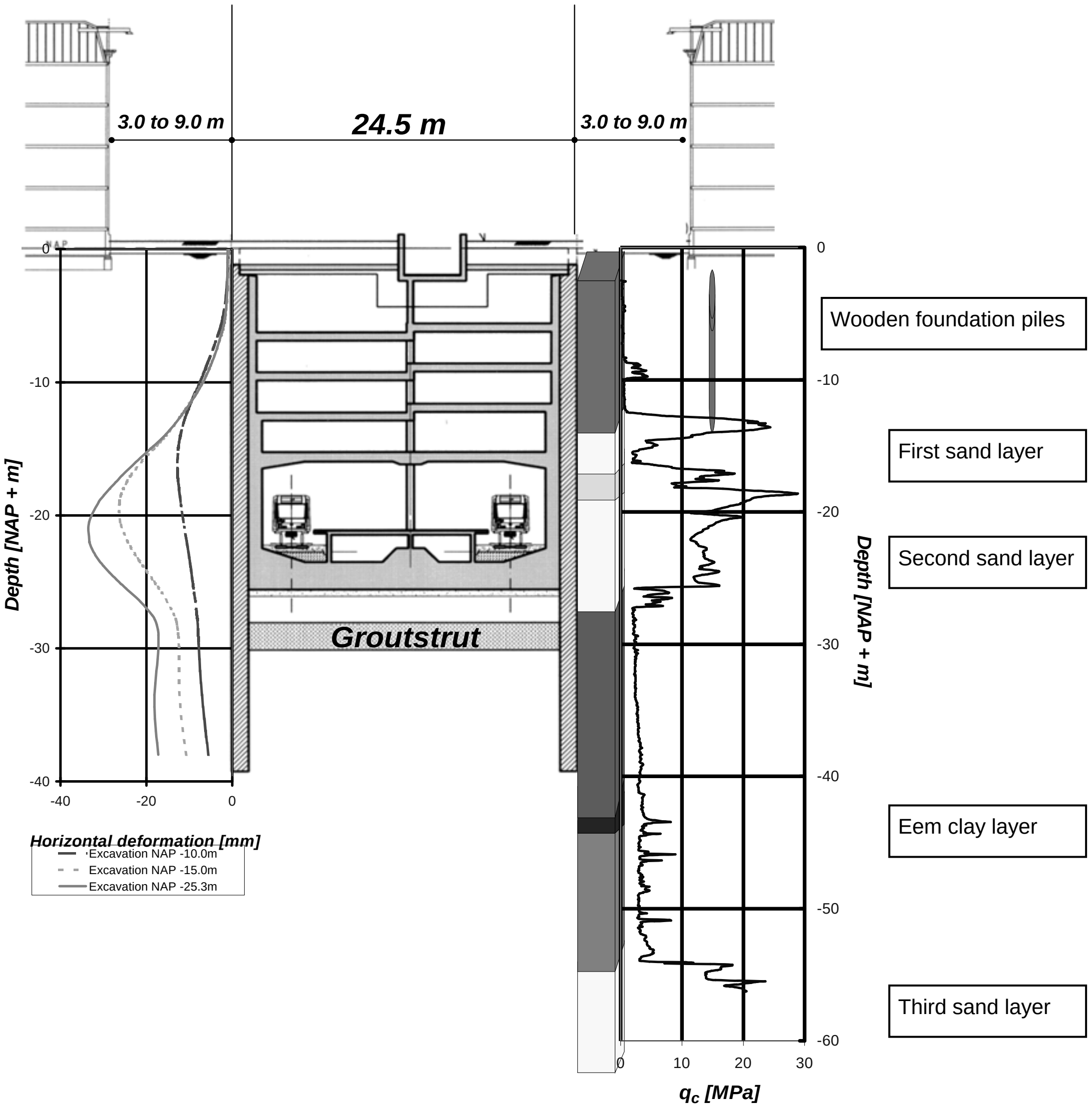


Fig. 3: *Representative soil profile of station boxes (Rokin Station).*

strength and stiffness. In July 2004, 7 columns, 3 m in length, were constructed at a depth of 33 m using both a conventional jet grout system and also using the Super Midi system, a jet grout system licensed by Keller from the Chemical Grouting Company in Japan. The Super-Midi system is to all intents a conventional double system excepting that the monitor or nozzle housing and the nozzles themselves have been specially engineered to improve hydraulic efficiency[2].

The results of the trial indicated that the conventional system was practically limited to about a 2.2 m diameter column whereas the Super-midi system was able to achieve a diameter of 2.6 m. In order to achieve the high strengths required in the clays, the trial columns were constructed in two phases:

- The first phase (Precut) consisted of the construction of the column to the full design diameter using a very weak grout (density 1.16 kg/l) and;
- The second phase (Jet Grout) consisted of reinsertion to the bottom of the column and then injecting a cement rich grout (density 1.75 kg/l) while withdrawing from the bottom of the column.

Using the two phase approach separates the construction of the diameter of the column from the achievement of the required strength. It is also a more cost effective method as the lift speeds required to create the columns in the very stiff Eem clay would be very low and thus would use large amounts of cement which would be wasted by being expelled with the spoil.

The final selected trial columns were cored and laboratory testing for strength and stiffness. The recommendations from the trial were to increase the cement content within the constructed column to improve the strength and stiffness. In order to achieve the required strength and stiffness, the cement content required was calculated to be 550 kg/m^3 of column. This requirement was introduced during the procurement process of Ceintuurbaan, Rokin and Vijzelgracht phase II. Initially the requirement was a minimum allowable compression strength of 5.5 N/mm^2 and a Young's Modulus ranging from 1500 to 2500 N/mm^2.

PRODUCTION JET GROUTING

A joint venture of Keller, Smet and Stumpf (KSS) were awarded the jet grouting sub contract by the station box contractor Max Bogl and site works commenced in December 2004 at Vijzelgracht station. Only the east side of the station box was available with the west side to be constructed at a later stage in the programme.

The contract between the client and the contractor stipulated that there would be cooperation to minimise the cement usage and waste spoil generated as these were to be paid for at unit rates whereas the rest of the column construction was based on a lump sum. This arrangement was possible because of the two stage approach to column construction. The contractor would be responsible for creating the diameter in the precut phase and would cooperate to maximise the required lift speed for the design diameter, whereas the Client could adjust the jet grout phase lift speed to vary the cement content. This was introduced after KSS had stopped the works due to a contractual difference of opinion. Initially the contractor was responsible for transport and disposal of the jet-grout spoil. Later during the procurement process after KKS had resumed working, this aspect was changed. The contractor was responsible for the cement content (550 kg/m^2), diameter and location of the columns and for transport of the jet-grout spoil, the client was responsible for strength, stiffness and disposal of the jet-grout spoil.

Because of the above, an initial trial construction programme was agreed during which diameters, inclination, strength and cement content would be measured. This was restricted to the initial 30 columns.

Following the completion of the first phase of Vijzelgracht, it was decided by the client that a further bidding process should take place for the jet grouting as this was included as a provisiona sum in the bills of quantities. After a competitive bid process, Bilfinger & Berger (B&B) were awarded th jet grouting contracts for Ceintuurbaan and Rokin stations and finally the main contractor, Max Boegl (MB) with technical assistance from Smet completed the second phase of Vijzelgracht. As neither B&B nor MB had access to the super midi system they carried out the jet grouting using a conventional double system with a design minimum diameter of 2.2 m.

Quality control of the jet grouting

The production quality control was prescribed as follows:

- Diameter to be measured in situ for one column in 20;
- Cores were to be taken from one column in twenty and three samples tested for strength and stiffness and;
- Deviation of one column in twenty was to be measured full depth using an inclinometer system run down the actual jet grout rods.

Initial performance was therefore set as follows:

- Minimum diameter 2.6 m;
- Maximum hole deviation 1% of depth;
- Minimum stiffness 750 MPa;
- Minimum strength 5.5 MPa.

As Keller had performed the initial trial, the jet grouting parameters were well known at the start of the works and the main interest was to obtain the required strength and stiffness. Initial jet grout parameters are given in Table 1.

Table 1: *Initial jet grout parameters for Vijzelgracht station box*

Parameter	Precut Phase	Jet Grout Phase
Grout jet pressure (Bar)	360	360
Grout jet flow (l/min)	470	380
Withdrawal rate (cm/min)	7	18
Grout density (kg/l)	1.16	1.75
Air pressure (Bar)	Greater than 8	Greater than 8

The measurement of diameter was by a hydraulically actuated set of calliper arms (known as the "SPIN" or spider) and is described in detail below. Initial measurements confirmed that the diameter was at least 2.6 m and therefore the precut jet grout parameters remained unaltered. The emphasis was thus to reduce the cement and spoil usage for the jet grout phase.

Accordingly columns were constructed with withdrawal rates varied between 18 and 10 cm/min for the jet grout phase in an attempt to establish the maximum acceptable rate. Of the 686 columns constructed, 20 were constructed with a withdrawal rate of 18 cm/min, 277 at 15 cm/min, 312 at 12 cm/min and 77 at 10 cm/min.

Due to contractual problems the initial trial phase was eventually abandoned and production jet grouting continued with the jet grout withdrawal rate being adjusted from time to time based on the results of laboratory testing on retrieved cores.

A number of cores were taken from completed columns at various times including from two columns at a considerable time after construction (about 6 months) in order to gain more information on long term strength. In order to guarantee maximum quality, triple tube coring with a core diameter of at least 100 mm was utilised throughout the project.

Figure 4 shows the variation with strength and jet grout withdrawal rate while Figure 5 shows the variation of tangent modulus with jet grout withdrawal rate.

The results generally show a trend of decreasing strength and stiffness with increasing withdrawal rate as expected as the theoretical column cement content is decreasing. The graphs also show that the required strength and stiffness were easily exceeded.

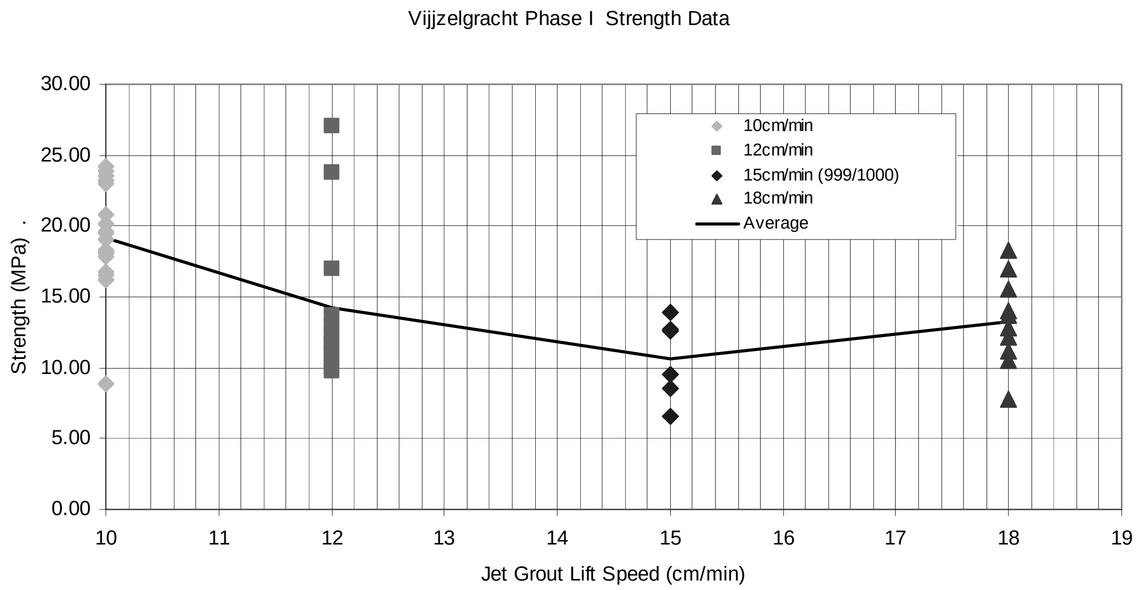


Fig. 4: *Results of core strengths compared to jet grout withdrawal rate.*

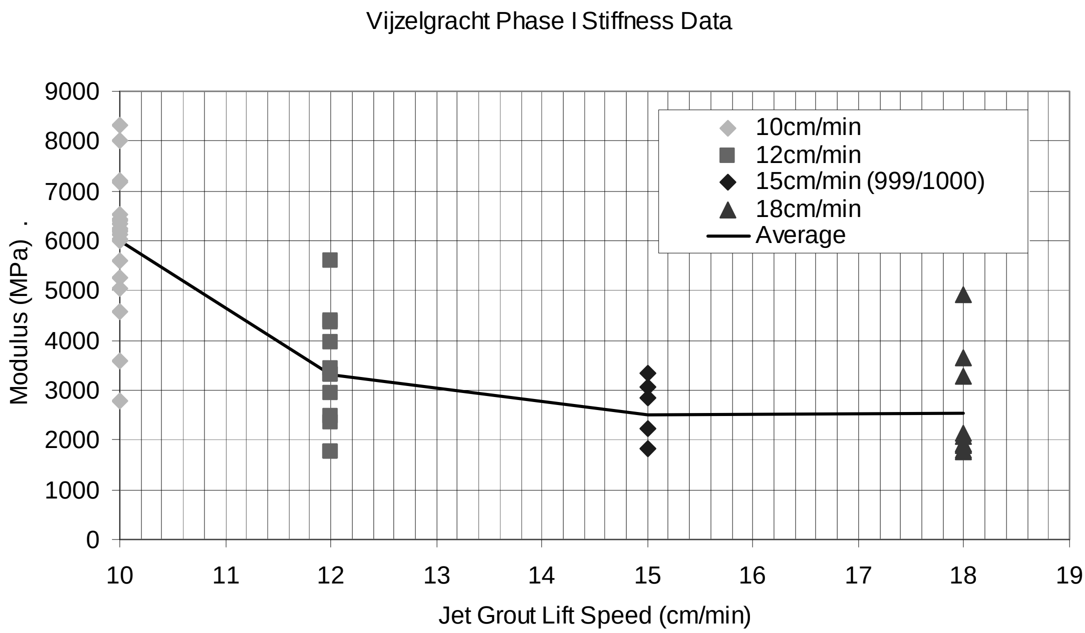


Fig. 5: *Results of core stiffness compared to jet grout withdrawal rate.*

Cement content was measured on most of the core samples. This was accomplished by oven drying a sample of core, crushing it and then determining the calcium content by the use of X-ray diffraction techniques. Samples of the original Eem clay were checked to ensure that there was no calcium content to effect the analysis. Cement contents in over 90% of all samples exceeded the 550 kg/m^3, with some values exceeding 1000 kg/m^3 where certainly neat grout had been present. For all the station boxes the average cement content ranged from 736 to 863 kg/m^3.

The inclination of the jet grout boreholes was measured on all the station boxes. The jet grout rigs used had long masts (generally around 30–35 m in length) and at most only required a single rod change to achieve the design depth. The deviation of the boreholes averaged at between 0.5% and 0.7% with only a few columns exceeding the 1% maximum, generally due to obstructions. The inner diameter of the jet grout rods allowed an inclinometer survey to be carried out down them, at Rokin and Ceintuurbaan station boxes the contractor used a built in inclinometer system manufactured by Lutz that measured every column which gave very good information on the as built layout.

In order to control the quality of the jet grout strut, a database was developed that allowed all relevant parameters to be recorded and checked. Figure 6 shows a typical example of a quality control record for an individual column. The record details all the design parameters and the actual parameters as interpreted from the rig instrumentation data provided by the contractor. The record allows all important quality issues to be recorded and checked and then provides the final column approval for payment. In use, any columns which had wrong or missing quality information was rejected until the contractor could provide the required explanation. Often this was the provision of alternative supporting documentation if the rig instrumentation had malfunctioned.

The database was used on all the station box sites and eventually the site supervision could fill in all the quality data as the works proceeded and thus column quality checking by the client's technical representative (RD Geotech) was simplified as the database automatically generated reports on parameters, diameter and deviation.

Figure 7 shows an example of a report of both precut and jet grout flow rates. This gives a good indication of both diameter and cement content as if the precut flow rate was reduced this would probably indicate a reduced diameter and also if the jet grout flow rate

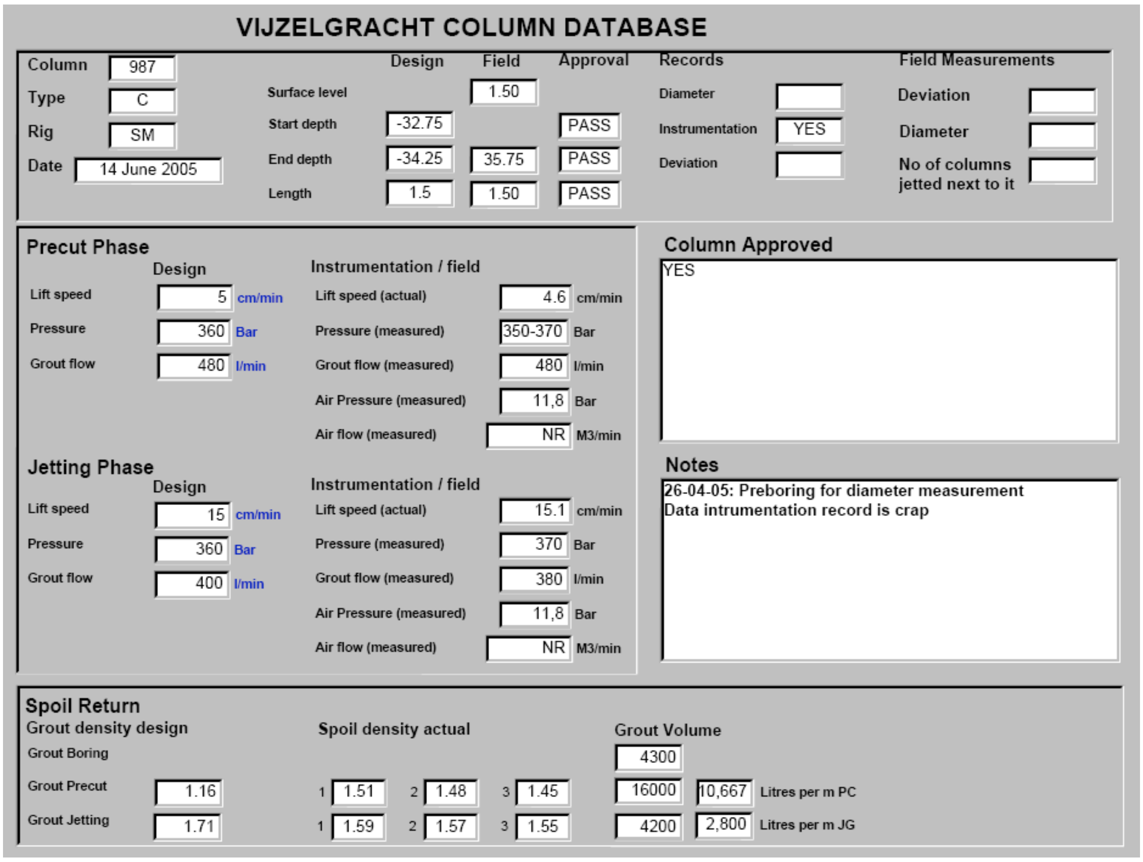


Fig. 6: *Example of database record.*

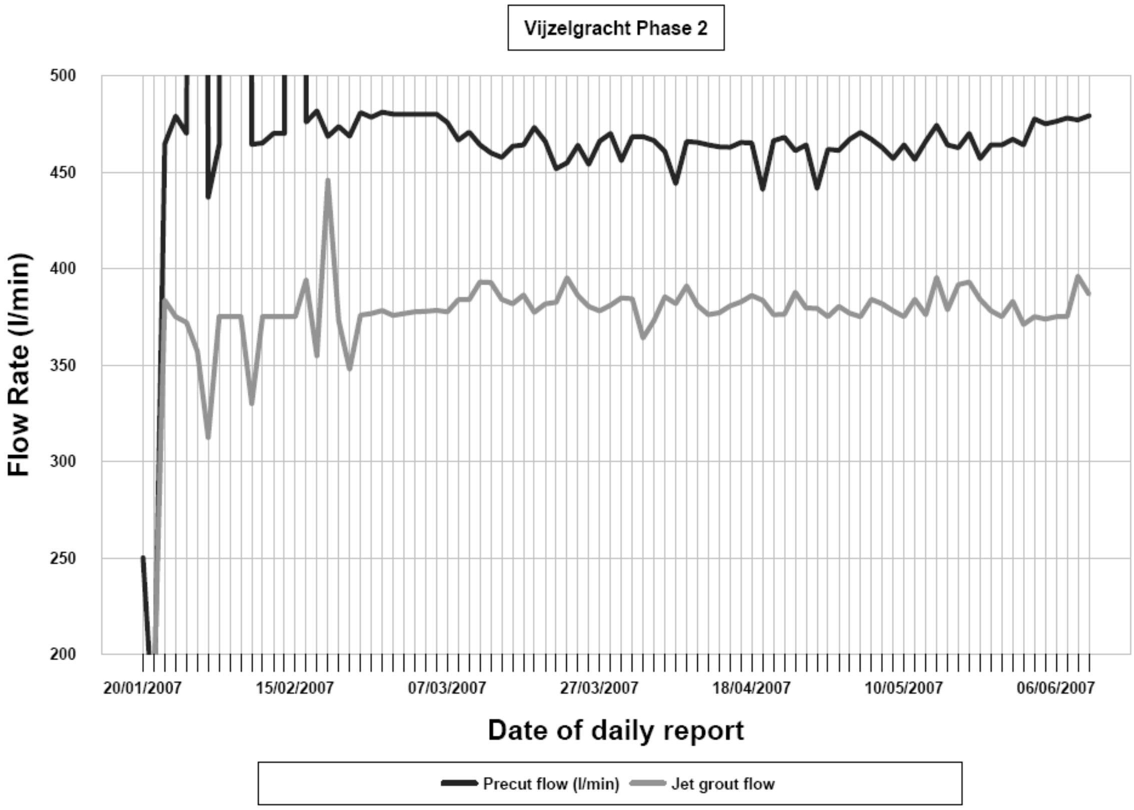


Fig. 7: *Example of database report for quality assurance.*

was reduced this would indicate a reduced strength. Generally a tolerance of 5 litres/minute was allowed on the flow rates, if the flow varied from the design by more than this limit then the column was rejected and was discussed in detail.

Diameter measurement by the "SPIN"

For the trials in 2004, Keller developed a hydraulically actuated calliper system to measure the diameter of the column in-situ at a depth of 35 m. This was a technical feat as it was necessary to ensure that the hydraulic connections were robust and that the callipers could open and close freely to allow the callipers to be withdrawn from the column. Figure 8 shows a photograph of the calliper system with the arms extended. To measure the diameter the arms are extended from a vertical to a horizontal position by the first hydraulic circuit and then the arms extend horizontally by the use of a second hydraulic circuit. The extension of the arms is measured by noting the change in volume of a calibrated piston and as soon as any pressure build up is observed this is taken to be the edge of the column. This system was also used at Vijzelgracht in 2005 for phase 1. In general the callipers were successful but a stabilising weak jet grout column was required over the top 20 m or so to prevent the callipers snagging on the way down on material falling into the borehole. This was normally carried out the day before the actual column was jetted and seemed to provide a solution to the problem.

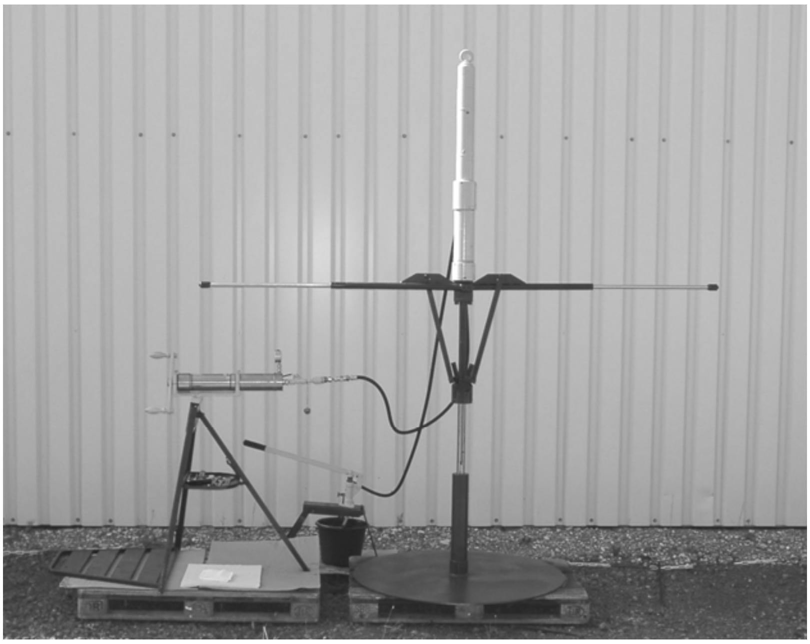


Fig. 8: *Keller Caliper system.*

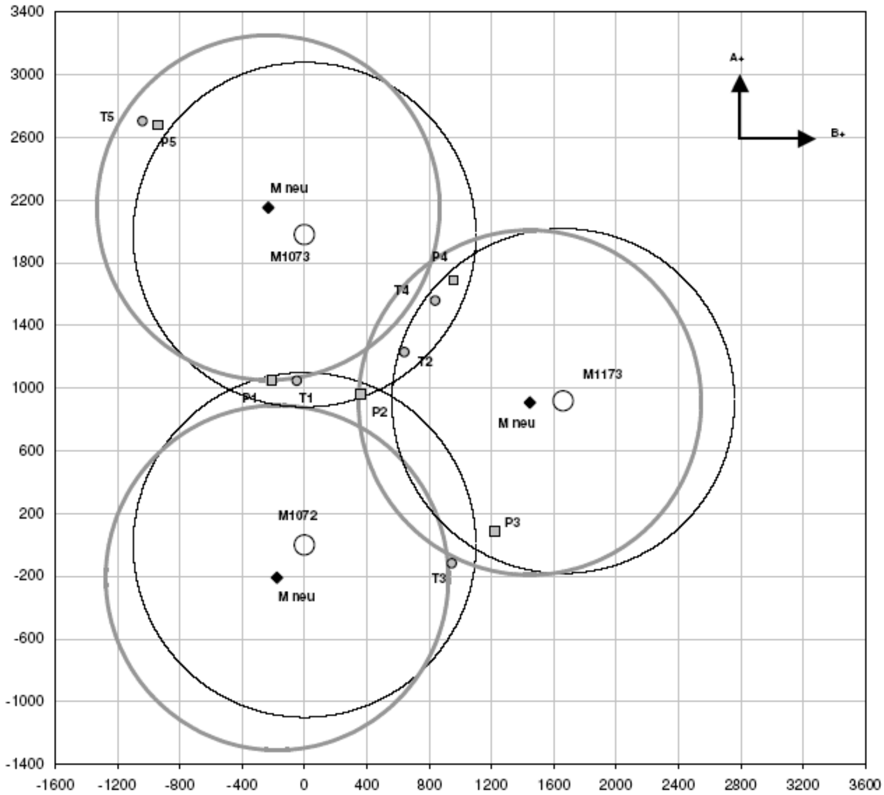
Sampling of jet grout spoil

In addition to the above quality control, the contract required the contractor to take three samples of the return spoil. If the return spoil density is assumed to be identical to that in the column and total mixing is assumed then a calculated diameter can be obtained. In a uniform material this can be reasonably accurate but not accurate enough for diameter measurement unless both the cement content and moisture content are determined in which case the result can be within 20% of the actual diameter. The density of the spoil return should not vary during the works as the precut parameters were not varied and the Eem clay is relatively uniform. Therefore any variation in spoil density could be a sign of variation in diameter and would trigger a more detailed column review.

Diameter measurement by the hydrophones

For Rokin Phases 1 and 2 and Ceintuurbaan station boxes, the jet grouting was carried out by Bilfinger & Berger who had developed there own method of diameter measurement using hydrophones. This functioned by placing microphones around the perimeter of a column and then listening through microphones for the impact of the grout jets. In operation , two or three 50 mm holes were drilled to the depth of the bottom of the column and position just inside and outside the design column radius. In reality, because of the deviation of both column and hydrophone holes this was somewhat of a lottery and in a number of cases, the hydrophones ended up either too close or two far away from a column. Never the less, B&B could interpret the waveform and make some estimation of the position of the hydrophone in relation to the column diameter. Generally sets of three columns were measured which gave more chance of using the hydrophones successfully. Figure 9 shows a test in progress and Figure 10 shows the layout of column and hydrophones. The design diameter in this case was 2.2 m based on a conventional double jet grout system.

Fig. 9*: Diameter measurement by Hydrophone.*



O M1072 Column centre at surface
◆ M1072 Column centre at depth
T3 Hydrophone position at surface
P3 Hydrophone position at depth

Fig. 10*: Column layout for hydrophone tests.*

Stability problems of the construction site

Because of the very soft nature of the upper materials, the stability of the working platform was an issue on some occasions. Although the diaphragm walling plant had been considerably larger and heavier, they had not penetrated the working platform with drill holes. Drilling for the jet grouting had the effect of causing the platform material to fall into the 250–300 mm diameter drillhole and create a crater around the column. Various solutions were developed. At Rokin phase 1, 600 mm diameter steel casings were inserted to a depth of 6 m in advance of the jet grouting by an attendant piling rig while for phase 2, a 250 mm thick concrete slab with light reinforcement was placed over the working area and this then supported the rigs without a problem. At Ceintuurbaan and Vijzelgracht, the contractor filled in the craters each day and this allowed production to carry on without too much disruption.

Stiffness reduction of the first and second sand layer

As part of the excavation of the station boxes, it was necessary to install a number of tower cranes. Driven piles, founded on the 2nd sand layer had been installed previously for support. During erection of the first crane it became clear that the pile capacity was reduced and the crane had to be removed. Additional CPT testing carried out revealed that the cone resistance within the 2nd sand layer was only about 50% of the value prior to station box construction. This was the case for all the station boxes. It would appear that the drilling of the jet grout holes through this layer and the resultant jet grouting has reduced the stiffness of the layer. A number of the piles were load tested and based on these results, a number of further piles were installed to maintain the required load carrying capacity.

This effect should be considered in the future by designers as it could reduce the passive resistance within an excavation leading to unacceptable settlements. Additional numerical analysis was carried out to check the effect of the reduced ground stiffness. The necessary computations still need to be made for Rokin; For Ceintuurbaan W+B concluded that the decrease of the horizontal stiffness of the second sand layer could be compensated by increasing the pre-stress force in the steel tubular struts.

CONCLUSIONS

The quality control of deep jet grouting requires careful consideration. The use of a database recording all approval elements and all relevant jet grout parameters is essential to control the quality assurance process. Equally, in the event of problems, it becomes very simple to isolate the problem columns.

The use of a precut phase to create the column diameter and a following jet grout phase to ensure the correct strength allows the designer to specify diameter and strength independently for any soil type and is a significant advance over previous experience where achieved strengths in clays were low.

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Proceedings of the Institution of Civil Engineers
Geotechnical Engineering 157
July 2004 Issue GE3
Pages 101–106

Paper 13382

Received 20/05/2003
Accepted 12/01/2004

Keywords:
cofferdams & caissons/grouting/tunnels & tunnelling

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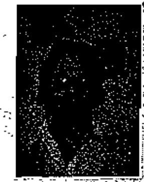

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Wastewater flow transfer tunnel: design and construction

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This paper discusses the design and construction of a 10·5 km long, segmentally lined, flow transfer tunnel. It covers the detailed lining and shaft designs, selection criteria for the tunnelling machines, shaft and tunnel construction methods, and specialist ground treatment. The paper discusses several construction problems encountered with shaft sinking and tunnelling, and describes how these were resolved. Particular note is made of caisson sinking of shafts and the design of the tunnel entry and exit portal eyes. Jet grouting in the varying ground strata around these portals and on other areas of the contract is considered.

1. INTRODUCTION

This paper is the first in a series of three papers on the Kingston upon Hull wastewater tunnel in England.

Hull's combined sewerage system was originally designed using brick-lined, egg-shaped sewers. The early gravity sewer outfalls into the Humber Estuary were tide-locked at every high tide, so the sewer system needed a large storage capacity. As the city developed, deeper-level interceptor sewers were constructed to collect the increased flows from both West and East Hull (plus additional land drainage), and they terminated at two new outfall pumping stations. These stations were constructed in the 1950s and 1970s respectively, and pumped screened, untreated flows into the estuary. Large areas of Hull lie below the high spring tide levels, and they are totally dependent on these ageing stations for both drainage and the avoidance of flooding. In order to reduce risk of future flooding, and to meet European directives on sewage treatment and outfall pumping, Yorkshire Water commissioned a multidisciplinary design team to determine an outline scheme for transferring flows to a new treatment works prior to pumping outflows into the Humber.

2. OUTLINE DESIGN

A hydroworks computerised flow model encapsulating the entire Hull catchment formed the basis of the scheme design. The design parameters were to provide treatment and outflow pumping for 25 m^3/s against the maximum high tide, along with a storage capacity of at least 90 000 m^3. These measures are designed to cater for a 1 in 30 year storm event. See Fig. 1.

Both pumped and gravity flow schemes were considered as solutions. The whole-life costing of a gravity flow tunnelled scheme, with its minimal maintenance costs, proved to be a more cost-effective solution than a pumped tunnel or pipeline scheme. The chosen design is similar in its concept to many of the new coastal flood and pollution alleviation schemes. Based on the results from the hydroworks model, an interceptor

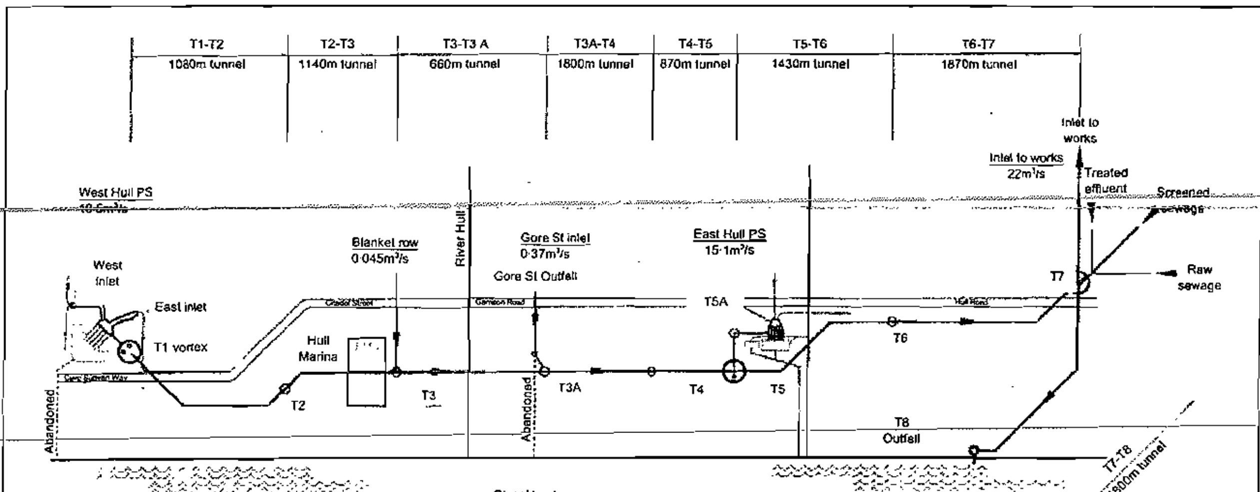


Fig. 1. Schematic layout of Hull flow transfer scheme

tunnel of 3·6 m internal diameter and 8·8 km long runs parallel to the coastline and carries the incoming flows by gravity to a new treatment works. The treated flows are then pumped, using an inverted siphon arrangement of shafts and tunnel, 1·7 km to the outfall structure at the River Humber. The gradients of the tunnels, at 1 in 1000 and 1 in 2000, are very shallow. This is due to a combination of having a long tunnel with flat ground profile, the desire to avoid the chalk aquifer at depth, and the need to limit the head for pumping the flows from the tunnel up to ground level for treatment. Such shallow gradients can encourage grit and silt build-up, but the high-volume first flush storm flows were considered sufficient to self-cleanse the tunnel. The inlets from the two existing higher-level pumping stations to the tunnel are through vortex drop shafts.

3. DETAILED DESIGN

The £52·5 million contract for the design and construction of the tunnel and shafts was awarded to Morgan Est Tunnelling Division (formerly Miller), which had been an integral member of the outline design team. The treatment works were let under a separate contract. The design of the tunnels, shafts, flow controls and all temporary works was undertaken in-house by Morgan Est's Tunnelling Design Department, with the outfall structure and pump station modifications design being sublet to Mott MacDonald (Sheffield).

3.1. Ground conditions

The ground conditions for tunnelling comprised mainly normally consolidated glacial clays to the west and medium dense glacial sands and gravels to the east. In the centre of the old city, adjacent to the River Hull, a localised band of alluvial sands and clays entered the tunnel horizon, and these were instrumental in a subsequent collapse of the tunnel (a matter that will be addressed in a subsequent paper). The sands and gravels possessed a high silt content, which would make shaft construction by the wet sunk caisson method more difficult. The silt would be held in suspension in the shaft caisson balancing water, and would prove to cause problems in tremying the shaft bases. The hydrostatic groundwater head in the sands and gravels was 2 bar at tunnel axis, and although not shown on the SI reports, it was subsequently found to be linked to the tide levels in the adjacent River Humber.

3.2. TBM selection

The variable nature of the soils at the tunnel horizon, combined with the 2 bar of groundwater pressure and a requirement to limit surface settlement through an urban area, led to a mixed face earth pressure balance (EPB) tunnelling machine being selected. A pair of identical Lovat RME167SE tunnel boring machines (TBMs) were utilised for this contract (Fig. 2). These machines could be driven in full EPB mode (where the pressure generated in the TBM head balances or exceeds the external soil pressure, thus reducing both volume loss and ground settlement), or in open mode. It is questionable whether true full EPB operation was actually achieved in the clay, owing to its tendency to form lumps when mined; however, surface settlement was maintained within acceptable limits throughout the tunnel drives.

These TBMs were 4·226 m nominal diameter, and with backup sledges weighed 236 t each. The head of the TBMs could be articulated by up to 2°, which allowed the machine to negotiate curves down to 200 m radius. They were each powered by a dedicated 3·3 kV electrical supply, drawing 1000 kW of power when mining. To cater for the mixed ground conditions the cutting head was fitted with ripper teeth, scraping teeth and disc cutters. Regular replacement of teeth was necessary, and was carried out at the intermediate shafts and at predetermined maintenance stops. The TBMs incorporated ports to allow the addition of both foams and polymers into both the plenum chamber and the screw in order to condition the excavated soils and generate a plug seal in the screw when mixed with the clays and sands. The machines were driven 24 h per day on two 12 h shifts, and advance rates of 28 m per shift were achieved in both the clays and the sands, with annulus grouting being carried out through the rings. This rate of progress compared well with the predicted rate of 14 m per shift.

Fig. 2. Lovat 4·1 m diameter EPB tunnel boring machine

Spoil handling from the plenum chamber was by screw conveyor, onto a belt conveyor and into muck skips for transportation to the surface. Two dedicated tips were available, and no treatment of the spoil was deemed necessary prior to tipping.

3.3. Tunnel lining design and construction

The tunnel lining was designed by Morgan Est in conjunction with Charcon Tunnels, who manufactured the rings. Two types of 250 mm thick one-pass reinforced concrete ring were developed. The ring was designed using the Muir-Wood–Curtis approach for a tunnel in elastic ground, and caters for K_0 values between 0·7 for the clays and 0·5 for sand. The ring was a six-plate, fully tapered, trapezoidal design with a single ethylene propylene diene monomer (EPDM) gasket located close to the extrados. It was known that problems had occurred on other schemes with placing the last segment of a six-plate ring, and careful attention was paid to the segment taper design in conjunction with the ram stroke of the TBMs in order not to replicate these problems. Shear pads were cast into the circle joints to help ensure a good alignment and build of the ring. However, in practice some problems were encountered when these pads actually caused bursting of the concrete if a tight fit or ring misalignment occurred at the circle joints. The benefit of incorporating shear pads for the alignment of rings is debatable, and several other ring connection methods are currently being tested by tunnel ring

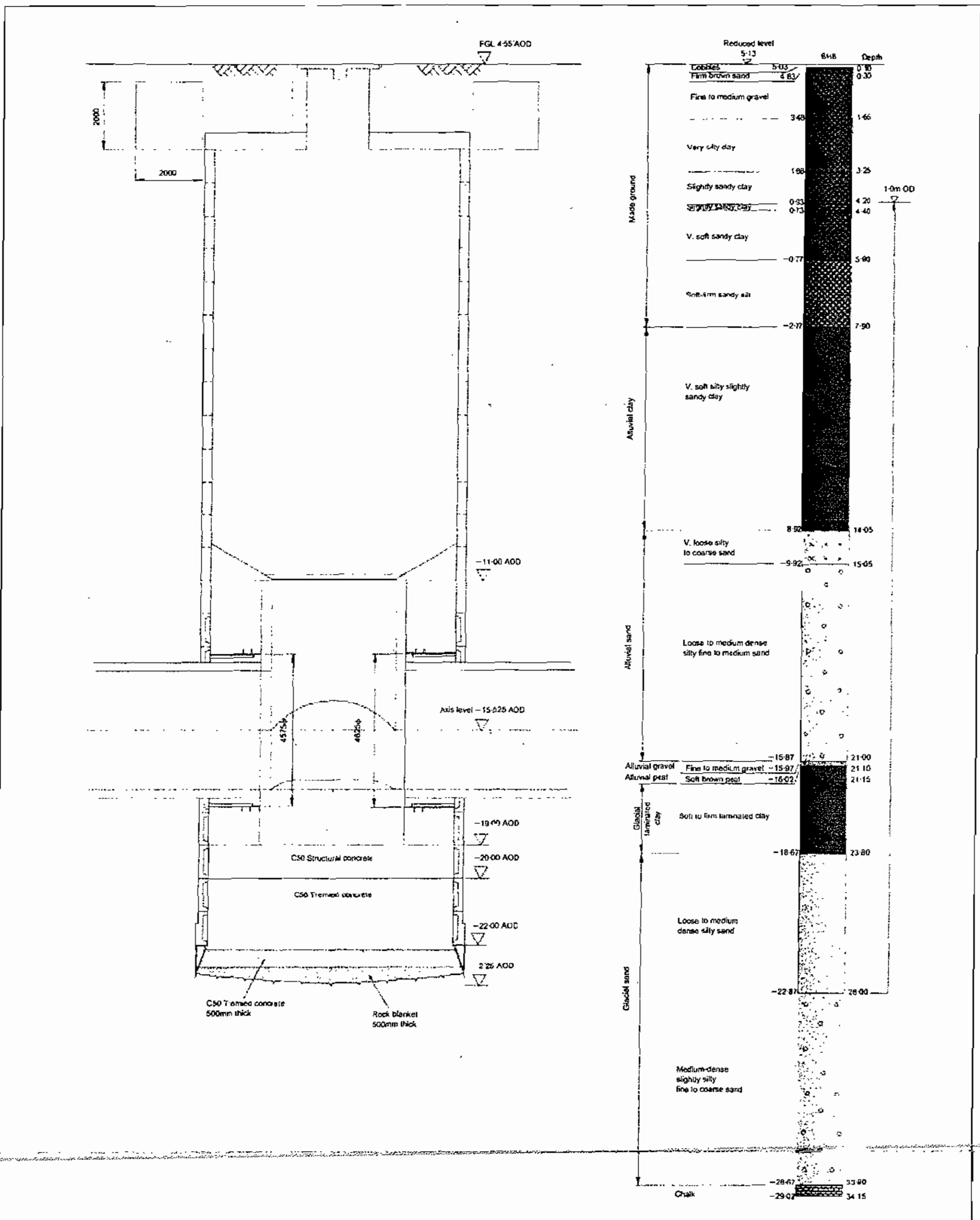


Fig. 3. Typical shaft layout and associated ground conditions (dimensions in mm, elevations in m)

manufacturers. In a departure from conventional ring manufacture, the segments were vertically cast by Charcon/Tarmac using a carousel system. The congestion of rebar and grouting inserts created a problem in achieving the design cover to the building bolts. The bolts would therefore be removed after grouting the annulus and the bolt holes grouted up to ensure that the design cover was achieved to the rebar adjacent to the bolt holes. The rings were constructed using the segment erector in the TBM, which handled the segments using a screwed-in lifting ball located in an insert at the centre of each plate. The same insert (complete with one-way acting valve) was utilised for grouting the annulus.

3.4. Shaft design and construction

A total of 10 shafts of 7·5 m, 10·3 m or 12·5 m internal diameter were designed (Fig. 3), including one additional shaft for TMB recovery purposes. All the shafts were designed to be jacked down as wet caissons with a bentonite-filled external annulus, because every shaft passed through or was founded in the water-bearing glacial sands and gravels (Fig. 4). The jacking collars and jacking post arrangements were a Morgan Est design, and sacrificial steel cutting edges were also designed for each shaft (Fig. 5). Charcon one-pass shaft rings with a modified cover to meet the design durability requirements were utilised. These rings were developed on previous contracts to deliver bentonite down the centre of the segments to a collar, which injects the bentonite into the extrados annulus just above the cutting edge where it is needed.

This system worked successfully, and the same system could then be utilised to inject a 4 : 1 PFA : cement grout into the annulus to displace the bentonite once the shaft had reached formation level. Back-grouting of the annulus by this method proved partially successful. This system requires a substantial grout supply to be on hand in order to pump grout continuously to fill the annulus. If stoppages occur then the grout can quickly set in the tubes, preventing further grouting. Back-grouting through the rings is then required to augment this system and to ensure that the annulus is filled with grout.

3.5. Shaft sinking by the wet caisson method

The shaft bases were designed in two stages. An initial mass concrete tremied plug was installed under water to allow the shaft to be pumped dry. Owing to the uncertainties of the concrete quality (the concrete having been poured under 20 m head of very silty water), a reinforced concrete base designed to resist full hydrostatic pressure was cast in the dry above the tremied plug to ensure durability for 120 years. The tunnel portal structure was cast internally, and was designed as a plate with integral holes. The design of this structure was carried out in-house using finite element methods, with the portal structure keyed into the shaft. In practice, the portal structure was constructed with the launch and reception steel sealing cans being cast into the portal concrete (Fig. 6).

Soft eyes could not be incorporated in the shaft owing to the high jacking loads required. The eyes were broken out prior to the launch/arrival of the TBM with the ground held back by an arrangement of jet-grouted ground and soft piles outside the shaft. At the T2A recovery shaft, these piles were replaced by an ice wall design. This decision was based on the success of an ice wall used to recover the collapse of the tunnel, as will be described in a subsequent paper. This method of shaft construction proved to be very successful; however, caution is advised with regard to constructing deep tremied bases. The silt content in the shaft balancing water was very high as a result of the excavation process, and this led to a layer of silt being formed at the bottom of the shaft. A range of chemical flocculants were added to encourage the silt to settle out of suspension, and several 'hoovering' operations of the formation were carried out with submersible pumps prior to pouring the base slab.

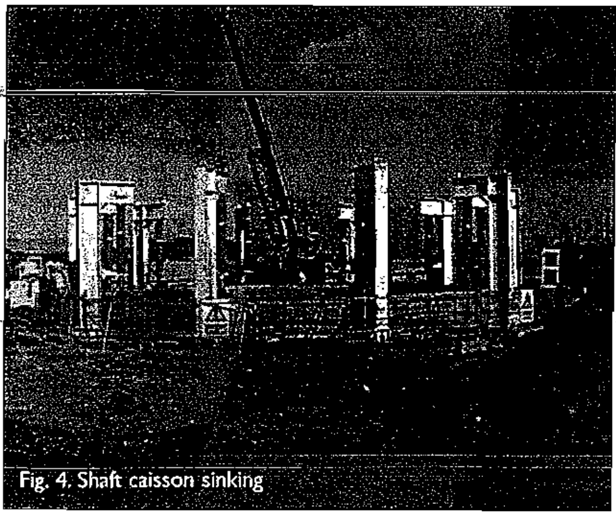


Fig. 4. Shaft caisson sinking

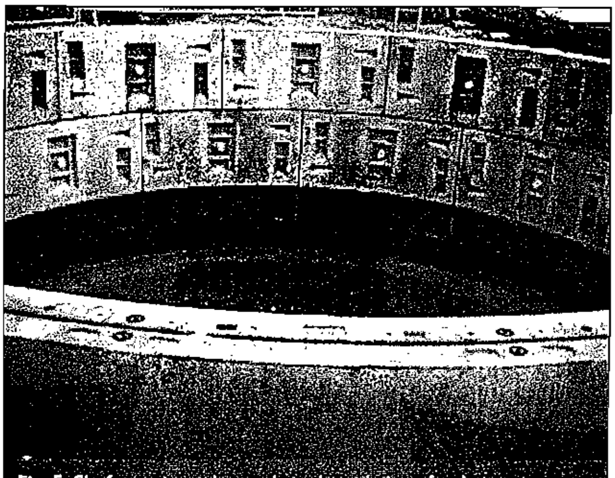


Fig. 5. Shaft cutting edge and pocketed rings for base connection

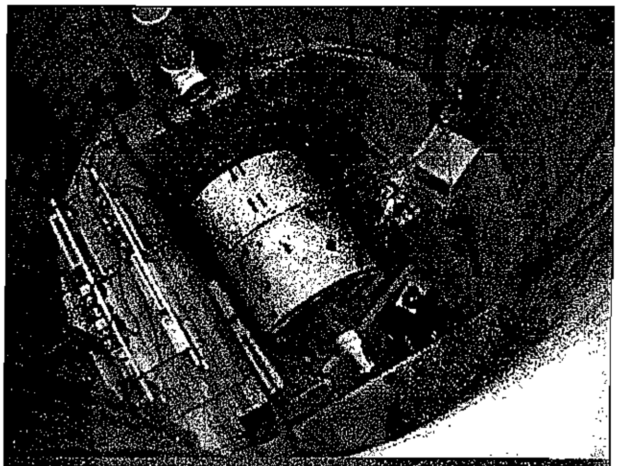


Fig. 6. Launching the TBM

Divers were required to operate the tremie, and working at 20 m depth severely limited their dive time. The tremying operation caused whatever silt was still lying in the formation to form a bow wave in front of the concrete, and it became trapped and compacted against the shaft wall. To the divers, working in darkness, the compacted silt is indistinguishable from wet concrete. On dewatering of the shaft this silt pocket blew out, causing an inrush of groundwater. A ring of deep wells around the shaft was required to lower the water table and allow construction of the reinforced base. The design solution for subsequent shafts was to add flocculant, 'hoover' the base, and then add a stone blanket, 500 mm thick, to the formation to control the movement of any remaining silt. This was covered with geotextile, which prevented any silt from passing through. The geotextile was fixed to a steel frame to resist buoyancy. This proved to be a successful solution, as no further problems were encountered.

The operation of the tunnel under dry weather flows relies on the shafts to vent and to draw in air to prevent vacuum locking. All shafts are designed with a replaceable activated carbon odour control system. This can be bypassed under storm flow conditions, which will cause large volumes of air to be rapidly expelled as the surge wave propagates backwards up the tunnel.

4. GROUND TREATMENT

The combination of the granular ground conditions and high groundwater pressure at the tunnel horizon led to the need to provide stabilised ground at critical sections of the tunnel construction. Ground treatment was utilised for three construction activities: the backshunt connecting tunnel, the soft eye construction at the shafts, and the TBM maintenance blocks between shaft locations.

4.1. T7 backshunt

The backshunt tunnel is 47 m long, and connects the downstream shaft of the flow transfer works (T7) to the newly constructed pumping station in the wastewater treatment works (WWTW). This section of tunnel was constructed first to provide a backshunt tunnel that would accelerate the launch of the TBM from T7 travelling westward.

The tunnel was constructed as an open-faced hand drive, 4·2 m in diameter, with a precast segmental primary lining and an in-situ concrete secondary lining to 3·6 m internal diameter that was constructed after completion of the main tunnel drives. The tunnel horizon is located approximately 22 m below ground level within the glacial sand and gravel stratum, with a hydrostatic groundwater pressure of 2 bar.

Ground treatment of sands and gravels comprised jet-grouted interlocking columns of a design diameter of 1·2 m and at a spacing of 1·1 m. The depth of the block was 8 m, thus providing 2m of stabilised ground around the extrados of the tunnel. The jet-grouting process comprised a pressurised jet of water to 'cut' the ground into a loose state, which was then injected with cementitious grout. The depth at which the columns were being constructed was one of the greatest ever carried out in the UK. The interlocking of the columns was crucial to the success of the treatment in providing a stable face with no flowing water to the tunnel drive.

During construction, a series of small inundations of material occurred where ungrouted sands and gravels flowed into the tunnel under pressure. The material choked itself, and the water pressure dissipated over time, suggesting that only pockets of pressurised material had been encountered. However, owing to the paramount consideration of safety during the tunnel construction, further interlocking columns were constructed along the perimeter of the grouted block in conjunction with a series of dewatering wells. Tunnel construction continued successfully, and the launch of the TBM was subjected to minimum delay. Constructing the tunnel through the jet-grouted columns gave a rare insight into the 'black art' of ground treatment. The boundaries of the columns were easily identifiable where they had not interlocked, and where interlocking had occurred the grouted face was of a consistent nature.

The areas where interlocking of the columns had not occurred, as seen in the tunnel, were probably a result of the higher actual mass density of the ground when compared with the standard penetration test (SPT) results from the site investigation used for column design. The sensitivity of the column diameter to the density of the ground was highlighted by these localised sections of reduced diameter, and is an important consideration during the design. Although the verticality tolerance of the columns is as accurate as for conventional piling methods, the depth of the columns exaggerated any off set from vertical that might have contributed to the presence of pockets of ungrouted ground. It did, however, provide an important lesson in the need to provide overlap in the column layout design to take into account the worst-case drilling tolerance scenario.

4.2. Shaft eyes

As detailed earlier, the shafts were constructed as wet caissons using a precast concrete one-pass lining. These were not suitable to drive the TBM through, so soft eyes were required for the TBM junction. To prevent the inundation of the ground while this was carried out a jet-grouted/soft pile block was constructed adjacent to the shaft comprising respectively 1200 mm and 1400 mm diameter jet-grouted columns and 900 mm diameter soft piles of weak concrete mix. See Fig. 7.

The soft eyes were very successful in that no ground was lost during TBM entry or exit. The presence of the piles seemed to provide positive support to the jet grouting process. Stable dry ground was encountered, which gave positive ground support, so that the shaft lining could be removed prior to the arrival of the TBM.

4.3. Maintenance blocks

Maintenance blocks are stabilised areas of ground between the shaft locations where access to the head of the TBM can be

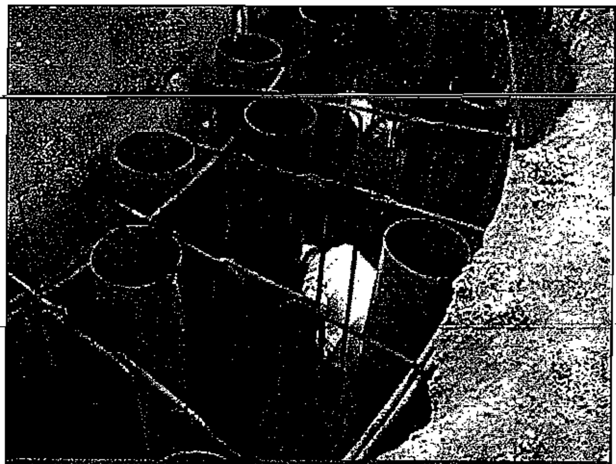


Fig. 7. Shaft jacking collar showing inserts for jet grouting

gained to change the cutting tools. Access to the front of the TBM is via a porthole in the bulkhead that separates the pressurised chamber where the EPB is controlled from the rest of the tunnel, which remains at atmospheric pressure.

Eight maintenance blocks were required in total. Two of these blocks were constructed using the jet grouting techniques utilised for the backshunt. However, in the light of the difficulties encountered with the backshunt tunnel, combined with the even more restricted access to the head of the TBM, a review of the construction method used was carried out by the project team. It was decided to change the construction method to a combination of low-pressure compressed air (LPCA) and dewatering, for a number of reasons:

(*a*) The LPCA/dewatering method was flexible, in that the level of LPCA could be tailored to the actual ground conditions.
(*b*) Dewatering was a less intrusive method of controlling water pressure, and was only temporary; jet grouting would leave a 'hard' area around the tunnel lining.
(*c*) The cost of dewatering and LPCA was comparable to that of the jet grouting, provided that the decision to use this method could be made early to spread the capital cost of the LPCA bulkheads over a larger number of maintenance blocks.
(*d*) Disposing of the water was much easier than dealing with waste grout, thus reducing the impact on the surrounding environment.
(*e*) The presence of the LPCA bulkheads within the tunnel gave extra protection if any problem arose outside the maintenance block locations. This was to become very important in the rapid control of the tunnel collapse that will be discussed in a subsequent paper.

Further grouting was carried out at the maintenance blocks that had been constructed earlier using jet grout columns in order to provide more confidence that the blocks would be stable. This was combined with dewatering wells to reduce the groundwater pressure to which the block was subjected. Both methods used for the maintenance blocks were successful in providing safe access to the head of the TBM. However, control and disposal of waste grout were more problematic than providing temporary water discharge for the dewatering. By using the dewatering to reduce the groundwater level, the level of LPCA utilised was generally below 1 bar, thus minimising the potential adverse health problems for the operatives.

5. CONCLUSIONS

The design and construction of the tunnel using wet sunk caisson shafts, a tapered trapezoidal ring and a robust earth pressure balancing TBM proved to be a successful combination.

The use of different ground treatment techniques was a critical activity in the successful completion of the works. The ground conditions, coupled with the extremely onerous groundwater regime, meant that the risk in carrying out underground works was high, and thus ground treatment had to deliver safe solutions. Problems were encountered in the early stages of the project, but the teamworking approach ensured that value engineering continued throughout the works to provide the best solution for the client.

Please email, fax or post your discussion contributions to the secretary: email: mary.henderson@ice.org.uk; fax: +44 (0)20 665 2294; or post to Mary Henderson, Journals Department, Institution of Civil Engineers, 1–7 Great George Street, London SW1P 3AA.

OBJ3/P3/C2

Transport and Works (Inquiries Procedure) Rules 2004

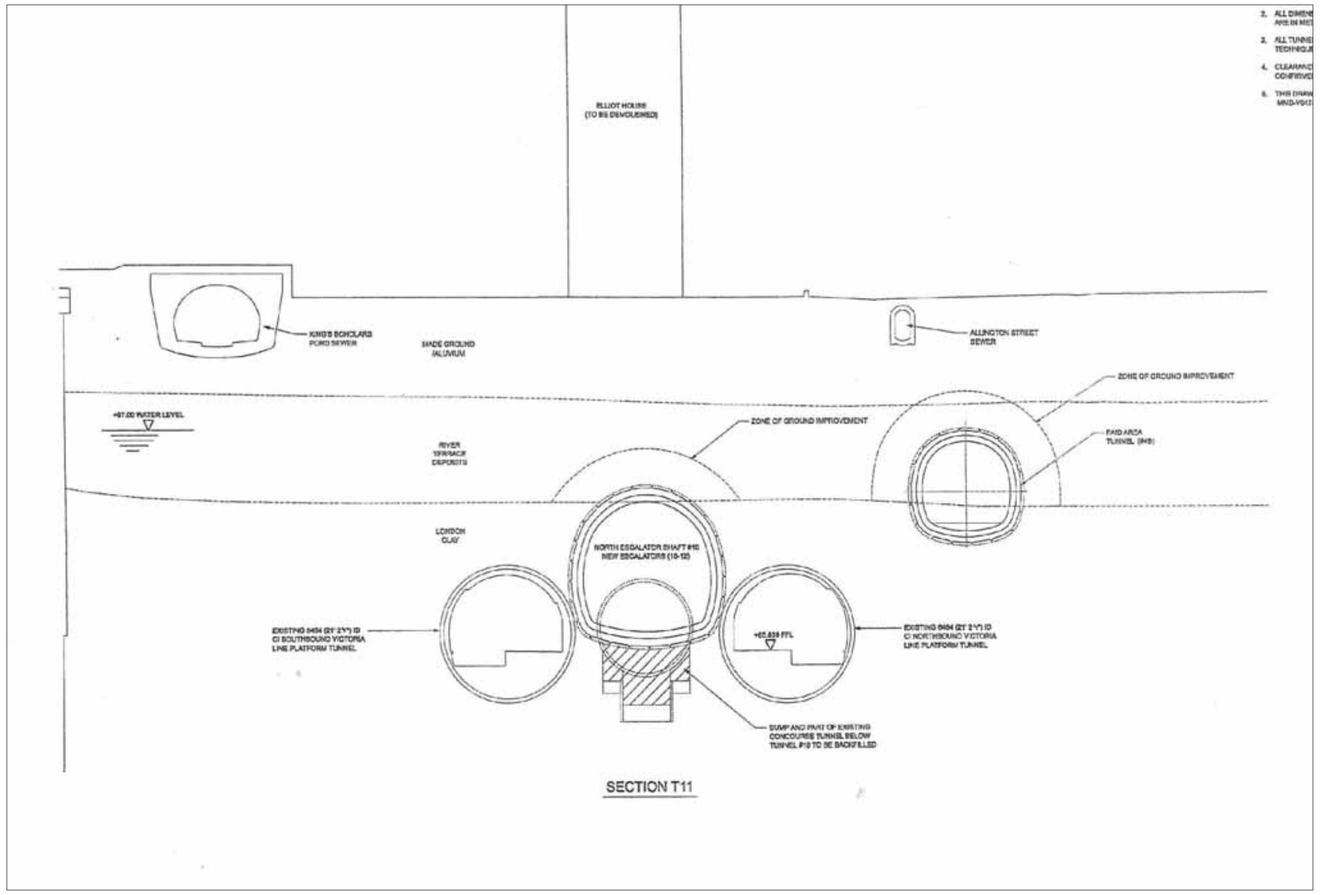
Proposed London Underground (Victoria Station Upgrade) Order

LAND SECURITIES PLC AND OTHERS (Objector No. 3)

REBUTTAL PROOF OF EVIDENCE of TIM CHAPMAN of ARUP

APPENDIX 2

COMPARISON OF JET GROUTED COLUMNS



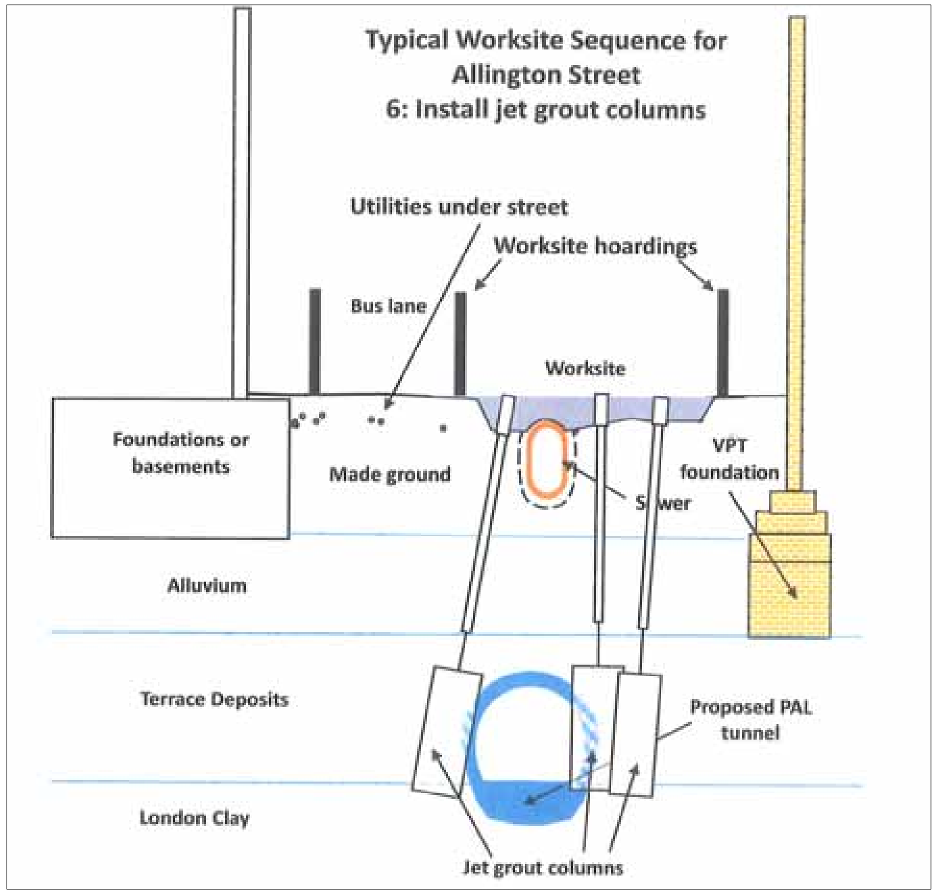
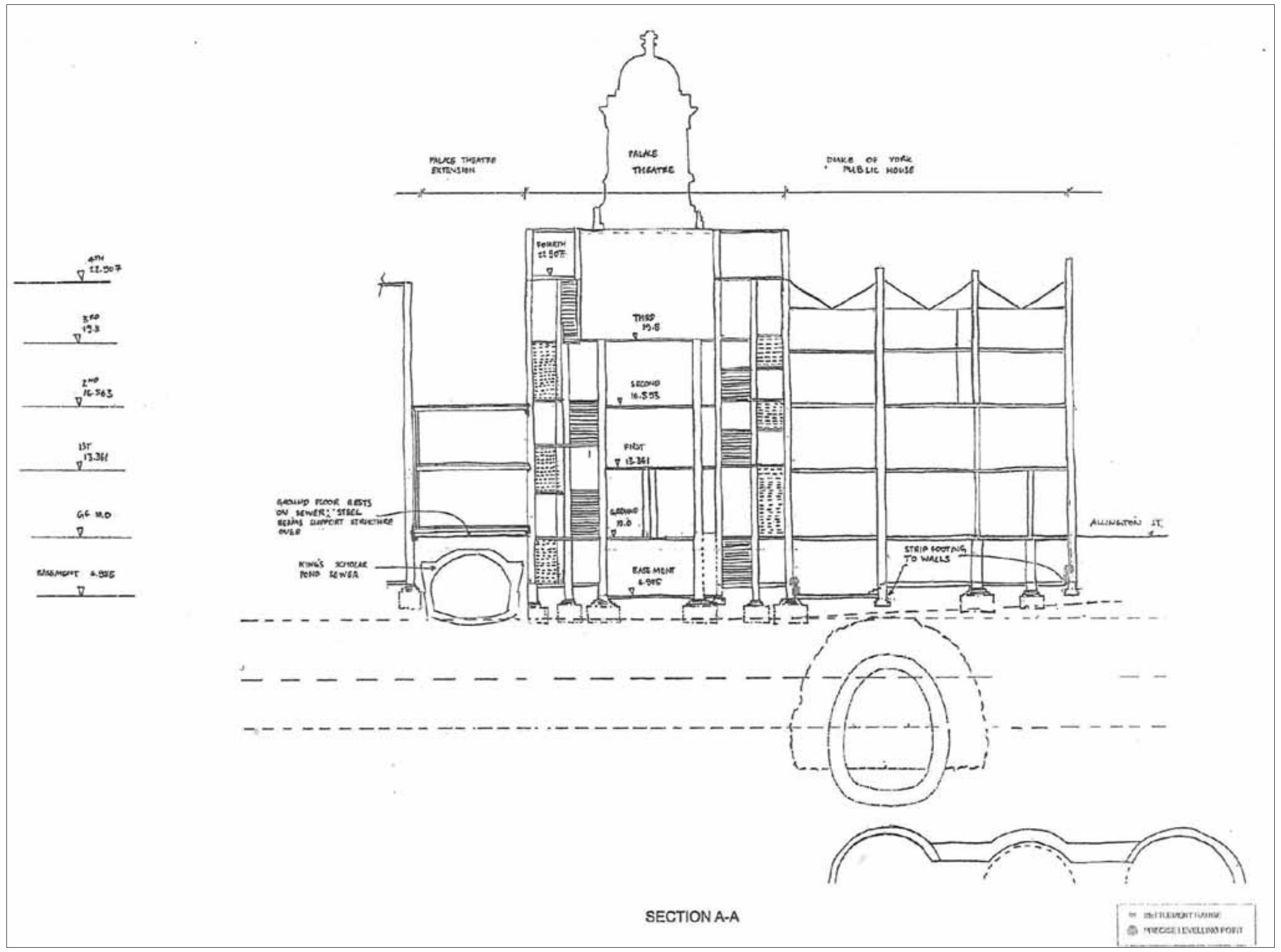
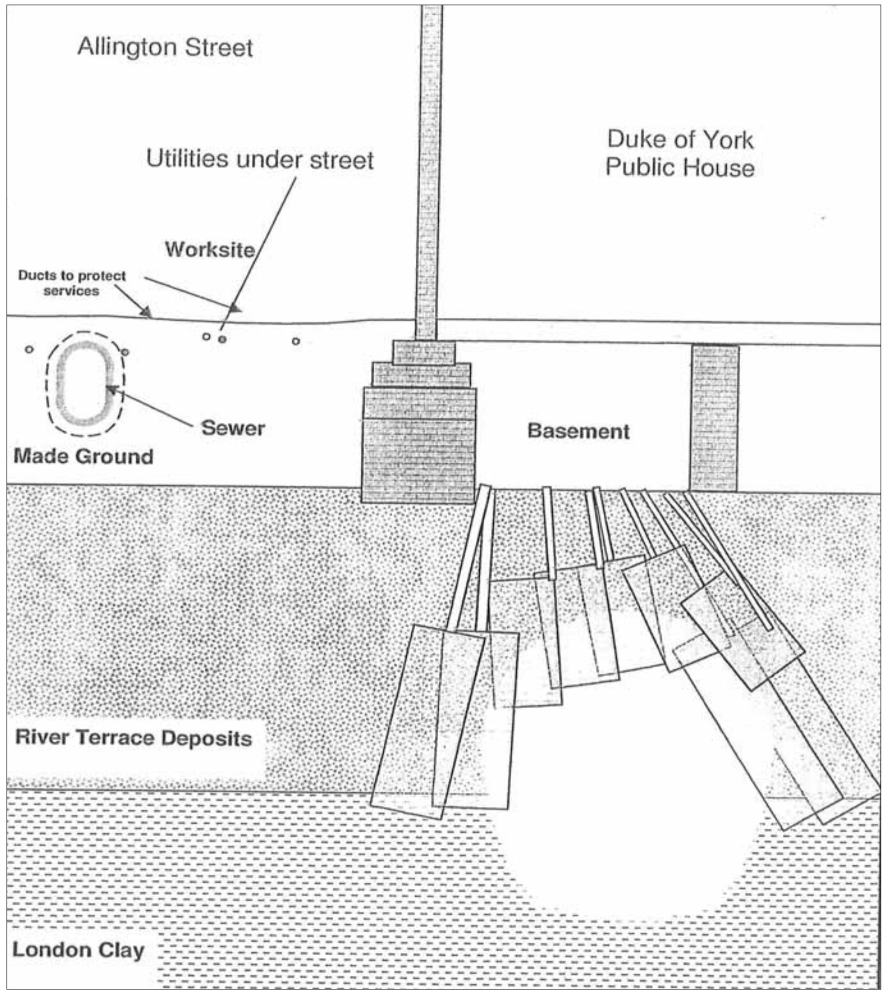


Figure from Appendix 10, Robert Essler's proof of evidence



Mott MacDonald drawings, Phase 3 Potential Damage Assessment of Victoria Palace Theatre



Mott MacDonald figure from Appendix I, Phase 3 Potential Damage Assessment of Victoria Palace Theatre

OBJ3/P3/C2
Rebuttal Proof of Evidence of Tim Chapman

Comparison of Mott MacDonald's drawings and Robert Essler's evidence for jet grout column illustration