The Institution of Structural Engineers

The Structural Engineer

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Crossrail LEARNING LEGACY

STATION DESIGN AND CONSTRUCTION

PLATFORM EDGE SCREENS

OVERHEAD LINE GANTRIES

SPECIAL ISSUE

STRUCTURAL ENGINEERING FOR THE ELIZABETH LINE

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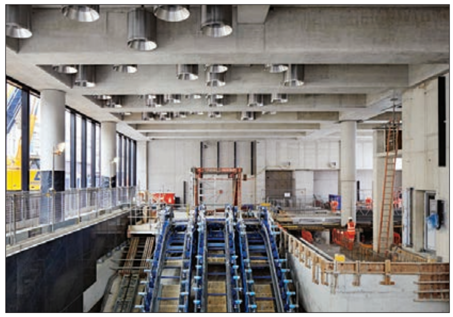

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Upfront

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Front cover: TOTTENHAM COURT ROAD STATION © CROSSRAIL LTD

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PRESIDENT
Faith Wainwright
MBE, FREng, DEng (Hon), FIStructE, FICE

CHIEF EXECUTIVE
Martin Powell

EDITORIAL

HEAD OF PUBLISHING
Lee Baldwin

MANAGING EDITOR
Robin Jones
t: +44 (0) 20 7201 9822
e: robin.jones@istructe.org

EDITORIAL ASSISTANT
Ian Farmer
t: +44 (0) 20 7201 9121
e: ian.farmer@istructe.org

ADVERTISING

DISPLAY SALES
Lucy Filamba
t: +44 (0) 20 7880 7632
e: tse@redactive.co.uk

RECRUITMENT SALES
Beth Fifield
t: +44 (0) 20 7880 6235
e: tsejobs@redactive.co.uk

DESIGN

SENIOR DESIGNER
Nicholas Daley

DESIGNER
Callum Tomsett

PRODUCTION

PRODUCTION EXECUTIVE
Rachel Young

EDITORIAL ADVISORY GROUP

Will Arnold MIStructE
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The Institution of Structural Engineers
International HQ
47–58 Bastwick Street
London EC1V 3PS
United Kingdom
t: +44 (0)20 7235 4535
e: mail@istructe.org

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Editorial

Leaving a learning legacy

Gordon Masterton Guest Editor

There is something special about megaprojects that brings out the best in engineering design and construction. It could be that the longer-term durations for the planning, design and build phases allow more measured and thoughtful decision making; it could be that the weight of public exposure and expectation creates an added incentive to succeed; it could be that the prestige of being part of the programme attracts the best teams in the best organisations; it could be that the extended schedule nurtures a team spirit and a collaborative way of working that is difficult to achieve in a typical shorter-term project.

I was privileged to be part of the Crossrail programme for four years and I witnessed all of those.

There's another important obligation for those involved in megaprojects – to faithfully record and curate the narrative of how the project was delivered. We should expect megaproject teams to pass on an important learning legacy to the engineering profession, in the spirit of one of the fundamental aims of all our great engineering institutions. Other professionals not directly involved deserve to know how challenges were overcome, how the science and art of engineering was used and advanced, and ultimately society deserves to know how the project performed against the objectives defined at the outset. Megaprojects, with their associated resources, personnel and prestige, give us a greater opportunity to learn from experience. Megaprojects should be the living laboratories for defining and prudently advancing our state-of-the-art in the management of risk, including financial risk – the greatest of all skills required by engineers.

This special issue is part of that learning legacy.

The Structural Engineer has had the foresight to recognise the important role of structural engineers in the success of the Elizabeth line (as the service will be known) and this has attracted papers with some of the best examples of innovation, imagination and structural design creativity deployed on the programme.

I have tried to select those that demonstrate a range of structural engineering applications. The stations were obvious fertile ground, and five that feature both below- and above-ground interchanges are included; but the challenges of designing elements such as the platform edge screens, ventilation towers, overhead line gantries and oversite developments demonstrate the breadth of structural design and construction creativity required within complex infrastructure systems.

I'm particularly pleased that there are 21 engineering authors who have invested their time in that worthy aim of creating a knowledge legacy. I hope you enjoy reading these as much as I enjoyed selecting them.

Gordon Masterton OBE, FREng, FRSE, FIStructE, FICE is Chair of Future Infrastructure, the University of Edinburgh. From 2009 to 2013, Gordon, then Vice President of Jacobs, was the UK Government's Project Representative for Crossrail.

The Structural Engineer
- provides structural engineers and related professionals worldwide with technical information on practice, design, development, education and training associated with the profession of structural engineering, and offers a forum for discussion on these matters
- promotes the learned society role of the Institution by publishing peer-reviewed content which advances the science and art of structural engineering
- provides members and non-members worldwide with Institution and industry related news
- provides a medium for relevant advertising

The Institution
- has over 27 000 members in over 100 countries around the world
- is the only qualifying body in the world concerned solely with the theory and practice of structural engineering
- through its Chartered members is an internationally recognised source of expertise and information concerning all issues that involve structural engineering and public safety within the built environment
- supports and protects the profession of structural engineering by upholding professional standards and to act as an international voice on behalf of structural engineers

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People and Papers Awards 2018

Congratulations to all the winners of the Institution's People and Papers Awards 2018, which were presented at a luncheon held at Church House in Westminster, London on Thursday 7 June.

The Awards recognise outstanding contributions to structural engineering and to Institution life, through innovative educational initiatives, papers published in *The Structural Engineer* and *Structures*, lectures, regional group activity, and much more. They cover the whole range of Institution life, from outstanding young professionals to those recognised for a lifetime of achievement and service.

Deborah Lazarus is presented with her Lifetime Achievement Award

Papers Awards

Murray Buxton Award

Medal: Paul Fast and Derek Ratzlaff

'Design of the Grandview Heights Aquatic Centre, Surrey, Canada'

Commendation: Dean Podesta

'Heathrow Airport Terminal 2B, Phase 2 – a new pier facility constructed in an operational airside environment'

Guthrie Brown Award

Medal: Jacob Borchers

'Out of the rubble: how can engineers overcome the problems between design and implementation of seismic-resistant schools in rural Nepal'

Derrington Construction Award

Medal: Sulo Shanmuganathan and Pan Ruodong

'Rejuvenation of the Makatote rail viaduct – a historic steel structure in New Zealand'

Husband Prize

Paul Jackson and Stuart Moore

'Strengthening of the Hammersmith Flyover, London (Phase 2) – design of the prestress'

Clancy Prize

Joe White and Hamish McKenzie

'Seismic strengthening of the Majestic Centre, a 30-storey office tower in Wellington, New Zealand'

Jacob Borchers is presented with the Guthrie Brown Award

Daniel Dowek (centre), winner of the Young Structural Engineering Professional Award, alongside Zacc Richards and Vivi Kefala

Sulo Shanmuganathan receives her Derrington Construction Medal

ALL IMAGES: JIM WINSLETT

Oscar Faber Award
Medal: Bill Harvey
'Elevarch – the world's first masonry bridge arch lift'

Sir Arnold Waters Medal
Medal: Paul Fast
'Timber engineering: Innovations in structural timber design and construction'

Structures Prizes

Best Research Paper Prize
Andrew Liew, Leroy Gardner and Philippe Block
'Moment-Curvature-Thrust Relationships for Beam-Columns'

Best Research into Practice Paper Prize
Chris Burgoyne and Owen Mitchell
'Prestressing in Coventry Cathedral'

Education Award

Excellence in Structural Engineering Education Award
Winner: Alastair McDonald
'First-year design – the design of experimental structures'

Commendation: Gareth Whittleston, Jonathan Haynes and Alan Mardan
'Three applications of augmented reality technologies in structural engineering education'

Commendation: Christian Málaga-Chuquitaype and colleagues
'Emerging technologies for a dynamic learning (... while learning dynamics)'

Awards for Young Engineers

Kenneth Severn Award
Winner: Radu Trancau
Commendation: Michael Minehan

Educational Trust Pai Lin Li Travel Award
Kavinda Isuru Nanayakkara

Young Structural Engineering Professional Award
Winner: Daniel Dowek
Runner-up: Vivi Kefala
Commendation: Zacc Richards

People Awards

Service Award
Nicholas Charles Buxton
Jeffrey Fisher
Haydar Abdul-Muhsin Ibrahim
John Joseph Owen Muddiman
Eleana Savvidi
John Anthony Veares

Lewis Kent Award
Edmund Dwight Booth
Natasha Emma Chandler
David Cormie
Sarah Caroline Kaethner

Keith Eaton Award
Siu Shu Eddie Lam

Lifetime Achievement Award
Deborah Susan Lazarus

Joe White collects the Clancy Prize

Edmund Booth receives his Lewis Kent Award

David Cormie proudly displays his Lewis Kent Award

Susan Giahi Broadbent received a Service Award

Eleana Savvidi collects her Service Award

John Veares receives his Service Award

OBITUARY

Professor Michael Dickson

CBE, BA, MS, CEng, FREng, FIStructE, FICE, Hon FRIBA, FRSA

Michael Dickson, Past President of the Institution, passed away on Monday 28 May 2018 after a battle with cancer. An inspirational man in so many ways, he was one of the great leaders, thinkers and mentors in our profession.

Michael studied Mechanical Sciences at the University of Cambridge, and Structural Engineering at Cornell University, USA. His time at Cornell introduced him to Peter Rice, who lured him back to Ove Arup and Partners, where he worked with, among others, Sir Ted Happold. In 1976, Ted was joined by Michael and six others in setting up BuroHappold Engineering in Bath.

Michael rose to be Chairman of BuroHappold from 1996 to 2005. From its modest beginnings, today the firm has 14 offices in seven countries and employs over 1400 staff. This expansion included a five-fold increase in size during Michael's chairmanship, a graphic illustration of his leadership abilities. However, it was Michael's exceptional design ability which so many people across the world still see daily in his artefacts. This ability was based on an holistic outlook of producing a beautiful built environment. Nowhere is this illustrated better than in the many wide-span enclosures which he designed with Frei Otto. Michael would travel to Stuttgart to see Frei in the early days of BuroHappold, and such interaction, which included Ian Liddell, Ted Happold and Mike Barnes, led to splendid designs, including the Mannheim and Munich Aviary enclosures.

To provide a flavour of Michael's subsequent leadership of BuroHappold in terms of internationally recognised design prowess, examples of projects in which Michael had influence include the Emirates Stadium, Ascot Racecourse, the Millennium Dome, the Weald and Downland Museum Gridshell, The Savill Building in Windsor Great Park and the Great Court Roof in the British Museum.

Michael's leadership in the field of engineering did not stop at BuroHappold. Michael was Chair of the Construction Industry Council, Chair of the New Construction Research and Innovation Strategy Panel and, of course, President of The Institution of Structural Engineers. He chaired the Institution's Task Group on 'Building for a sustainable future – construction without depletion' many years before our politicians heeded the call for sustainability to be a mainstream part of life. Given that construction is the UK's single largest industry, worth 10% (£100bn) of GDP, the introduction of this philosophy into design consultancy across the UK was a crucial step.

But it was Michael's desire to see his experience of structural and architectural engineering passed on to the next generation which will endure as one of his greatest legacies. He was a tremendous role model for the younger generation – full of life, enthusiasm, optimism, creativity and can-do attitude. By placing sustainability at the heart of engineering decisions, he was able to excite the next generation of Michael Dicksons through tutoring them in the technical and artistic attributes of building design.

Michael's involvement with the Department of Architecture and Civil Engineering at the University of Bath stretched back to 1976, and he was still regularly tutoring students on interdisciplinary design projects right up until 2014, as Visiting Professor and as the holder of an Honorary Doctorate in Engineering from the University. Michael also encouraged his colleagues to tutor students, a culture which continues to thrive, to the exceptional benefit of the students and, equally importantly, to the tutors too.

Another example of Michael's commitment to education was his strong involvement in The Happold Foundation, which has as its altruistic aim to inspire younger engineers to dream a bit, and thereby to help to change the world for the better.

One of the courses which Michael lectured was 'Architectural Structures', which took students through the extraordinary history of some of the world's greatest structures. Michael and Professor Pete Walker had been co-authoring a book entitled *Structural Materials for Sustainable Architecture*, which collates this depth of knowledge to the benefit of all. It is nearly complete, and Pete is determined that this book will be published as a tremendous legacy of, and fitting tribute to, Michael.

Michael leaves behind his wife, Effie, and his two daughters, Amy and Sarah, to whom the Institution extends its most sincere condolences. He will be greatly missed by all who knew him.

Michael Dickson CBE
22 September 1944–28 May 2018

Structural engineers recognised in Queen's Birthday Honours

Congratulations to John McRobert MIStructE, who has been made an Officer of the Order of the British Empire (OBE) for services to engineering in Northern Ireland, and to Roma Agrawal MIStructE, made a Member of the Order of the British Empire (MBE) for services to engineering.

John McRobert has worked for the Northern Ireland Roads Service (now the Department for Infrastructure) since its formation in 1973. His early career was in structural design, primarily bridges, then 25 years in highway maintenance and new works before returning to lead the Highway Structures Unit. He has a strong interest in development of new materials and methods to achieve improved results in construction and maintenance.

Roma Agrawal is Associate Director at AECOM. As a structural engineer, she has left an indelible mark on London's landscape, from footbridges and sculptures to skyscrapers including The Shard. She is a tireless promoter of engineering and technical careers to young people, particularly under-represented groups such as women.

UK to apply to stay in European Standards system after Brexit

The UK is to apply to stay in the European Standards system for industry products and services after Brexit, the *Financial Times* reports. According to the newspaper, Business Secretary Greg Clark has written to Scott Steedman, director of the British Standards Institution (BSI), urging him to 'take the steps you feel are necessary' to maintain national influence in the setting of European and international benchmarks.

Clark's letter, dated June 8, gives government approval to a bid to change the statutes of the European Standards organisations (CEN and CENELEC), which specify that members must come from the EU or the European Free Trade Association or are likely to join those blocs, the newspaper reported.

BSI aims to continue to provide UK experts with the standards development framework to support trade in the UK, across Europe and globally. To enable this, the organisation's post-Brexit position is that BSI should remain a full member of CEN and CENELEC.

For more information on BSI's position, see www.bsigroup.com/en-GB/about-bsi/uk-national-standards-body/standards-policy-on-the-uk-leaving-the-eu/Brexit-and-standards-position-statement/

Engineering consultancy wins Queen's Award for Innovation

Bryden Wood has won a Queen's Award for innovation that bridges the gap between construction and manufacturing.

Over two decades, Bryden Wood has been trying to address the need for a fundamental shift within the UK construction industry in terms of both the design and delivery processes. The award is a recognition of its success in this endeavour and cites two key aspects of its work: first, the use of new data-driven tools and techniques in the design process, including Chip Thinking; second, advanced Design for Manufacture and Assembly (DfMA) and a move away from traditional construction towards assembly.

At design, structures are broken down into small, coherent components called 'chips'. Each 'chip' undergoes extensive data analysis and is repeatedly tested in digital simulations for maximum efficiency.

'Chip thinking' allows for rapid building design and means a library of building components or 'chips' with all accompanying data is ever-expanding. Bryden Wood makes this information open source and publicly available, including to competitors, as a way of gathering feedback to continually improve design.

The design seeks to eliminate as much site work as possible through the use of off-site fabrication. Composite, modular components are designed to work together and be assembled. Rather than using traditional construction techniques, the asset effectively slots and bolts together.

'This award recognises the brilliant work our team has been doing to tackle low productivity in the sector and drive a more manufacturing-led approach,' says co-founder Mark Bryden. 'Since Martin Wood and I started the company in 1995, we have led the adoption of more advanced construction techniques and the application of DfMA. The work continues today as we strive to attract more creative, digitally-savvy people into construction.'

A quick guide to CROSS

Confidential Reporting on Structural Safety (CROSS) is a unique reporting scheme sponsored by The Institution of Structural Engineers, The Institution of Civil Engineers, and the Health and Safety Executive. Its purpose is to share lessons learned from structural safety issues and help prevent future failures – by providing insight into how safety issues occur and spurring the development of safety improvement measures.

CROSS depends on professionals to submit reports: the more reports submitted, the better CROSS can identify and quantify safety issues across the industry. Anyone involved in the buildings and civil engineering industry is welcome to submit a report.

To find out more, see 'Five things to know about CROSS', a quick guide from Paul McNulty, Senior Engineer at Structural-Safety, available at www.istructe.org/blog/2018/confidential-reporting-on-structural-safety-five

Foreword

Chris Sexton
Technical Director, Crossrail Ltd, London

I am pleased to have been asked to contribute a foreword to this timely publication. As work on the Crossrail programme comes to a conclusion and we pass the baton of largest infrastructure project in Europe to High Speed 2, this is a valuable collection of insights into the design and construction process for the structural elements of the programme. Crossrail has always been committed to sharing lessons learned, embodied most effectively in our Learning Legacy initiative, through which we have published many examples of insight and good practice on a dedicated website.

Crossrail is a programme involving the construction of 10 new stations, 42km of new tunnels including shafts, portals and viaducts, and the repurposing and upgrading of existing infrastructure. It has been an extremely complex scheme to design and construct, with some of the challenges being met for the first time in a generation. It is very positive for industry and the nation that we now have a pipeline of projects that will add value to the UK, and a reinvigorated workforce, skilled and ready, to deliver them.

The papers in this issue represent a good cross-section of the project's endeavours. Contributions are included from our framework design consultants, our contractors, consultants to our oversite development (OSD) partners, and designers working for our key industry partner, Network Rail.

Crossrail's design journey began with the outline feasibility designs created to enable a Bill for the scheme to be submitted to Parliament. Alongside the Bill process, the design was developed by a framework of multidisciplinary consultants, who in most cases were successful in being appointed to become Crossrail's Framework Design Consultants (FDCs) after the Crossrail Act was passed. The civil and structural elements of the scheme were then taken to RIBA Stage F to enable construction contracts to be let with an employer's design. The FDCs were engaged through NEC3 Professional Services Contracts to deliver design packages which were location- or technical discipline-specific and the coordination between them was managed by the Crossrail Chief Engineers Group. We would encourage future projects to integrate design consultants into the project as closely as possible, as this fosters good communication and, ultimately, the best outcome for the scheme.

Collaboration on design is wider even than Crossrail Ltd. Our stations and other structures need to integrate into a dense world city, and to interface with existing railway infrastructure. To this end, we have worked closely with the OSD partners to provide structures that will support high-value developments which work for the railway user and will result in maximised financial contributions back to the scheme. We have also established a function to oversee the On-Network Works which acknowledges Network Rail's expertise and responsibility for its own assets, but ensures delivery to the functional requirements set down by our sponsors, the Department for Transport and Transport for London.

I would like to thank *The Structural Engineer*, and Guest Editor Gordon Masterton, for bringing us this special issue and would encourage those whose interest in the design and construction of the Crossrail programme is piqued by these papers, to seek out more at https://learninglegacy.crossrail.co.uk/

"IT HAS BEEN AN EXTREMELY COMPLEX SCHEME TO DESIGN AND CONSTRUCT, WITH SOME OF THE CHALLENGES BEING MET FOR THE FIRST TIME IN A GENERATION"

Chris Sexton joined Crossrail in 2010 as Technical Director. His accountabilities include engineering, integration, technical information, sustainability, quality and assurance. Chris joined Crossrail from Laing O'Rourke where he was Head of Engineering for the European business. His first career was in the Royal Engineers. His final military role was Chief Engineer of the Army, responsible for the UK's military engineering capability worldwide.

FURTHER READING

Further information about the Crossrail programme is also available in the following dedicated publications:

►**Black M., Dodge C. and Lawrence U. (2015) *Crossrail Project: Infrastructure design and construction* (Vol. 1), London: ICE Publishing**

►**Black M., Dodge C. and Yu J. (eds.) (2015) *Crossrail Project: Infrastructure design and construction* (Vol. 2), London: ICE Publishing**

►**Black M. (ed.) (2016) *Crossrail Project: Infrastructure design and construction* (Vol. 3), London: ICE Publishing**

►**Vaughan Williams R. and Black M. (eds.) (2017) *Crossrail Project: Infrastructure design and construction* (Vol. 4), London: ICE Publishing**

Proceedings of the Institution of Civil Engineers – Civil Engineering

►**Volume 170, Issue 5: Themed issue on Crossrail Project: designing and constructing the Elizabeth line (May 2017, pp. 2–64).**

►**Volume 170, Issue 6: Themed issue on Crossrail Project: programme managing the Elizabeth line (November 2017, pp. 2–63).**

Damage assessment and monitoring for buildings on the Elizabeth line

Deborah Lazarus
MA (Cantab), CEng, FIStructE, FICE, FRSA
Consultant, Arup, London, UK

Hyuk-Il Jung
BSc, MEng, PE(Korea)
Associate, Arup, London, UK

NOTATION

BRE	Building Research Establishment
SCL	sprayed concrete lining
TAM	*tube à manchette*
TBM	tunnel boring machine

Synopsis
The Elizabeth line, due to open in December 2018, crosses London from west to east. The Crossrail project to construct the Elizabeth line has seen 21km of twin-bored tunnels constructed under central London, with eight new stations built on this section.

The damage assessment and monitoring carried out comprised a significant element of work in terms of the resources involved, both human and financial. The background to this work was the experience from a number of tunnelling projects in London, probably most significantly that from the London Underground Jubilee line extension. While all assets along the alignment were subject to the same process, the impact of the works around the stations and shafts was calculated to be greater than along the bored tunnels, and the extent of instrumentation and monitoring was correspondingly higher. Both automated with instrumentation installed and readily visible on many buildings in these areas throughout the duration of the works.

This paper looks at the damage assessment and monitoring of buildings around the stations, focusing in particular on the new station at Tottenham Court Road. It also provides an overview of the two very different tunnel construction methods used on the project – the so-called tunnel boring machine (TBM) and sprayed concrete lining (SCL) methods – and describes how these lead to the ground movement that is the principal source of potential damage to the buildings.

Finally, the paper considers briefly some of the lessons learned and how these might be applied to future urban tunnelling projects.

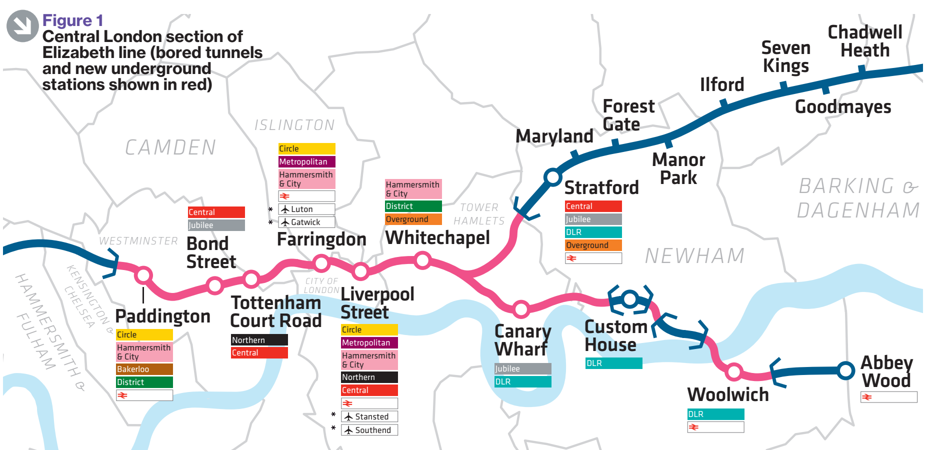


Figure 1 Central London section of Elizabeth line (bored tunnels and new underground stations shown in red)

Introduction
The Elizabeth line, due to open in December 2018, crosses London from west to east. The project has seen 21km of twin-bored tunnels constructed under central London, with eight new stations built on this section of the line (Figure 1). It should be noted at the outset that this was a major achievement: tunnelling below crowded streets was completed with little indication above ground, other than instrumentation on buildings, of what was happening below. Some aspects of how this was achieved are described below.

The construction of tunnels, even using the most modern machinery and control methods, still results in some volume loss and corresponding ground movement. The impact of this movement on the assets, both above and below ground, can be assessed to varying levels of accuracy and the likely degree of damage predicted.

Damage assessment followed the process set out in Crossrail Information Paper D12[1] and relevant Crossrail Civil Engineering Design Standards, which in turn had been developed from earlier work on the London Underground Jubilee line and High Speed 1. This process covered assets including buildings (both low-rise masonry structures on shallow foundations and taller, framed structures on piled foundations), other structures and statutory services. Considering buildings alone, there were approx. 4000 buildings along the route, of which around 300 were listed.

CROSSRAIL

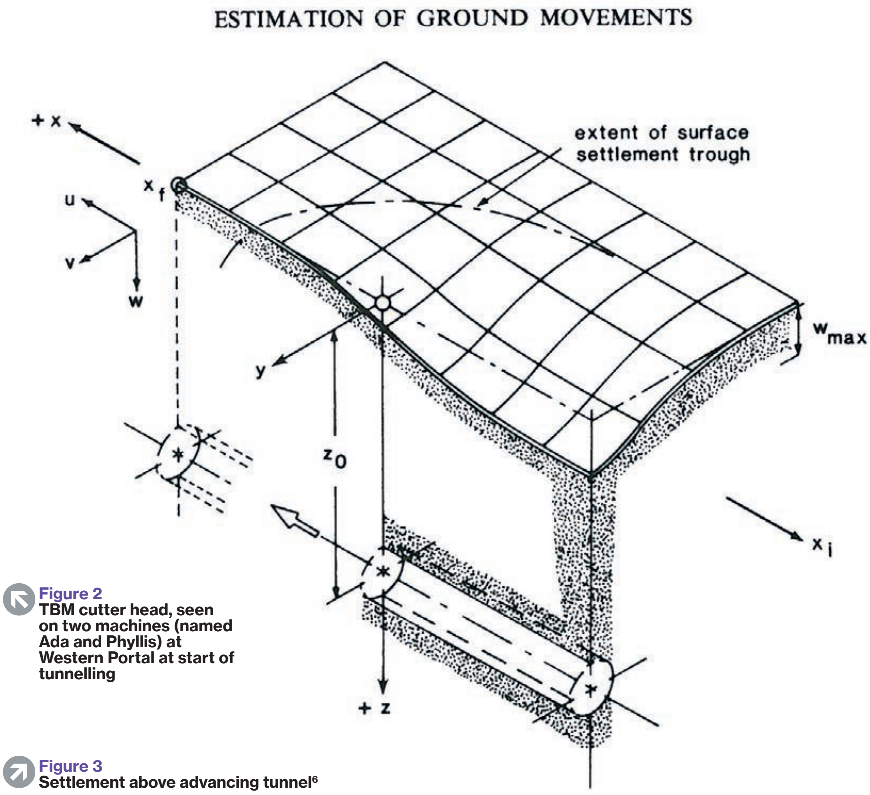


Figure 2
TBM cutter head, seen on two machines (named Ada and Phyllis) at Western Portal at start of tunnelling

Figure 3
Settlement above advancing tunnel[6]

FROM: SOIL MOVEMENTS INDUCED BY TUNNELLING AND THEIR EFFECTS ON PIPELINES, P.B. ATTEWELL, J. YEATES, A.R. SELBY, COPYRIGHT (1986) BLACKIE, REPRODUCED BY PERMISSION OF TAYLOR & FRANCIS BOOKS UK

This paper looks at the damage assessment and monitoring of buildings around the stations, focusing in particular on the new station at Tottenham Court Road; these included a number which were listed, some over 300 years old. Inevitably, for a project of this size and complexity, there are areas which have had to be omitted or covered only briefly. Further information may be found in other publications, notably *Crossrail Project: Infrastructure Design and Construction*[2–5] which comprises four volumes of papers.

The paper also provides an overview of the two very different tunnel construction methods used on the project and describes how these lead to the ground movement that is the principal source of potential damage to buildings.

The paper then explains the three-phase damage assessment method used on the project, including the approach used for heritage assessment and protection. The approach to mitigation of the impacts of ground movement is described, including the extensive monitoring of both the ground and assets along the alignment. The process of compensation grouting, which was used widely as mitigation around the stations, is explained. Examples of buildings around the new Tottenham Court Road station are then used to describe in more detail some of the mitigation measures adopted during construction, the monitoring installation and monitoring results.

Finally, the paper considers briefly some of the lessons learned and how these might be applied to future urban tunnelling projects.

Tunnel construction methods

Two different construction methods were used for the construction of Elizabeth line tunnels: tunnel boring machine (TBM) and sprayed concrete lining (SCL).

There are two different types of TBM: the earth pressure-balanced TBM and slurry TBM. The selection of the appropriate TBM is dependent on the ground conditions. In the London Clay along most of the western part of the route, the earth pressure-balanced TBM was used; and for the tunnels driving through chalk, a slurry TBM was used. More detailed information can be found in specialist publications.

The TBM is equipped with a rotating cutter head (Figure 2) at the front of the machine's steel shield body. The machine is designed to apply face pressure to the excavated ground face so as to balance earth and groundwater pressure until the (permanent) tunnel lining is constructed. Precast concrete segment rings are assembled at the back of the TBM to support the ground, and the TBM pushes against the ready-built ring to move forward.

SCL excavation (ground mining) is completely different and is carried out with the use of excavators. The tunnel section is excavated for a short length (in London Clay, typically 1m), and shortly after the excavation, sprayed concrete is applied to the exposed ground to provide ground support. The process is then repeated, with successive excavation and sprayed concrete application cycles. When the tunnel section is large and full-face excavation is considered unstable, it can be excavated by dividing it into several smaller sections to limit the size of unsupported ground.

On the Crossrail project, TBMs were used for the construction of the running tunnels (internal diameter 6.2m) between the stations, and SCL was used for the construction of station tunnels such as platforms, concourses (internal diameter approx. 9m) and cross-passages (internal diameter approx. 6m). The running tunnels are broadly 20–35m below ground level between the stations. At some of the stations, due to the various constraints in the construction programme, the TBM drove through the platform tunnels before SCL excavation started. In these locations, the bored tunnels were then subsequently enlarged by SCL to the final profile.

When a tunnel is excavated, the ground loses force equilibrium around the tunnel and thus the ground deforms. The face pressure (in the case of TBM) and tunnel lining (segment lining or SCL) provide support to the ground, which can limit the ground movement, but in soft ground such as London Clay (as opposed to rock), it is not possible to construct tunnels with zero ground movement. This is due to the fact that deformation of the ground moves ahead of the excavation face (Figure 3)[6], and also that there is always a time gap between the excavation and the construction of the lining, resulting in further ground movement.

Global best practice widely accepts that tunnelling-induced ground movements in soft ground can be estimated by assuming the settlement trough fits the Gaussian probability curve (perpendicular to the tunnel drive) and the cumulative probability curve (parallel to the tunnel drive). The buildings are assumed to deform following the predicted ground settlement trough (known as the 'greenfield' settlement profile).

Movements along the tunnel alignment were generally predicted to be small, with correspondingly minor impacts on buildings. This correlated with the results recorded during the works; volume losses (Figure 4), particularly on the western drive (between Paddington and Farringdon) through London Clay, were generally lower (<0.5%) than the fairly conservative value of 1% assumed in the damage assessment calculations. Around the stations, the predicted values were higher; horizontal movements around the deep excavations were greater than those around the bored tunnels, and the larger platform tunnels and cross-passages were constructed using SCL techniques which also produce larger movements. Volume losses for SCL works were assumed to be 1.5% for the purposes of assessment. Further explanation of volume loss can be found in Burland[7].

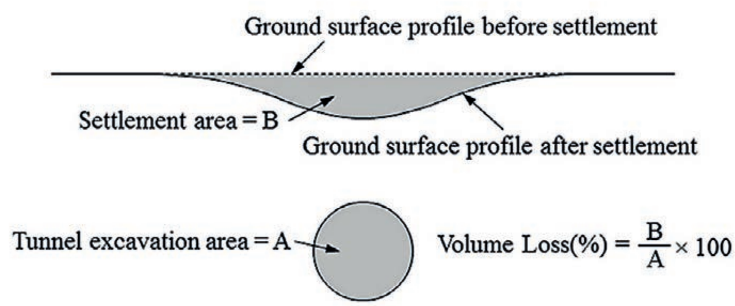


Figure 4 Definition of volume loss

Classification and assessment of building damage

Building damage classification

While the focus of this paper is 'damage', it is worth considering what is meant by this term. Damage is a highly subjective and often emotive subject; in relation to buildings, this is perhaps the case particularly where the perception is that the damage has been caused by the actions of others. It may relate to aesthetics, to function and serviceability – the serviceability limit state – or in more extreme cases of structural damage (with a possible risk of instability) – the ultimate limit state. Most buildings experience some degree of cracking at some stage, often in finishes, but might not be regarded as 'damaged'.

On the Crossrail project, it was recognised at the outset that the works would result in some degree of ground movement and that some damage was predictable – often seen as cracking, but with the potential for other consequences such as jamming of doors or windows. The classification of damage followed the procedure set out in Burland *et al.*[8] and Mair *et al.*[9]. Engineers will be familiar with the so-called 'Burland' classification described in BRE Digest 251[10]. For listed assets, an additional score was assigned to account for building sensitivity. Tables 1 and 2, reproduced from Crossrail Information Paper D12[1], show the values that were used to provide an overall risk level.

Damage assessment process

For the purposes of this paper, the process described is necessarily simplified to some degree, but it is intended that sufficient information is provided, together with appropriate references for further detail where required.

For all assets that were located within a zone such that they might be affected by the works, given in Crossrail Civil Engineering Design Standards Part 8[11] as those located within the 1mm settlement contour, a three-phase damage assessment process was set out as summarised below.

The standard methodology for building damage assessment adopted on the Crossrail project refers to a number of research papers and the methodology used on projects such as the Jubilee line extension, which assumed that buildings behaved as elastic beams and moved as per greenfield ground movements. Full references are included in Crossrail Information Paper D12[1]. The classification is considered to be conservative for many of the buildings, as it is based on case studies of loadbearing masonry buildings on shallow foundations. Framed buildings are considered to be more robust, but this is not quantified within the methodology. For buildings on piled foundations, an alternative methodology is adopted which considers settlements calculated at three different levels along the length of the piles.

Phase 1

Simple criteria (predicted settlement from bored tunnels or from the excavations less than 10mm and predicted ground slope less than 1/500) were used to eliminate buildings subjected to minimal effects. This set the limiting criterion as Damage Category 1

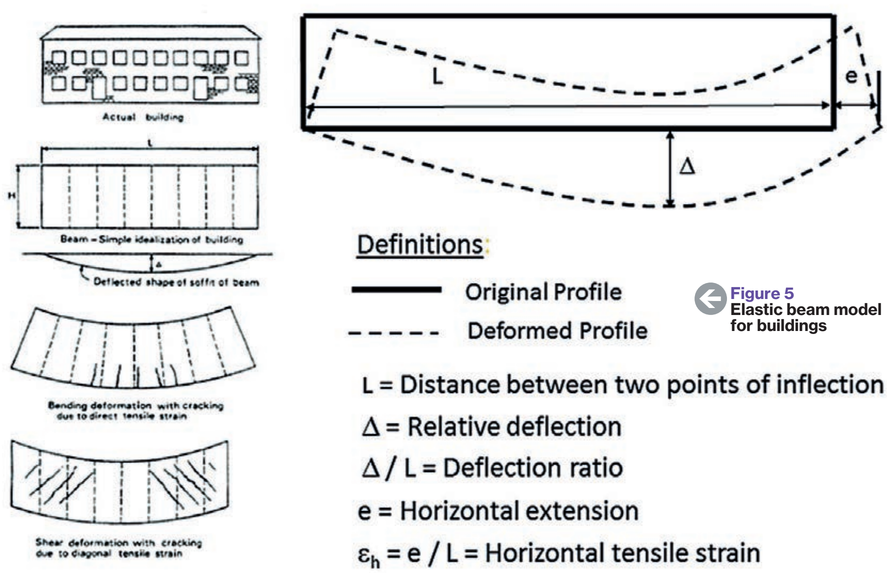


Figure 5 Elastic beam model for buildings

TABLE 1: BUILDING DAMAGE CLASSIFICATION*[1]

Risk category	Max tensile strain (%)	Description of degree of damage	Description of typical damage and likely form of repair for typical masonry buildings	Approx. crack width† (mm)
0	0.05 or less	Negligible	Hairline cracks	
1	>0.05 and ≤0.075	Very slight	Fine cracks easily treated during normal redecorations. Perhaps isolated slight fracture in building. Cracks in exterior brickwork visible upon close inspection	0.1 to 1
2	>0.075 and ≤0.15	Slight	Cracks easily filled. Redecoration probably required. Several slight fractures inside building. Exterior cracks visible; some repointing may be required for weathertightness. Doors and windows may stick slightly	1 to 5
3	>0.15 and ≤0.3	Moderate	Cracks may require cutting out and patching. Recurrent cracks can be masked by suitable linings. Repointing and possibly replacement of a small amount of exterior brickwork may be required. Doors and windows sticking. Utility services may be interrupted. Weathertightness often impaired	5 to 15 or a number of cracks greater than 3
4	>0.3	Severe	Extensive repair involving removal and replacement of sections of walls, especially over doors and windows required. Windows and door frames distorted. Floor slopes noticeably. Walls lean or bulge noticeably, some loss of bearing in beams. Utility services disrupted	15 to 25 but also depends on number of cracks
5		Very severe	Major repair required involving partial or complete reconstruction. Beams lose bearing, walls lean badly and require shoring. Windows broken by distortion. Danger of instability	Usually greater than 25 but depends on number of cracks

* Based on work of Burland *et al.* (1977)[8] and includes typical maximum tensile strains for the various damage categories (column 2) used in Phase 2 settlement analysis
† Crack width is only one aspect of damage and should not be used on its own as a direct measure of damage

('very slight') as defined by Rankin[12], and these buildings were not subject to further assessment. This phase comprised an initial screening using upper bound parameters and assumed greenfield conditions.

Phase 2

In the next phase, a generic assessment was undertaken for buildings within the 10mm settlement contour. The greenfield settlement is imposed on buildings, i.e. it is still, conservatively, assumed that the settlement behaviour is not modified by the stiffness of the building, which is taken to be completely flexible. In addition, the deformation due to horizontal ground movement is considered.

Figure 5 shows the simplified elastic beam model for the simple case where a building (represented as a two-dimensional (2D) element) is located transverse to a tunnel below and entirely within the sagging zone of the settlement trough. In practice, of course, there was great variation in the orientation of buildings in relation to the tunnel alignment and, in some cases, the eastbound and westbound running tunnels were sufficiently close so that the resulting settlement troughs had multiple sagging and hogging profiles due to the interference of the two troughs. A building's response to the settlement is also influenced by the relative location of the building in relation to the sagging or hogging profile of the settlement trough.

TABLE 2: SCORING FOR SENSITIVITY ASSESSMENT OF LISTED BUILDINGS[1]

	Criteria	
Score	**Sensitivity of structure to ground movements and interaction with adjacent buildings**	**Sensitivity to movement of particular features within building**
0	Masonry building with lime mortar not surrounded by other buildings. Uniform facades with no particular large openings	No particular sensitive features
1	Buildings of delicate structural form or buildings sandwiched between modern framed buildings which are much stiffer, perhaps with one or more significant openings	Brittle finishes, e.g. faience or tight-jointed stonework, which are susceptible to small movements and difficult to repair
2	Buildings which, by their structural form, will tend to concentrate all their movements in one location	Finishes which, if damaged, will have significant effect on heritage of building, e.g. cracks through frescos

Using the procedure described by Burland[6] and Mair *et al.*[9], the risk category for each building was assessed as defined in Table 1. For those where the category was assessed as *less* than 3, i.e. 'negligible', 'slight' or 'very slight', the assessment process was taken no further other than for:

- buildings with a foundation level >4m, or >20% of the depth to the tunnel axis for those affected by the bored tunnels
- buildings on shallow foundations and within a distance from an excavation equal to the greater of the excavated depth of superficial deposits or 50% of the total excavation depth
- listed buildings
- buildings where it was considered that further assessment was needed to determine whether protective works were required and/or what these should be.

Phase 3

In the next phase, buildings were considered individually rather than as part of an area analysed generically.

Figure 6
Compensation grouting shaft with TAM array installed[14]

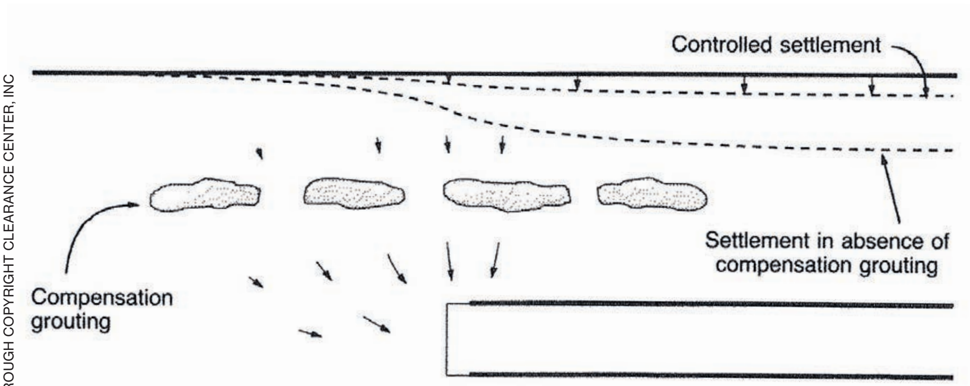


Figure 7
Principles of compensation grouting[15]

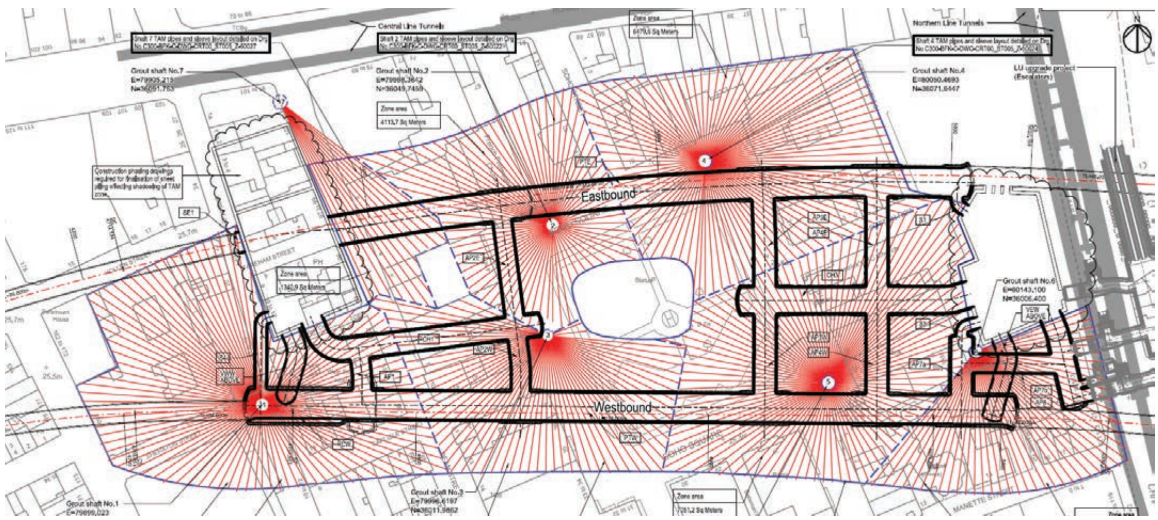


Figure 8
TAM array in Tottenham Court Road station; radially located red lines indicate TAMs from compensation grout shafts (indicated as o on plan)

The Phase 3 assessment was taken through several iterations as required, the intention being to understand whether increasing levels of accuracy would credibly reduce the risk of damage to an 'acceptable' level, with a risk category (or, for listed buildings, a total score) below 3. Refinements included numerical modelling of the soil–structure interaction in conjunction with the tunnel excavation, and also more detailed assessment of the actual structure. Visual inspections were undertaken by structural engineers, often in conjunction with built heritage specialists for the listed buildings, to determine the form of the building and its condition. In a few cases, generally associated with buildings with retained facades but also where the visual inspection identified other specific areas of concern, structural investigations were specified.

The findings from these surveys were included in the Damage Assessment Report produced for each individual building. Perhaps unsurprisingly, defects were identified in some buildings which were felt to need remedial works irrespective of the predicted impact of the Crossrail works; as a matter of professional practice, these were drawn to the attention of the building owner in a brief report, although in the majority of cases this prompted neither a response nor subsequent action by the building owner. Where it was felt that failure to carry out necessary repairs ahead of the works entailed some level of risk to the structure, these were undertaken by Crossrail.

These surveys were entirely separate from the *defect surveys* discussed below, which were undertaken on properties within the zone of influence of the works.

Listed buildings were assessed to determine their damage category in the same way as non-listed buildings. They were then, however, subject to a more detailed assessment process involving:

- agreement of methodology through consultation with English Heritage (now Historic England) and local authorities
- a desk-based study (of available information taken from archives, etc.)
- examination of damage assessment results for the listed buildings
- site visits (by structural engineers and/or heritage specialists to examine form, context (adjacent buildings), features, alterations where visible, repairs, condition)
- initial assessment (identification of sensitive features, fixtures, structure and their weaknesses)
- scoring of structural sensitivity to potential damage
- scoring of heritage sensitivity to potential damage
- identification of buildings where further assessment, mitigation or other measures might be required.

Approaches to mitigation

General approaches

Given the number of properties within the zone of influence of the works (within the 1mm settlement contour) and the range of construction types, age and use, it was reasonable to anticipate some degree of pre-existing deterioration in at least some of these. Defect surveys as described in Crossrail Information Paper D12[1] were carried out in advance of the works to provide a record of the pre-works condition as a reference for agreeing any changes which could be ascribed subsequently to the works. Around the stations and shafts, where demolition of adjacent buildings was required, the impact of these preliminary works was assessed and defect surveys were undertaken prior to the commencement.

The starting point for mitigation was the commitment by Crossrail to keep *predicted* damage levels below Category 3, consistent with 'moderate'. For listed buildings, the total score was the key parameter: this was the combination of the risk category and the sensitivity score, with this combined impact to be <3. A refinement, however, was introduced during the works which recognised that no mitigation would be required in exceptional cases where there was a very high sensitivity score but negligible damage predicted. Here, it was this combined impact which was to be <3.

The primary and (preferred) means of mitigation was to control movement at source by controls on tunnelling and excavation with contractual limits on volume loss. Monitoring, both automated and manual, of asset and ground movements was widely used, with a range of instrumentation installed route-wide.

Ground treatment was also implemented where appropriate, although this in itself also has some impact, as described below.

In a limited number of cases, repairs and/or protection were indicated prior to the works, generally due to the pre-existing condition or the presence of sensitive elements such as stone 'cantilever' stairs, which in some instances were a cause for concern.

In many instances, the preferred solution, including for listed buildings, was to allow cracking to occur and to allow repair with appropriate materials and methods once ground movements had ceased, as indicated by ongoing monitoring (see below). In the majority of cases, pre-emptive interventions were thought likely to be more intrusive and lead to greater impact on historic fabric, an approach which was agreed with the heritage authorities.

Ground treatment: compensation grouting

Where physical mitigation was indicated, the ambition was to make this non-intrusive wherever possible. Around stations and shafts, *compensation grouting* was adopted as the principal means of mitigation. While this did not control settlements to an absolute (target) value, it reduced the unmitigated movement and, more importantly, was used to limit the deflection ratio (Fig. 5) to a value consistent with damage category 1 (very slight) or less, and a maximum ground slope of 1:1000.

Compensation grouting is described in more detail in other papers (e.g. Bezuijen, 2010)[13], but a brief outline of the technique is included here. A grout shaft is installed at a specified location to enable an array of *tubes à manchette* (TAMs) to be drilled out horizontally to lengths of up to 80m from the shaft. The tubes are installed radially at a number of depths within the shaft (Figure 6). Grout is injected through a selected TAM using two rubber packers which select the part of the tube where the grout will be injected. The grout is injected at high pressure so that the ground is fractured horizontally, then the grout penetrates through the fractured cracks and heaves the ground to compensate for settlements (Figure 7).

Although compensation grouting is intended to mitigate ground movement, the installation of the grout shaft and TAMs themselves results in some ground settlement. While vertical ground movements due to the installation of the shaft are insignificant, the TAM installation process can cause settlement, in some cases of a magnitude of 10–20mm, influenced by both ground conditions and also the installation method. Settlements may also be larger in the zone closer to the shaft where the density of TAMs is much higher (see the red radial lines near the compensation shafts in Figure 8). This initial settlement is later compensated for by injecting grout through the TAMs before the main tunnel construction work starts, although this process, known as 'priming', can itself result in ground heave in excess of that anticipated.

While the grout process might be thought of as a continuous reactive process, the reality is not a smooth re-levelling, but rather a series of small step changes. Where settlements are predicted, the ground may also be lifted *in advance* by a 'jacking' process. The movements can be tightly controlled using a series of hydraulic levelling cells (Figure 9) generally installed in the basement of a building – or more accurately a group of basements nearby.

Further information on implementation of the process is provided in the case studies below.

Mitigation in buildings

The general approach to mitigation has been described above, i.e. minimising physical intervention where possible, in conjunction with monitoring of buildings. This resulted in a relatively low level of building works, complemented by site visits when concerns were raised either by building owners/ occupants or by unexpected trends in monitoring results.

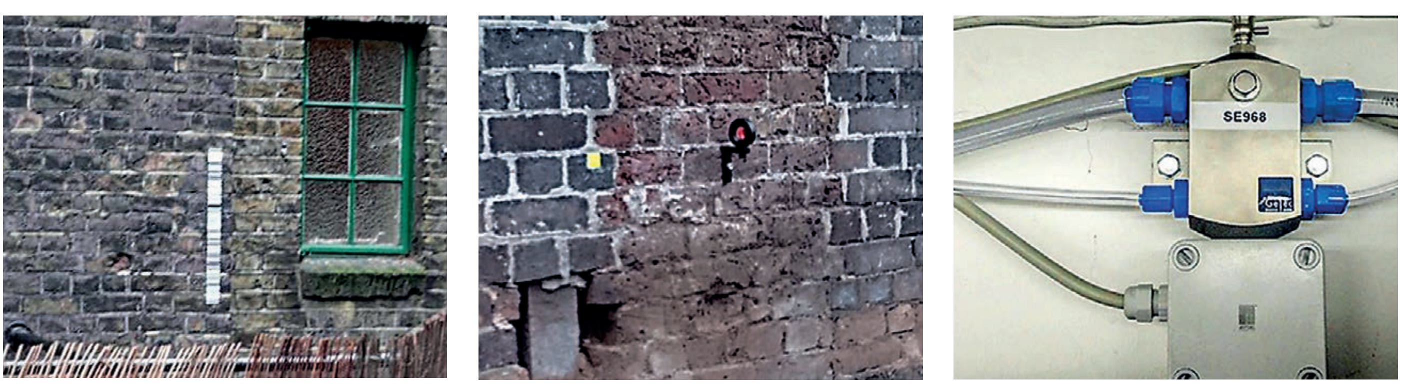
Works included application of film to windows in a small number of buildings

Figure 9
Monitoring techniques

a) Invar scale on building facade – used for precise manual levelling

b) Prism on building facade – part of automated monitoring installation using Automated Total Stations

c) Hydraulic levelling cell – part of automated monitoring installation used to control building slopes



where there were concerns that movements might lead to breakages, and a number of 'protection' schemes for elements such as stone 'cantilever' stairs where it was considered from inspections that there was, albeit low, some risk of collapse. In these cases, a structure was designed to be in position in the event that there was a collapse, but it was installed initially without contact to avoid imposing stresses into the element.

Repairs were carried out prior to the works in some buildings where defects were identified which required rectification more urgently. There were also repairs implemented during construction in some cases, even where it was not clear that the Crossrail works were the contributory factor: the overriding principle was to mitigate risk as far as reasonably possible.

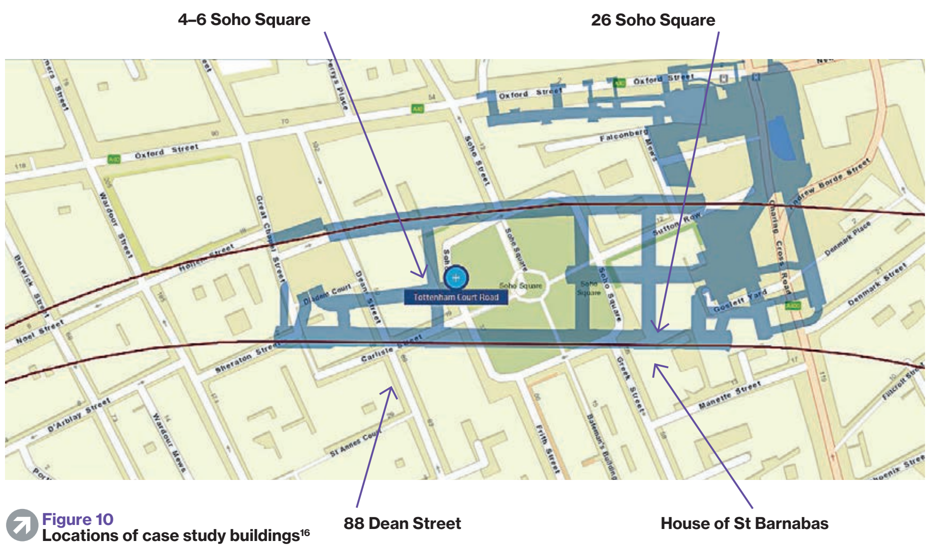


Figure 10
Locations of case study buildings[16]

Monitoring

Monitoring of both buildings and the ground was undertaken extensively across the project as part of the asset protection strategy. This provided information on when and how contingency measures should be adopted.

It confirmed that the ground and the assets were behaving as anticipated. It also both provided information for design verification and allowed construction control, providing confirmation that excavations were being implemented in a controlled manner.

Techniques used included manual monitoring using studs and Invar calibrated scales (Fig. 9a); automated monitoring of prisms (Fig. 9b); hydrostatic levelling cells (Fig. 9c) and tiltmeters; and, at a later stage, satellite technology to look at ground movements in specific areas. This is an area where there are continuing developments and some changes might be anticipated for subsequent projects.

As is common practice, a 'traffic light' system of trigger levels was adopted: **green** (proceed, no issues), **amber** (monitor more frequently, review calculations and start implementing contingency measures if trends continue) and **red** (a value not to be exceeded; in cases where this occurs, measures to be implemented to prevent further movements, with work suspended).

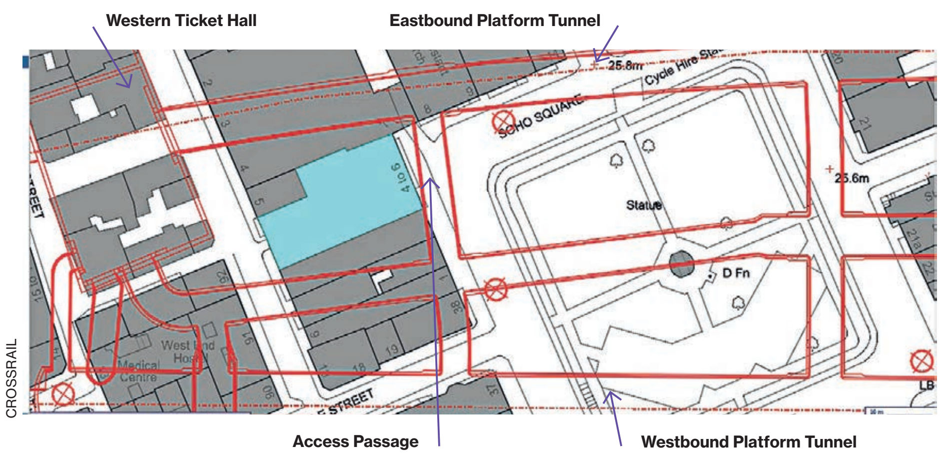
Amber values are close to those calculated from analysis, while **red** levels should be based upon an acceptable 'damage' criteria.

In many cases, as for other movement monitoring, it is trends that are important and any obvious cause which might underlie the increased movement should be investigated.

Case studies

Buildings around the new Tottenham Court Road station which went through the assessment process included some of the oldest on the alignment: around Soho Square and in Denmark Street, in particular, several were over 300 years old. There was also more modern construction, including Centre Point. A few examples of the older buildings are included here, with some further detail of the works carried out. Locations are shown in Figure 10.

Figure 11
Plan location of 4–6 Soho Square showing new station tunnels below



4–6 Soho Square

The building at 4–6 Soho Square is located at the northwest corner of the square (Figure 11). It is one of a number of buildings on 'mixed foundations', i.e. a combination of deep (piled) and shallow foundations. There is no prescribed methodology for buildings on 'mixed' foundations and the damage assessments were carried out on a building-specific basis. Due to its unusual form, the assessment and protection of 4–6 Soho Square are described in some detail.

The original building fronting Soho Square (Figure 12) is linked to 6 Dean Street on the west side. It was originally constructed as a warehouse *c.*1801–04 for John Trotter, 'store-keeper general' for army supplies during the Napoleonic wars. The warehouses were altered in 1816, when Trotter converted them into a 'bazaar'. The shop front and ground-floor level were reconstructed *c.*1890. At that time, there was an open area between the rear of 4–6 Soho Square and the rear of 6 Dean Street. This phase of the building is of loadbearing brickwork with timber floors. It is four storeys at the front on Soho Square and three storeys at the rear on Dean Street; a vaulted brick basement with a concrete slab occupies the entire site.

Figure 12
Front (east) elevation of 4–6 Soho Square

Alterations took place in the mid-1980s in both the basement and at the upper levels. The open area between the buildings fronting Soho Square and Dean Street was infilled to provide two wings of full-height accommodation to the north and south and a double-storey atrium in the centre. The two 'wings' are steel-framed with a mansard roof to the south wing and flat roof to the north wing. A number of archive drawings were obtained, although these did not show full construction details and attempts to locate any further information were not successful.

Figure 13 provides a schematic section through the building looking north, providing an overview of the different foundation systems.

It was recognised at an early stage of the detailed design that the specific arrangement of construction in this building was particularly complex in terms of its likely response to ground movements; a detailed assessment was therefore carried out.

The new floors comprise precast concrete units with an *in situ* topping. The units generally span across the width of the wings onto edge beams which, in turn, are supported on perimeter steel columns. The columns are tied together by transverse steel beams. The edge beams are connected to the party walls on the north and south sides. While the details on the archive drawings indicated that some provision for differential settlement between the buildings was intended, this would have been limited in magnitude and intrusive investigations showed that the tubes in which the threaded studs are located have been concreted up.

The implications of differential movement between 4–6 Soho Square and the adjacent buildings were therefore considered on the assumption that there was little, if any, provision to accommodate such movement.

The foundations of the original buildings are corbelled brick or stone strip and pad footings, although some walls were underpinned during construction of the 1980s link blocks. Foundations to the link blocks are piled, with the steel frame taken through the vaulted substructure. Trial pit information from the refurbishment shows strip footings under the original brick walls/vaults are typically founded at approx. 500mm below basement level, with various local deepenings at the original timber column positions to approx. 1500mm below basement level. The latter approximately matches the founding level of the pad

"THIS BUILDING WAS PARTICULARLY COMPLEX IN TERMS OF ITS LIKELY RESPONSE TO GROUND MOVEMENTS"

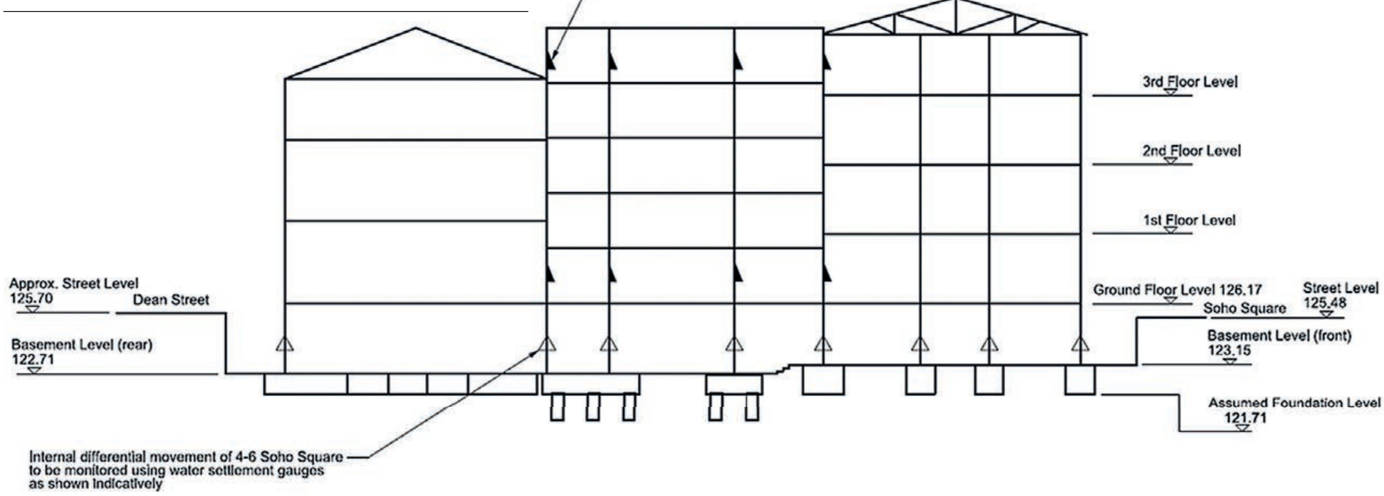


Figure 13
Schematic section through 4–6 Soho Square (east–west, taken through piled wing)

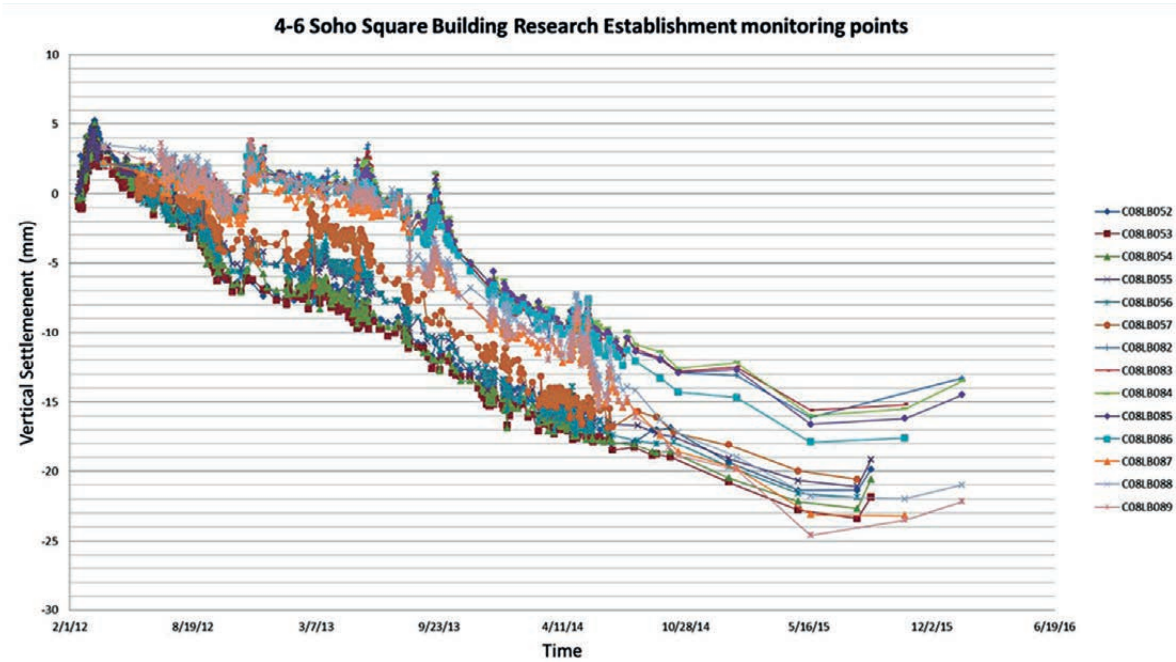


Figure 14 Time–settlement plot for 4–6 Soho Square

Figure 15 Heritage features: plasterwork in House of St Barnabas

footings under internal timber columns. Pile caps and ground beams for the new foundations are set just below the basement slab. Record drawings indicate that the walls adjacent to the new foundations were underpinned to a level approx. 1500mm to 2000mm below basement level, to enable the construction of the pile caps and ground beams. Crossrail archive information shows piles under the link blocks extend 22m below ground-floor level, although there is no record of pile diameters.

While it was accepted that the risk of some minor cracking to 4–6 Soho Square could not be eliminated, neither significant cracking of the walls nor of floor slabs and finishes were acceptable impacts in terms of damage to the structure and to its heritage value. Hence, it was concluded that mitigation measures were required, with any minor defects being repaired once movements had ceased.

The Phase 3 assessment of 4–6 Soho Square assigned a total score of 3 to the building, looking at the most onerous construction stages for critical sections through the building. This resulted in the adoption of compensation grouting as a protective measure to reduce the magnitude of ground movements, generally allowing a reduced damage category of 1 to be assigned to the building. This is consistent with possible crack widths of up to 1mm. The grouting was needed to address both differential movements between the piled and non-piled elements within the building, and also those between the piled elements and the party walls, recognising that the latter are also part of the adjoining buildings and thus subject to separate control. Grout shafts around Soho Square are indicated in Fig. 8, together with the TAM arrays from each. As part of the mitigation, compensation grouting could be used to control the movements of those parts of the structure on ground-bearing foundations, as was adopted commonly for buildings around the stations.

Although grouting was not used to control pile movements, in order to maximise coverage it was proposed, unusually, that the grout TAMs would be 'threaded' through the piled areas, requiring more accurate TAM positioning but offering greater control of ground movements. This also allowed grouting below the piled areas should this be necessary; again, while this was not a measure commonly utilised, it was decided after careful review that this would be instigated if differential settlements between these and the ground-bearing areas exceeded the 'trigger' level of 5mm specified in the Specification for Control of Ground Movements[17]. The same trigger level was specified for differential movements between the piled areas and the adjoining buildings, namely 3 and 7 Soho Square and 5 Dean Street. In the event, these triggers were not reached and no compensation grouting was required in the piled areas.

Settlement of the foundations to the party walls and the areas of slab adjacent was controlled by the grouting arrays below the neighbouring buildings. It was therefore essential to control the grouting process for all these buildings as a single unit and instrumentation was arranged accordingly. It was recognised there was still some risk that, in the basement areas below the piled wings, it might not be possible to mitigate the movements fully by compensation grouting; it was therefore accepted that some cracking could occur.

Compensation grouting was also used to control the differential settlements along the party wall lines to avoid damage to the connections between the party walls and the 1980s framed structures. In the absence of effective allowance in the construction details for differential movement, it was necessary to ensure that there was very specific control

Figure 16 Heritage features: stone staircase in 26 Soho Square (protection structure seen below)

"A PROTECTION STRUCTURE WAS INSTALLED BELOW THE STAIRS"

of the compensation grouting in order to minimise the impact of the predicted ground movements on the various elements of the building and its neighbours.

The following additional measures were undertaken to minimise ground movements at source as far as practicable, using controls on construction:

- A volume-loss control zone, where a lower volume-loss target is set, was introduced for the eastbound running tunnel. During construction, the TBM was driven with tighter face-pressure control when driving through the volume loss control zone.
- Volume loss was minimised during construction of the three adjacent station tunnels that affect settlement of the buildings. This was achieved by sequential excavation of the tunnel sections so limiting the size and the length of unsupported ground at the excavation face.
- Ground movements resulting from construction of the Western Ticket Hall were carefully controlled by minimising the deflection of the embedded wall.

Specific instrumentation was installed on the outside and inside of 4–6 Soho Square and the adjoining buildings to allow monitoring to be undertaken as part of the construction contract. This was for control of the compensation grouting process and movements of the building. This instrumentation comprised hydraulic levelling cells in the basement with prisms, Building Research Establishment (BRE) studs and Invar scales on the external facades. Tiltmeters were also installed on internal columns. Figure 14 shows the time–settlement plot for the BRE studs at ground-floor level. This provides some indication of the monitoring data obtained during the project; such plots enabled the impact of particular construction activities to be assessed.

During the course of the works, no more than hairline cracking (consistent with damage category <1) was identified. Given the works (structural and non-structural) being undertaken in the building concurrently with the Crossrail works, it was not possible to be definitive as to causation. It may be concluded that the mitigation measures adopted, namely compensation grouting and specific construction controls, prevented unacceptable levels of cracking being experienced during the course of the works. Monitoring also suggested that that there was no noticeable difference in response to ground movements between the piled and non-piled areas of the building.

While this was an unusually complex structure, the process described provides an overview of the extent of assessment and monitoring required to comply with Crossrail Ltd's obligations to safeguard the assets along the alignment.

The House of St Barnabas (1 Greek Street)

The House of St Barnabas is a Grade 1 listed building. Internally it has some very fine plasterwork (Figure 15) and it is acknowledged as a fine example of a Georgian interior; Pevsner[18] describes it as 'one of the best and best-preserved mid-C18 houses in London'. Located at the east end of Tottenham Court Road station, it was afforded special protection, with the appointment of separate heritage specialists, extensive pre-works condition surveys and monitoring installations both internally and externally. Pre-works mitigation included some repairs to brickwork and plaster and the installation of a protection frame below the fine open-well cantilever stair staircase.

26 Soho Square

Immediately to the north of the House of St Barnabas, the building at 26 Soho Square is Grade 2* listed. It also contains decorative plasterwork and another fine staircase (Figure 16). A protection structure was installed below the stairs until such time as ground movements were shown from the monitoring to have diminished to the specified level (<2mm per year).

Both here and in the House of St Barnabas, there was clearance between the protection structure and the staircase throughout, and no further works were required.

88 Dean Street

The building at 88 Dean Street, also Grade 2* listed, showed signs of past movement both externally and internally. Of most concern was the pronounced outward lean on the front elevation, visible from the street and confirmed by accurate survey. While investigations confirmed that the facade was restrained by the internal floors, the cause of the movement remained unconfirmed and there was some concern as to potential stability even with the very small ground movements predicted. Accordingly, it was decided that a scaffold structure would be erected externally, separated from the facade but designed to hold it in the event of major movements.

The facade was monitored and periodic inspections were carried out. In addition, some repairs were carried out while the scaffold was in place to improve the integrity of the facade.

Conclusions and lessons learned

The damage assessment process carried out followed that set out in Crossrail Information Paper D12[1], which was established at the start of the works, and was itself derived from other major tunnelling contracts undertaken in the late 20th century. The results may be deduced from the outcome, namely that there were no incidents which required urgent intervention during the course of the works. Post-works repairs were always anticipated, but it was possible to carry these out in a planned manner.

A number of useful lessons have been learned from what was done and it is important that these are considered for future works.

While overall extensive monitoring was carried out along the alignment, for any given building this was, in most cases, not sufficient to understand its behaviour and response to ground movement in detail. In the majority of cases, this is not an issue, but for a specific building where there are particularly delicate and/or valuable finishes (e.g. The House of St Barnabas), or where the overall condition pre-works is a concern, a more tailored approach is likely to be needed.

The damage category does not identify specific crack locations, but rather the likelihood of cracks up to a given width occurring. Likely locations could be predicted, but other cracks were found to occur, particularly where there were previously unidentified defects/weaknesses. Overall, damage levels were low.

There is some potential ambiguity as to whether the predicted crack will be in the finishes and/or in the structure. Originally, the methodology was for masonry buildings on shallow foundations: further review is needed for modern framed buildings on piles. Additionally, assessment of the behaviour of buildings where a facade on shallow foundations has been retained in a new development comprising a framed structure on deep piled foundations should be undertaken.

Further research work is being undertaken to look at the effects of building geometry and the extent of facade openings. The results of academic research should be evaluated for further projects requiring widescale asset assessment. Neither the visual inspections nor the condition surveys provided a detailed record of the building condition. In order to do this, the initial risk assessment has to identify where this should be prioritised. In addition, there are more sophisticated methods of recording condition using photography for subsequent comparison; while this would not be required in all cases, the initial screening should be used to determine where this is best used.

Development of more sophisticated monitoring is continuing. As for assessment methodology, alternative strategies may be used in the future.

The three-phase assessment system provided a good basis for asset protection. With any large infrastructure project, the number of assets implicated will be very significant and improvements in the methodology which may be more effective without increasing the risk should be kept under review.

The extensive use of compensation grouting around the new stations has effectively mitigated the impacts of ground movements. The effects of TAM installation may not always be negligible and selection of the appropriate method needs to be kept under review.

While the assessment process has proved effective over a wide range of structures, with different ages, in varying states of repair and with varied finishes, it does not remove the overriding requirement to use engineering judgement and proceed accordingly at all times.

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Design of Farringdon Elizabeth line station

Ghanshyam Kumar
BEng, MTech, MEng, CEng, MIStructE
TfL Engineering (formerly Structural Associate, AECOM, London, UK)

David Sharples
BSc (Hons), CEng, MICE
Technical Director, AECOM, London, UK

Synopsis
Farringdon is one of eight new underground stations being built in central London for the Elizabeth line and will be one of the key interchange stations on the new line. Upon completion, over 140 trains per hour will pass through the Farringdon interchange, making it one of Britain's busiest stations. With Thameslink, Elizabeth line and London Underground services, it will be a key link in bringing passengers from outer London to the business hubs in the City and Canary Wharf. The station will also provide direct rail links to three of London's five airports.

Farringdon Elizabeth line station comprises two platform tunnels, each 245m long, between new ticket halls over 300m apart. Each ticket hall has been designed to accommodate future oversite developments.

This paper discusses the structural engineering challenges encountered during design and construction of the two ticket halls on constrained sites surrounded by existing transport infrastructure, utilities and historic buildings.

NOTATION

AOD	above ordnance datum
BIM	Building Information Modelling
CAD	computer-aided design
ETH	East Ticket Hall
LU	London Underground
mATD	meters above tunnel datum (AOD +100m)
OSD	oversite development
SCL	sprayed concrete lining
SH-W1	circular shaft (West Ticket Hall)
SH-W2	rectangular shaft (West Ticket Hall)
SH-W3	escalator shaft (West Ticket Hall)
SH-E3	trapezoidal shaft (East Ticket Hall)
TBM	tunnel boring machine
WTH	West Ticket Hall

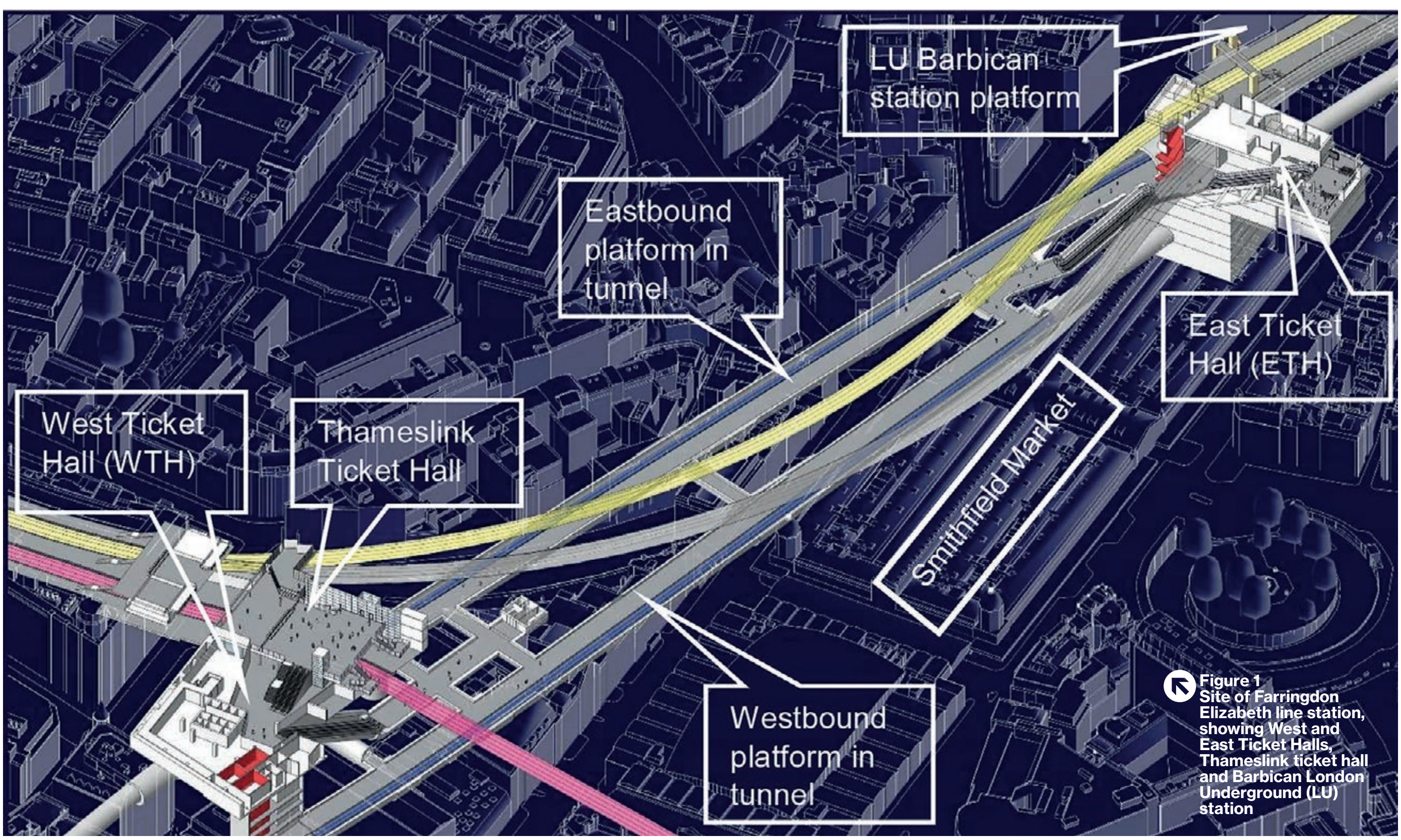

Figure 1 Site of Farringdon Elizabeth line station, showing West and East Ticket Halls, Thameslink ticket hall and Barbican London Underground (LU) station

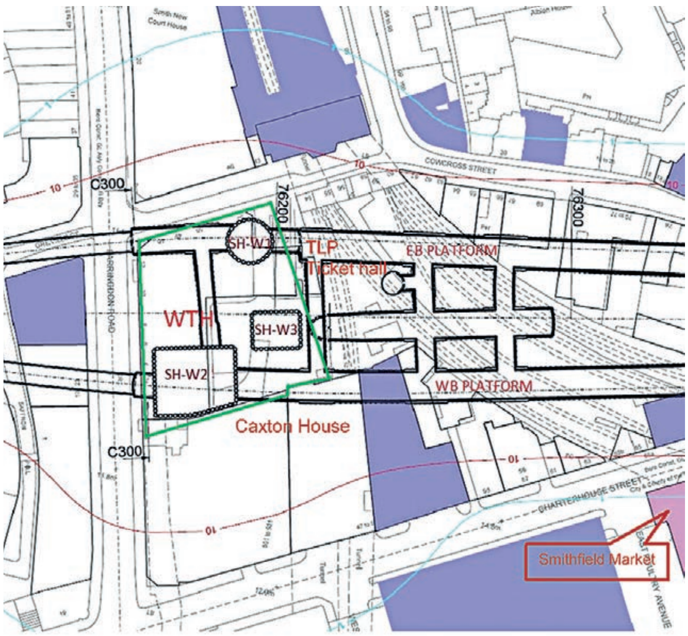

Figure 2 Existing structures and listed buildings around West Ticket Hall (WTH), including eastbound (EB) and westbound (WB) platform tunnels and Thameslink (TLP) ticket hall

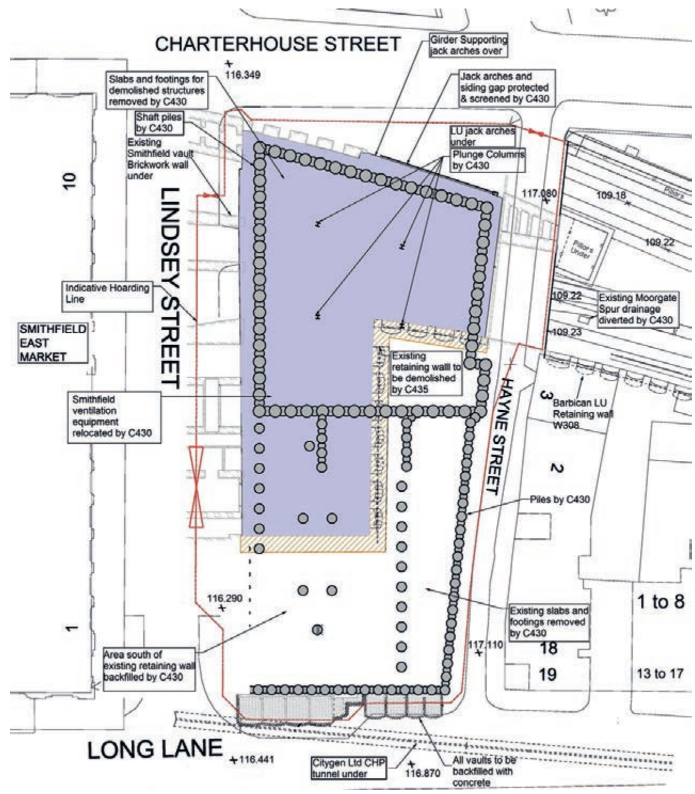

Figure 3 East Ticket Hall site at time of advanced works

Introduction

Farringdon Elizabeth line station is located in an area of London with a rich history. Former buildings on the site were demolished to allow construction of the ticket halls, shafts, etc. to take place. Both the East Ticket Hall and West Ticket Hall sites are surrounded by roads, railways and structures, with the original ground level typically 4–6m below the existing road level. These sites have a long history of railway-related activities, including railway maintenance workshops, goods yard operations, refuelling and storage of trains. The two sites are roughly rectangular and their overall dimensions are approx. 70m × 32m and 65m × 48m respectively.

The new station stretches the length of the neighbouring Grade II* listed Smithfield Market: from its West Ticket Hall immediately to the west of, and integrated with, the recently constructed Farringdon Thameslink ticket hall; to its East Ticket Hall adjacent to Barbican London Underground station (Figure 1).

During construction, Farringdon was also the termination point for four tunnel drives using four tunnel boring machines (TBMs): two commenced their journey from Limmo (near Canning Town station) in the east and two from Royal Oak just outside Paddington in the west. The design and construction phasing of the station, therefore, had to make provision to accommodate their arrival, decommissioning and removal/disposal.

Designing and constructing a new underground station in this highly constrained location presented significant challenges and opportunities. The final design has been driven by efficiency, physical constraints, buildability and construction phasing, as well as the need to provide a functional station appropriate to its location and historical setting. This paper presents an overview of the design and considers the constraints that influenced both the design and construction.

Existing infrastructure

West Ticket Hall

The West Ticket Hall occupies the site of the former 1960s Cardinal Tower and is located to the southwest of the existing Farringdon London Underground station. It is bounded by Cowcross Street to the north and Farringdon Road to the west. Demolition of Caxton House, which formed the southern boundary of the site, assisted construction activities. The new Farringdon Thameslink station ticket hall adjoins the eastern boundary.

A masonry retaining wall on the Farringdon Road boundary dates back to the development of the area as a goods depot in the late 19th century. To the north, the former Cowcross Street bridge structure had previously been infilled with mass concrete supported on piled foundations and fronted with a masonry skin. The former Caxton House building to the south, another 1960s development, has a basement at approximately the same level as the basement of the demolished Cardinal Tower building.

The River Fleet, which originally flowed near the site, now flows in a culvert directly beneath Farringdon Road. A culverted branch of the sewer (the St John's branch), which was diverted as part of the Thameslink Project, crosses the southeast corner of the site. Figure 2 shows the surrounding infrastructure and the listed buildings as per a register maintained by Historic England (previously known as English Heritage).

East Ticket Hall

The East Ticket Hall site is located at the west end of Barbican London Underground station. It is an island site bounded to the north by Charterhouse Street, to the east by Hayne Street, to the south by Long Lane and to the west by Lindsey Street. The London Underground's Metropolitan, Hammersmith and City, and Circle lines run east–west through the north of the site in a cut-and-cover tunnel approx. 10m below street level. The roof of this tunnel is formed of jack-arch construction. The recently decommissioned Thameslink lines to Moorgate (the Moorgate spur) run just to the south in the same cutting and were partially covered by a steelwork deck at general street level.

Masonry arch bridges carry Hayne Street and Lindsey Street over the London Underground and Moorgate spur lines.

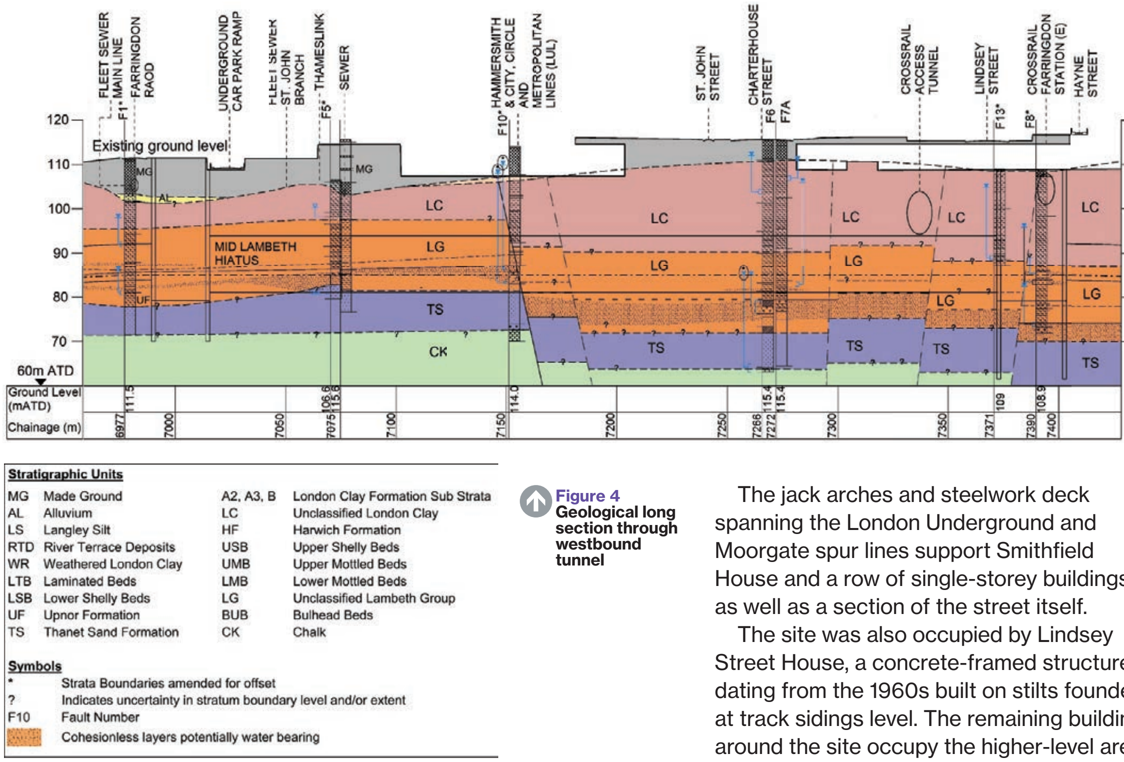

Figure 4 Geological long section through westbound tunnel

Sidings originally used by Smithfield Market extend under Lindsey Street and occupied part of the East Ticket Hall site to the south of the Moorgate spur tracks. A large masonry retaining wall previously cut through the site, forming the change in level between the track/sidings and street level.

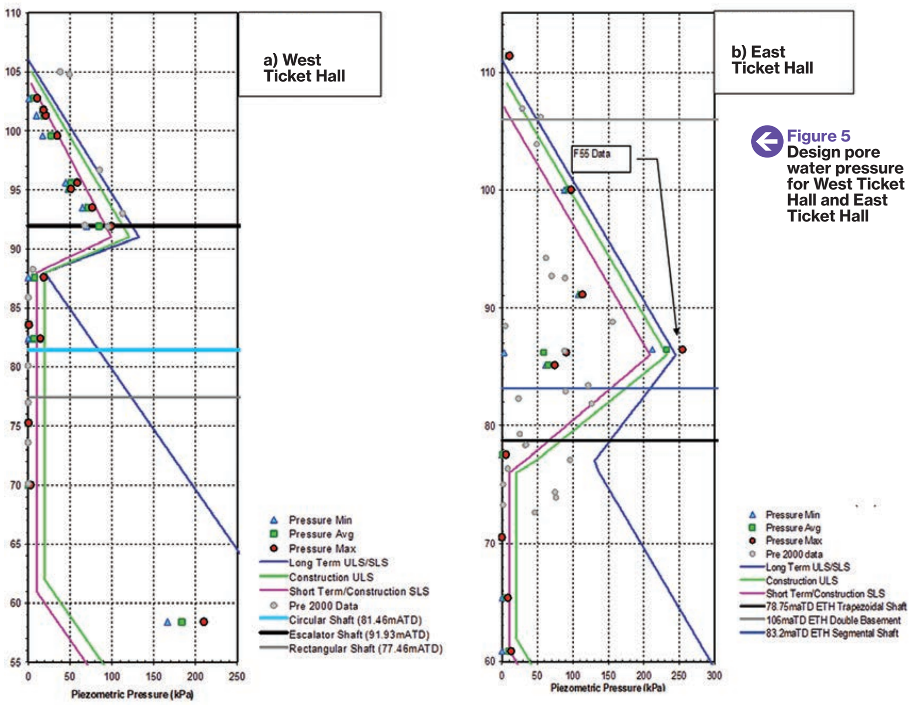

Figure 5 Design pore water pressure for West Ticket Hall and East Ticket Hall

The jack arches and steelwork deck spanning the London Underground and Moorgate spur lines support Smithfield House and a row of single-storey buildings, as well as a section of the street itself.

The site was also occupied by Lindsey Street House, a concrete-framed structure dating from the 1960s built on stilts founded at track sidings level. The remaining buildings around the site occupy the higher-level area behind the large retaining wall and date back to the 19th and mid-20th centuries, with single basements below surrounding street level. The East Ticket Hall site at the time of advanced works is shown in Figure 3.

Ground conditions

The general geology at Farringdon is typical of the London Basin, comprising made ground, alluvium (localised only), river terrace deposits, London clay, Lambeth Group, Thanet sands and chalk[1].

However, the ground around the site is very complex and heavily faulted, with London clay overlying Lambeth Group and Thanet sands (Figure 4). The layer of London clay is relatively thinner over the West Ticket Hall site compared to the East Ticket Hall site. Although the site is under-drained at depth, there are perched water tables and water-bearing gravels which create particular problems for tunnelling activities. The distribution of alluvium is restricted to the infill of the former River Fleet and any associated tributaries over the western part of the site.

There are two aquifers at Farringdon[1]: a shallow aquifer within the river terrace deposits and alluvium; and a deep aquifer within the chalk and the Thanet sands formation and Upnor formation of the Lambeth Group. The two aquifers are separated by the low-permeability London clay and low-permeability units of the Lambeth Group. The shallow and deep aquifers are both water-

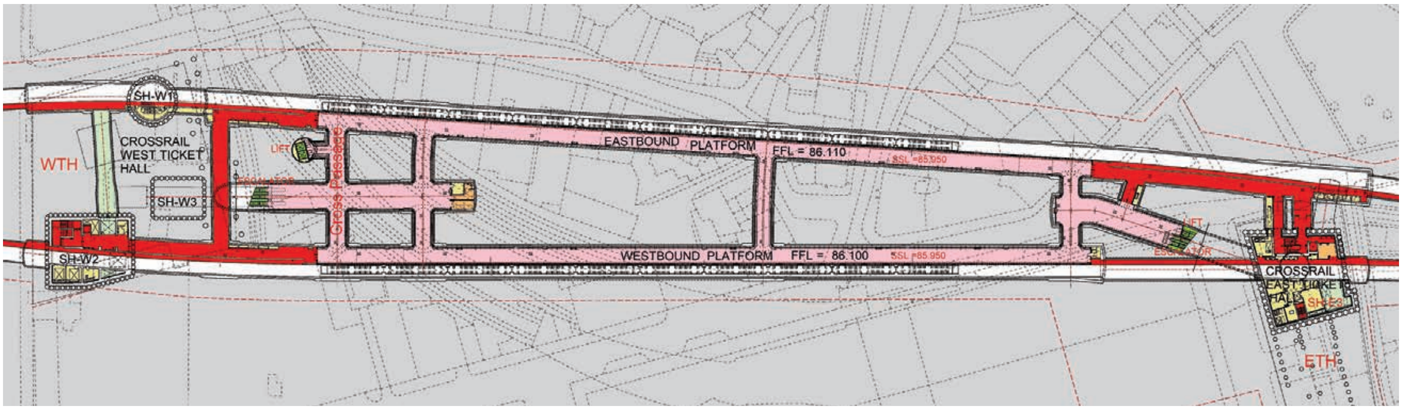

Figure 6
Platforms and passages (pink) were formed using SCL techniques

"THE OVERALL TUNNELLING STRATEGY AND CONSTRUCTION PROGRAMME EVOLVED OVER THE DESIGN PERIOD"

Figure 7
SCL enlargement of eastbound TBM tunnel

bearing. The water table in the deep aquifer is between 60 and 62mATD (meters above tunnel datum). The site geology, geotechnical risks posed by the heavily faulted ground, and presence of water-bearing gravel channels that could collapse during the tunnelling works, as well as the way these risks were mitigated through depressurisation and grouting, have been described in detail by Black (2017)[2], and Davis and Duarte (2015)[3].

Pore pressures were derived and the ultimate limit state values, during construction and in the long term, are summarised for both the ticket hall sites in Figure 5.

Construction strategy

The general tunnelling strategy across the other Elizabeth line station sites was for the 11.0m diameter platform tunnels to be constructed by sprayed concrete lining (SCL) enlargement of the bored and lined running tunnels formed by the passage of the TBM through the station. However, due to the variability of, and the risks posed by, the heavily faulted ground and presence of water-bearing gravel channels that could collapse during the TBM drive, a different strategy was adopted at Farringdon.

The original intention was to use a separate mini-TBM to depressurise the water in these channels and to form pilot tunnels between the two ticket halls which would then be enlarged using SCL techniques. This strategy also allowed for construction of the platform tunnels to start earlier in the programme, avoiding the need to wait for the TBMs to complete their journeys to Farringdon.

To facilitate the early start for the pilot tunnels, it was necessary to build the shafts at the West Ticket Hall quickly. This led to the use of bottom-up construction for the rectangular shaft (SH-W2) and semi-top-down for the circular shaft (SH-W1)[4] (Fig 2).

The overall tunnelling strategy and construction programme evolved over the design period, resulting in significant changes to the design and construction of the shafts. Subsequent geotechnical information, following a British Geological Survey three-dimensional (3D) model of the area combined with a Crossrail geotechnical database compiled over 20 years[3], was better than expected. This prompted a value engineering exercise which resulted in a decision to change the tunnelling strategy to use the larger-diameter TBM drives for the main running tunnel as pilot tunnels for the platform tunnels. The consequences of this decision for the design and construction are discussed in the following sections.

There was no opportunity to construct a cut-and-cover box as Farringdon is split between two sites (the West Ticket Hall site and the East Ticket Hall site) either side of the Grade II* listed Smithfield Market which are over 300m apart (Figure 6). Platforms were formed within the enlarged SCL tunnels (Figure 7). The four shafts at Farringdon are irregular in shape, are of different sizes and depths, and are close to existing rail infrastructure. Access to the piling platforms, several metres below road level, was difficult. Although diaphragm-walled construction can offer considerable benefits, such as smaller wall deflections and reduced overall wall thickness, the size and type of plant, and shape and size of shaft walls, meant it was not considered suitable for the small and constrained West Ticket Hall and East Ticket Hall sites, leading to the choice of secant bored-pile wall construction.

West Ticket Hall

Substructure

At basement level, the West Ticket Hall structure occupies the entire footprint of the former Cardinal Tower site. There is also a substantial sub-basement area for services connections between shafts, which also provides upper and lower machine chambers for the two banks of escalators. Above street level, the floors step back to suit the accommodation required by the Elizabeth line and to maximise the area available to the oversite development (OSD).

A piled raft at basement level is typically 1.5–2.0m deep with local thickening under heavy transfer structures. Vertical loads are carried by a combination of bored piles and shaft walls, and the design caters for the differential stiffness of these two foundation systems. A 2.0m thick and one-storey high cantilever wall, which acts as an inverted T-beam, transfers load from the pile-free zone on the northern side of the site above the eastbound SCL tunnel back to pile raft foundation and shafts. The shaft walls all comprise hard/soft secant piles consisting of 1200mm diameter bored piles at 1350mm centres, with 1200mm diameter soft piles in between. The hard piles were installed from a piling mat at about level 106.0mATD, with the toe level of the wall at 73.0mATD. The soft piles have a toe level into the London clay.

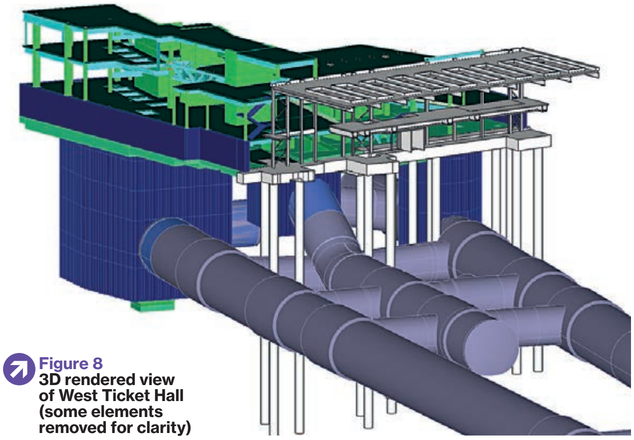

Figure 8 3D rendered view of West Ticket Hall (some elements removed for clarity)

Figure 10 Circular shaft SH-W1 (November 2012)

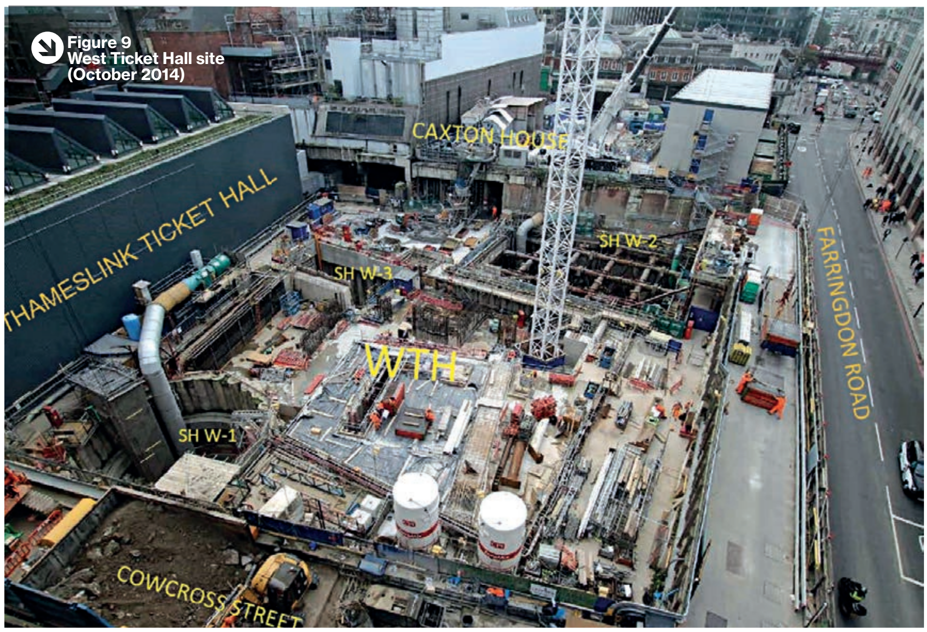

Figure 9 West Ticket Hall site (October 2014)

Figure 8 shows a 3D rendered view of the West Ticket Hall structure along with platform and escalator barrel tunnels. There are three shafts at the West Ticket Hall (Figure 9) and their design and method of construction varied depending on their function.

Circular shaft (SH-W1) on eastbound tunnel

The circular shaft is approx. 13m in diameter and 30m deep. The shaft was originally meant to be used for dropping and launching a mini-TBM for construction of the eastbound pilot tunnel, but the design had to be modified to accommodate the passing of the larger main TBM. Two openings in the shaft were formed using a stitch drilling technique, which provided access for construction of the SCL stub tunnels prior to the arrival of the eastbound TBM. It was then used as a construction shaft for access to platform level to form the enlarged platform tunnels and structures, to complete the fit-out within them, and to remove spoil from the enlargement.

The shaft was constructed using a hard/soft 1200mm diameter secant pile wall and was supported by 1.0m deep circular concrete waling beams during construction. Due to its circular shape, no propping was required during construction, as a series of ring beams resisted the applied lateral loading through compressive hoop forces (Figure 10). Subsequently, six levels of intermediate slabs, generally 600mm thick, were cast integral with the 400mm thick lining wall and circular waling beams through reinforcement couplers cast into ring beams. The 2.0m deep base slab of the shaft was constructed in stages to allow for the eastbound TBM to pass through at a lower level.

In its permanent condition, the shaft houses a service riser and maintenance stairs providing access to station platforms and tunnels.

Rectangular shaft (SH-W2) on westbound tunnel

SH-W2 is a large shaft, approximately rectangular on plan (25m × 21m). This shaft provides for tunnel ventilation, service risers and an emergency means of escape. Originally, the design had to facilitate rapid excavation of the shaft to allow the original pilot TBM to be launched. The subsequent change to the tunnelling strategy required changes to the temporary works propping and sequencing for the permanent works, as well as larger openings for the TBM to pass through.

The shaft is adjacent to Farringdon Road, which contains major services such as the Fleet sewer, major gas and water mains. The need to excavate the shaft rapidly had dictated the use of bottom-up construction, but ground movements were kept within specified limits by using stiff, tubular steel temporary frames at close centres. The shaft was propped with temporary steel frames at several levels as it was excavated, but the permanent works (i.e. the concrete walls and slabs) were constructed from the bottom up.

The shaft was excavated in two stages. In the first stage, it was only excavated to the level which allowed the TBM to bore through.

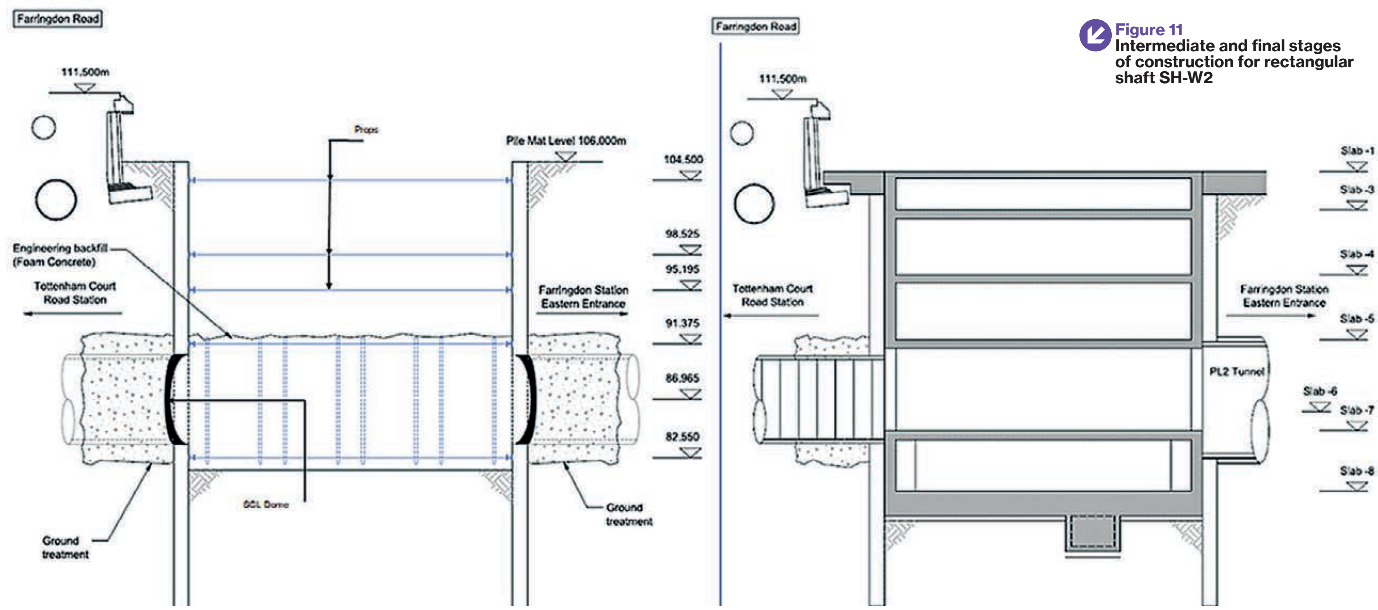

Figure 11 Intermediate and final stages of construction for rectangular shaft SH-W2

Subsequently, it was excavated further to base slab formation level (Level –8). Secant pile walls at both the east and west faces of the shaft were cut using stitch drilling techniques from inside the shaft, to form SCL domes and also to allow ground improvement works to be carried out before the arrival of the TBM. For stability, the shaft was filled with engineering fill (foam concrete) up to a height of 10m before the westbound TBM cut through the SCL domes.

The temporary propping installation and removal was sequenced such that it allowed sufficient headroom and construction access for operatives and machinery for downward excavation and for upward slab and wall installation. Figure 11 shows one of the intermediate stages and the final stage of the SH-W2 shaft construction sequence.

Lining and internal flange walls were installed to temporarily support the pile walls before the intermediate slabs were cast and propping was removed. The lining walls are subjected to full earth pressure due to several openings formed in the perimeter wall resulting in discontinuity of piles. The load path to the base of the piles is interrupted where the piles are cut for vent adits, running tunnels and platform tunnels. The lining walls are connected to the piles by drilled-in reinforcement such that the lining walls which span across the opening can act as a deep beam to transfer load from above, and also horizontal loads from the cut piles, back to the piles at lower slab levels.

Escalator shaft (SH-W3)

The (15m × 11m) escalator shaft was constructed bottom-up with one level of temporary propping. Its primary function was to facilitate the construction of an inclined SCL escalator barrel. It is a rectangular shaft/box structure, which is significantly shallower than the other two shafts, as it does not extend down to track level. It essentially acts as a work site from which the escalator barrel can be constructed down to platform level.

Superstructure

The ground floor and superstructure are steel-framed, with the steel beams generally designed to act compositely with reinforced concrete slabs. Slabs are cast on profiled metal decking which acts as permanent formwork. In view of the 120-year design life[5], the completed slab is reinforced to carry full design loads without any reliance on the metal decking. The frame is braced by reinforced concrete shear walls.

The OSD will be supported directly by the superstructure beams, columns and walls at predetermined locations. Storey-deep twin transfer trusses carry column loads from the OSD columns and span 21m over the apse and the service yard to provide a column-free zone at ground level. There are also more minor transfers of columns at the building perimeters.

Three main reinforced concrete cores provide overall stability for the West Ticket Hall and OSD structures and will interact with the stability of the integrated ticket hall when complete. The cores are designed to accommodate a maximum 10-storey OSD which will enable the full potential of the site to be realised. Horizontal wind loads are transmitted to these cores via the floor plates. In addition to the cores, portalised bays provide stability at Level +1 (mezzanine floors).

The Thameslink ticket hall, Elizabeth line West Ticket Hall and OSD are not separated by movement joints and therefore effectively act as a single structure.

The top level of each structure is designed as a crash deck, in accordance with the requirements of the Crossrail Civil Engineering Design Standards document[5], with plinths designed to accommodate the construction of the OSD columns. Cores typically incorporate couplers to allow them to be extended to full height throughout the OSD. Reinforced concrete crash walls are also provided between the OSD and the station.

All steel columns are encased in concrete for durability and fire protection, as are steel beams above the service yard.

East Ticket Hall

Trapezoidal shaft (SH-E3)

The East Ticket Hall is built over a very constrained site between Hayne Street and Lindsey Street. A two-storey basement over the southern half of the site houses mechanical and electrical (M&E) plant, while the northern half of the site is occupied by a large deep trapezoidal shaft (SH-E3), measuring 30m × 25m on plan, which extends 35m below road level.

Its plan size is dictated by the need to accommodate tunnel ventilation, service risers, plant rooms, vertical transportation (two escalators and an inclined lift) and emergency means of escape. Space-proofing requirements have meant that the shaft extends to the site boundary on three sides. A part plan at Level –2 (Moorgate spur level) is shown in Figure 12.

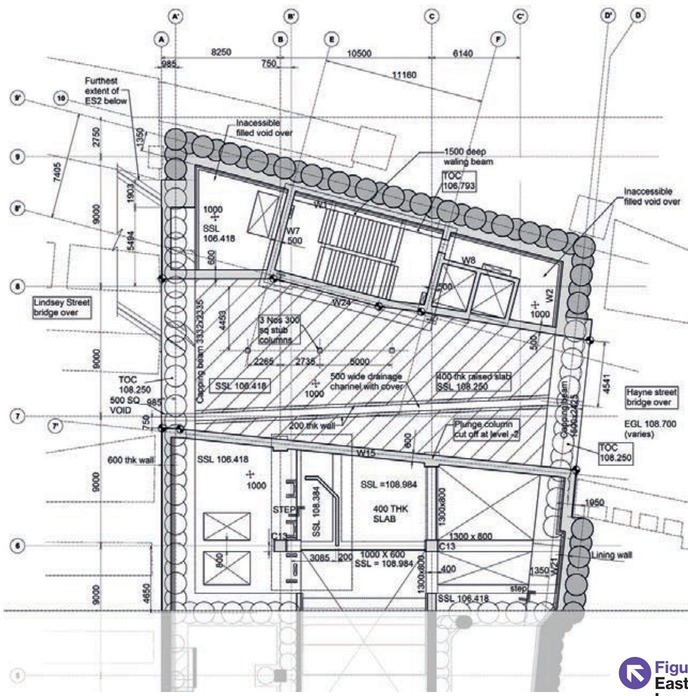
Figure 12
East Ticket Hall part floor plan at Level –2 (Moorgate spur level)

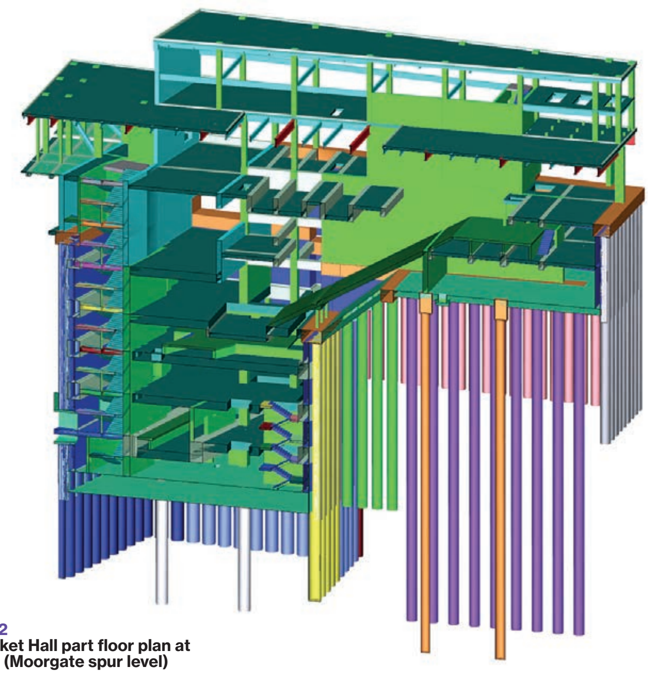
Figure 13
East Ticket Hall 3D long section rendered view

The trapezoidal shaft was constructed using hard-soft secant piles installed generally from the Moorgate spur level (Level –2, approx. 109mATD). The size and spacing of these piles are dictated by horizontal and vertical loading and lateral supports from floor diaphragms. Both secondary (male) and primary (female) piles are 1500mm in diameter. These are some of the longest (up to 51m long) cased piles (casing length up to 37m) ever constructed in the UK. The toe level of the wall is 12m below the base slab formation level at approx. 66.0mATD. A reduction in pile diameter below the temporary casing level (79.95mATD) from 1500mm (casing outer diameter) to 1400mm (auger diameter) resulted in a reduced size for the piling cage.

A 3D rendered long sectional view of the East Ticket Hall is shown in Figure 13.

The shaft is connected by the westbound running tunnel (twice), escalator barrel from the Elizabeth line platforms, two cross-passages and a ventilation adit from the eastbound Elizabeth line platform. Due to such a large number of openings on three sides of the shaft, vertical and horizontal loads are transferred through a complex mechanism utilising the combined stiffness and strength of the secant piles and the lining wall. At many locations, lining walls act as a deep beam and carry loads across the openings. Lining walls are connected to the secant pile wall through drilled-in steel reinforcement. Figure 14 shows the construction status of the East Ticket Hall in October 2014.

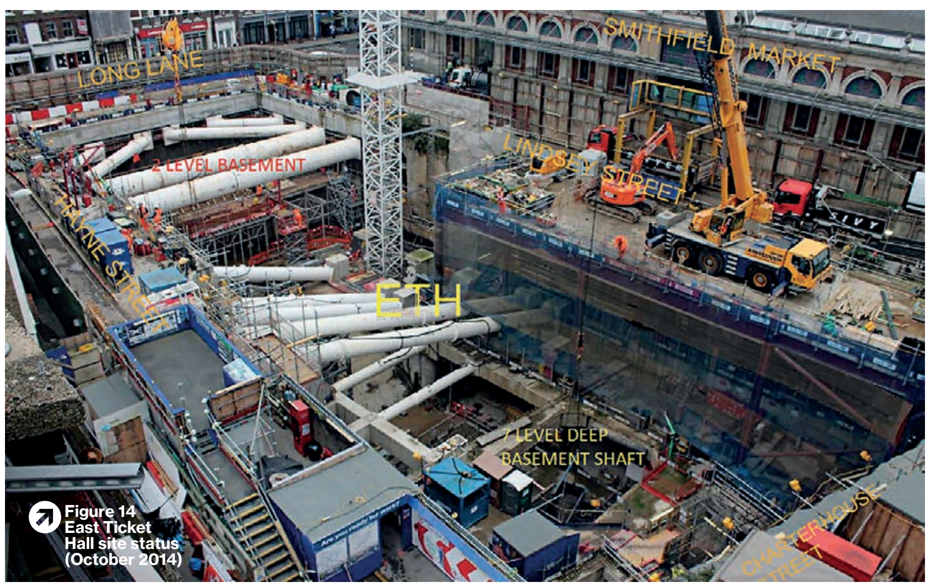

Figure 14
East Ticket Hall site status (October 2014)

The shaft is arranged on six floors commencing at basement Level –2 down to the base slab at sub-platform Level –7. Internal floors within the shaft are of reinforced concrete construction, generally 1.0m thick. In the permanent condition, the floors are supported at the perimeter by the secant piles and internally by columns and walls constructed off a 2.5m thick slab at the base of the shaft. The base slab, designed as a piled raft, also resists the upward loads due to long-term hydrostatic pressures and soil heave.

The trapezoidal shaft was built using top-down construction methodology, which integrates the temporary and permanent works. Top-down construction is inherently stiffer than bottom-up and therefore tends to reduce wall displacements and associated ground movements. This was considered to be highly beneficial in the case of the trapezoidal shaft, the perimeter of which is very close to a number of London

Figure 15
Top-down construction of trapezoidal shaft SH-E3 with provision for construction access

Underground and other third-party assets, including the Grade II* listed Smithfield Market. Figure 15 shows a seven-level-deep shaft cast top-down with construction access using plunge columns. Figure 16 shows preparation for the base slab (Level –7) construction.

During construction, the edges of the slabs were supported on the perimeter secant pile walls; internally, support was provided by four steel plunge columns made up of built-up I-sections of size 500mm × 600mm, which were 'plunged' into concrete bored piled foundations from ground level, before the concrete had set. Shear studs welded to the columns provided support to the slabs. These steel plunge columns were subsequently cast into the base slab, reinforced concrete columns and walls, forming part of the permanent works. When required by the design, some parts of the lining wall were also cast during top-down construction.

Once the base slab was cast, bottom-up construction of the lining walls, internal columns, walls and stairs could commence. There were a number of programme-critical activities associated with the construction of the shaft, including the time-consuming nature of the excavation and construction of the trapezoidal shaft before the arrival of the westbound TBM, which stopped just outside the shaft.

Superstructure

Structural slabs at Levels +1 to +3 are generally cast *in situ* concrete slabs on profiled metal decking. They are designed on the assumption that the profiled decking acts as permanent formwork only. The composite metal decking floor slabs span between steel beams supported by concrete-encased steel columns. All columns are encased in concrete for durability and fire protection, as are beams above the ticket hall and within the large station fan plenum at Level +2.

Where the OSD columns do not align with the station columns below, transfer beams are provided. Cantilever beams are used to support overhanging OSD columns at the northern and southern boundaries. Raking columns, formed of built-up steel sections, are provided at the north end of the site to form transfer structures cantilevering over the London Underground tunnels. Horizontal forces from the raking columns are transferred through floor diaphragm action and reinforced concrete shear walls to the shaft substructure.

The ground-floor slab in this area is hung from first-floor level and a compressible material has been provided between the underside of the Elizabeth line structure and the top of the existing jack-arch structure to further ensure no load is imposed on the London Underground jack arches.

Oversite developments

At both the West Ticket Hall and East Ticket Hall sites, the new ticket hall structures and their foundations are designed to support future OSDs of a maximum of 10 storeys. Both ticket halls incorporate stub columns and a crash deck to allow construction at a future date. The Elizabeth line stations therefore do not form completed architectural entities, but rely on the future OSDs to re-establish the urban setting.

The West Ticket Hall OSD is a so-called 'collaborative' development. This meant that close collaboration was required with the chosen developer and its professional team to agree locations for the columns and cores, and also for the load imposed on the station structure from the OSD. The presence of existing piles at the West Ticket Hall from the demolished building and the area sterilised by the tunnels imposed severe restrictions on the location of new piles. Some piles are sleeved to avoid load transfer from the piles to the tunnels and/or to reduce the impact of negative skin friction. A deep raft slab, together with a series of transfer structures, carries superstructure loads to the piled raft foundation and secant piled walls of the three shafts.

The East Ticket Hall OSD is not a collaborative development, as no developer had been identified during the design of the ticket hall. A design was therefore developed by a separate AECOM team for coordination with the station design. The team has generated a rationalised grid and floor plan layout incorporating a central core for an OSD of a maximum of 10 storeys to optimise the value of the site. The design has assumed a lightweight composite floor construction for the OSD. Figure 17 shows the architect's impression of the East Ticket Hall with completed OSD.

Design summary and lessons learned

The final station design was influenced by a number of factors, such as changing tunnelling strategies, continually updated geotechnical information, the contractor's construction sequence/methodology and programme changes. The ground conditions, particularly the presence of at least four major and four minor faults and water-bearing sand

Figure 16
Construction of East Ticket Hall base slab (Level –7) (plunge column exposed)

layers across the site, presented significant geotechnical challenges.

Several bespoke solutions had to be developed to build the SCL tunnels and the ticket hall structures. To mitigate the risk of bore hole collapse, piles had to be temporarily sleeved up to a length of 35m from the ground level. Piles located very close to tunnels were permanently sleeved to a depth determined by the design. Some of the secant piles were cast with localised fibre-reinforced plastic reinforcement for subsequent ease in forming openings through the wall. Similarly, some secant piles required to carry vertical load, but cut partially to form openings in the wall, were cast with a small steel I-section spanning across the opening.

The basement slabs and walls within the shaft structures had to accommodate a large number of irregular and asymmetrical openings, discontinuity, restricted head room, staged construction and temporary provisions for construction access. This led to development of many innovative solutions through complex load paths, transfer mechanisms and finite-element modelling. Both of the ticket hall structures incorporated a number of precast concrete stairs, modular units and prefabricated steel elements. Overall design loads expected from the station structures, including the future OSD, downward negative skin friction load due to tunnelling works and restrictions on pile toe level, meant that all base slabs (except the two-storey-deep East Ticket Hall) had to be designed as piled raft foundations. Thus, there was no opportunity to use void formers below these base slabs to alleviate uplift pressure from both short- and long-term soil heave.

Shaft walls were constructed using secant bored piling. Top-down construction was used, except where programme requirements dictated the need to complete shaft excavation at the earliest opportunity. Ground movements were controlled by a combination of the sequence of excavation and the stiffness of perimeter walls and horizontal props. Third-party liability due to surface settlement was controlled using grouting and depressurisation along the route.

Elizabeth line station structures are designed to the Eurocodes and are compliant with the Crossrail Civil Engineering Design Standards[5]. Computer-assisted (CAD) drawings were produced in 3D using Building Information Modelling (BIM)-compliant Bentley Systems MicroStation[6]. The composite CAD models contained both civil and services fit-out information. Co-location of design teams greatly enhanced collaboration between all stakeholders.

Conclusions

One hundred and fifty years after the world's first underground railway was built in London between Paddington and Farringdon, the time had come to provide a new railway fit for the 21st century and with it a new major interchange at Farringdon.

Building a new station of such a scale that it stretches between two London Underground stations at Farringdon and Barbican is no small undertaking in the heart of London, surrounded by historic buildings and infrastructure.

The key drivers of safety, economy and constructability have underpinned the resolution of the constraints and challenges posed by the ground conditions, heritage, infrastructure, sequencing and programme. Equally important has been the collaboration with other consultants, third-party stakeholders and the contractor to develop and implement the solutions required to resolve these issues.

The construction of heavy civil and structural elements of the station structures is now complete and has given way to fit-outs, trial running, and commissioning of Elizabeth line operations ahead of services commencing in December 2018. The new Farringdon station has become a catalyst for regeneration of the local area, bringing many new opportunities.


Figure 17 Architect's impression of East Ticket Hall when complete with OSD

Project team

Client: Crossrail Ltd
Station lead designer: AECOM (URS/Scott Wilson)
Civil and structural engineer: AECOM
Architect: Aedas
MEP engineer: AECOM
SCL platform tunnels: Mott McDonald
Main contractor: C435 - Bam Ferrovial Kier JV (BFK)
Temporary works designer: AECOM (scheme design); C435 - BFK (final design)

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A model railway station: implementing a 'kit of parts' solution to Custom House – the only above-ground station on the Elizabeth line's central section

Ed Newman-Sanders
CEng, MICE
Technical Director, Atkins, UK

Rosemary Smiley
MEng
Structural Engineer, Atkins, UK

NOTATION

DfMA	Design for Manufacture and Assembly
DLR	Docklands Light Railway
ETFE	ethylene tetrafluoroethylene

Synopsis

The new Elizabeth line station at Custom House was a unique opportunity for design and construction. It is the only above-ground station on the central section of the line and will welcome millions of visitors to London's largest conference centre, ExCeL, as well as providing vital connections for the Borough of Newham.

A joint team from Crossrail Ltd, Atkins, Arup, Allies & Morrison, and Laing O'Rourke collaborated to develop the striking station design, creating a beacon for both the Elizabeth line and the local community. Faced with many constraints, a 'kit of parts' strategy was developed for Custom House's construction, including prefabricated and standardised components.

This approach – where much of the station was built off site – minimised work on site, drove down programme times and costs, and reduced the impact on the local community. The approach also led to Custom House's excellent health and safety record – one of the best of any Elizabeth line station to date.

Introduction

The new Elizabeth line station at Custom House was a unique opportunity for design and construction. As the only above-ground station on the central section (Figure 1), it embodies the Elizabeth line's vision and identity.

The station will welcome regional and international visitors to London's largest conference centre, ExCeL London, and create an important transport interchange with the Docklands Light Railway (DLR) and local bus services. It also provides a focus for the regeneration of the local area, the London Borough of Newham.

The station development is made up of two parts: a new 24-hour public route from Custom House to ExCeL and the Royal Docks; and the Elizabeth line station itself, an elevated concourse ticket hall above an island platform.

Overcoming constraints

The Custom House site (Figure 2) presented a number of constraints:

- The existing DLR runs south of the main site, for the full length of the station. As an operational railway, it meant that constraints were imposed on contractors working adjacent to the boundary.
- The north boundary of the main site lies along the footpath to the south of Victoria Dock Road. This is a busy main road, carrying normal traffic, buses and pedestrians.
- A public right of way across the site enables pedestrians to travel from Victoria Dock Road to the DLR station and ExCeL, located to the south of the DLR station. This had to be maintained at all times, including access for people with restricted mobility.
- Although the majority of the site was clear of utilities, there were a number of major services running along the southern footpath of Victoria Dock Road, including a 600mm diameter intermediate pressure gas main and 12in. cast iron water main.
- A line of high-voltage cables overhangs

Figure 1
Stations on central London section of Elizabeth line

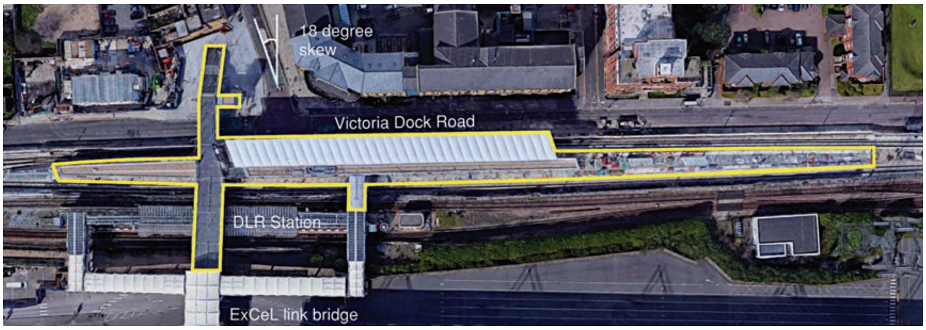

Figure 2
Plan of Custom House station showing 18° skew of site derived from 'urban grain' (relationship between Freemasons Road and Victoria Dock Road)

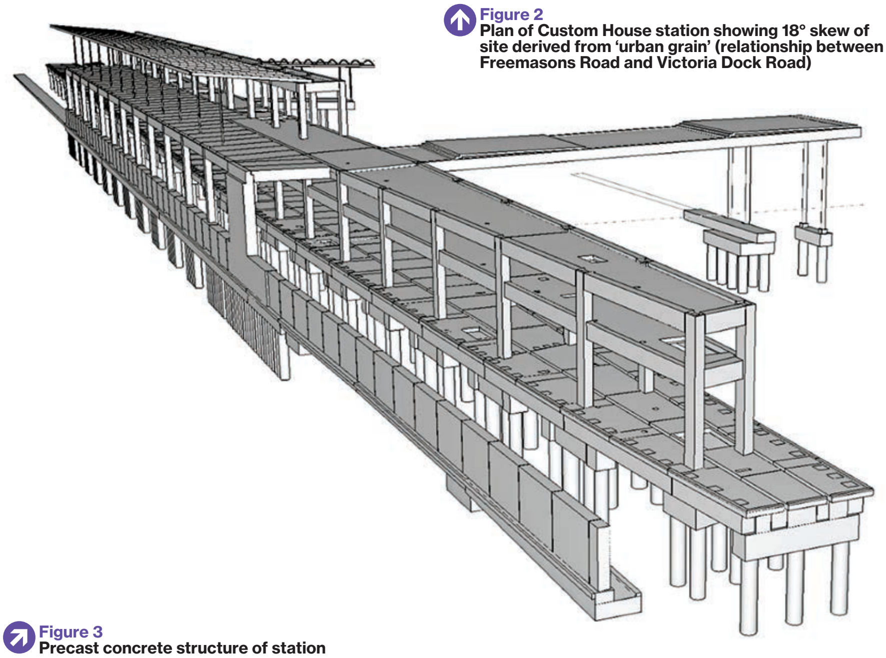

Figure 3
Precast concrete structure of station

Figure 4
A-frame component highlighted in red

the DLR to the south of the site, stretched from pylons to the east and west of the station. While not overhanging the main station site, they were close enough to be a major source of risk to any lifting operations on the site.

'Kit of parts' approach

The strategy for the construction of Custom House station included prefabricated and standardised structural components, with a 'kit of parts' forming the platform, columns, concourse slab and roof.

This unusual and innovative approach had a number of advantages:

- Work on site was minimised, driving down programme time and preliminary costs, and reducing the impact on the local community.
- Off-site manufacture took place during the Olympic 'blockade', further reducing programme pressures (during the 2012 London Olympics the road network needed to be clear – this did not affect construction of Custom House as much of the work took place off site).
- There were fewer deliveries and vehicle movements around the site, lessening the impact of traffic, noise and air quality on the local community.
- Construction activity was shifted from site to factory, improving working conditions and reducing health and safety risks.
- The more controlled conditions of the factory ensured more consistent and higher-quality production.
- The need for applied finishes was reduced, decreasing programme time, simplifying procurement and potentially lowering costs.

"THE CONTRACTOR CONVERTED THE PRECAST CONCRETE FRAME DESIGN TO FULL DfMA"

The development of a precast concrete solution (Figure 3) brought benefits to both the design and construction phases of the project. It used repetitious units, manufactured in factory conditions to a high standard and consistent finishes, which were delivered to site in batches to coincide with the construction programme. Swift installation by crane was made more acute by adjacent live railway overhead power lines and the restrictions this could have on the construction sequence.

As the project's main contractor, Laing O'Rourke utilised its Explore manufacturing facility in Nottinghamshire to fabricate the major components, which were then delivered on a 'just in time' basis to the site for positioning and commissioning.

The contractor converted the precast concrete frame design to full Design for Manufacture and Assembly (DfMA), splitting large A-frame elements (Figure 4) to be more easily transportable and adopting mechanical connections between the primary components.

This approach is revolutionising construction in the commercial building sector, but this was one of the first applications in a major rail infrastructure project. The seamless integration of the 'virtual' design model and the off-site manufacturing plant allowed the team to create highly precise, major structural elements, delivered exactly when needed. This innovative strategy has great potential for railway infrastructure projects in the years ahead.

Design idea

With Custom House being a new above-ground station, the team had the opportunity to design it as a free-standing building rather than an interior fit-out. This provided scope not only for more architectural expression, but for the station to serve as a beacon, both for the Elizabeth line and the surrounding community (Figure 5).

In addition to the system-wide equipment and wayfinding signage that is part of the Elizabeth line brand, the architectural design had to recognise Custom House's other ambitions: in terms of place-making, fitting into the urban setting of Newham, and its role as an ambassador for the capital's new rail transport system.

Robert Maxwell, Allies & Morrison's lead architect for Custom House, has described it as 'an urban temple' (Figure 6), elaborating: 'At its simplest, the form of the building laid out at the southern end of Freemasons Road produces a tripartite architectural composition. The plinth, or base, consists of a continuous monolithic wall needed for asset and vehicle collision protection; the principle facade, or middle, the colonnade capped by the edge and balustrade of the concourse; and the roof, ETFE [ethylene tetrafluoroethylene] pillows supported on slender steel columns' (Figure 7).

The shape of each of the structural columns is a parallelogram rather than orthogonal, with a rotation of 18°. This is derived from the relationship between Freemasons Road and the urban grain of the neighbourhood with the Victoria Dock Road that runs parallel to the railway lines (Fig. 2). This rotation is also combed through the floor finishes and steel superstructure supporting the roof.

At platform level, folded planes were introduced to the precast concrete soffit panels supporting the concourse. These fold in alternating directions to provide a simple vaulting pattern that is enhanced and lifted by edge lighting.

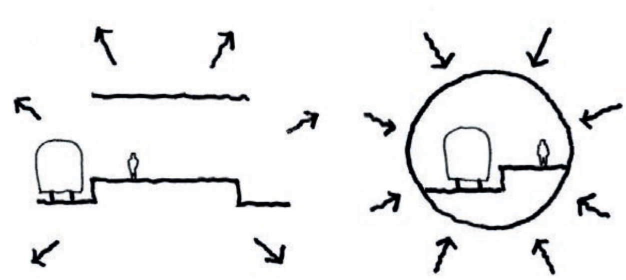

Figure 5
Unconstrained by underground setting, Custom House can serve as beacon for Elizabeth line and community, with transparent views and iconic architecture

The arrangement of the station produces a simple legible route from the entrance to the train doors (Fig. 7). The upper level is intentionally generous and open in feel to aid orientation and route selection for passengers. This open aspect enables observation and passive surveillance for both those approaching and within the station.

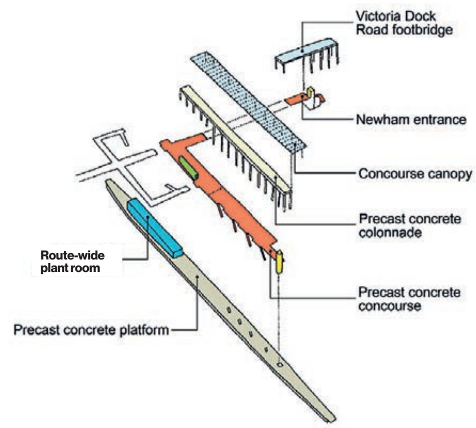

Figure 7
Exploded view of station's form

Design solution

Making good foundations

The made ground on the site is underlain with approx. 4m of alluvium, which includes peat layers. The station and track bed structures require piled foundations as a result of the unacceptable predicted settlements associated with building a ground-bearing structure on soft peat material. Piles were up to 25m long, with diameters of 450mm, 600mm and 750mm. The foundations are designed to bridge over the services running below the site where required, with continuous flight augured (CFA) piles and *in situ* concrete pile-caps supporting the superstructure.

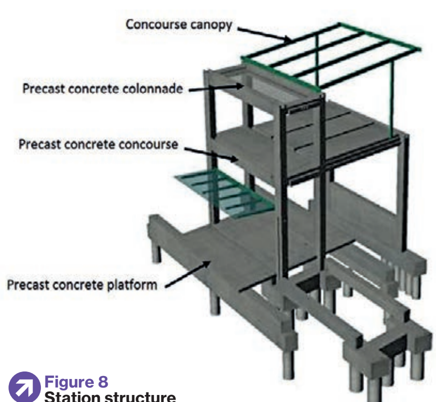

Figure 8
Station structure components

Platform

The platform structure comprises precast concrete panels spanning onto a system of primary beams supported on the *in situ* pile-caps and piles (Figure 8). Typically, there

Figure 6
Architectural design for Custom House station is intended to recall colonnaded front of temple

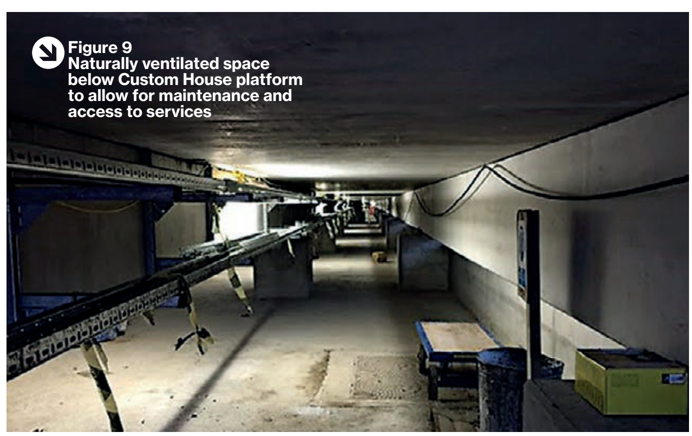

Figure 9
Naturally ventilated space below Custom House platform to allow for maintenance and access to services

Figure 14
Precast panels with triangular vaulted soffits are used on main span of concourse structure

Figure 10
Concourse structure along Victoria Dock Road with precast 'L' beams waiting to receive coffered floor panels

"THE CONCRETE SUPERSTRUCTURE CONSISTS OF A SERIES OF PRECAST REINFORCED CONCRETE FRAMES, COLUMNS, BEAMS AND FLOOR UNITS"

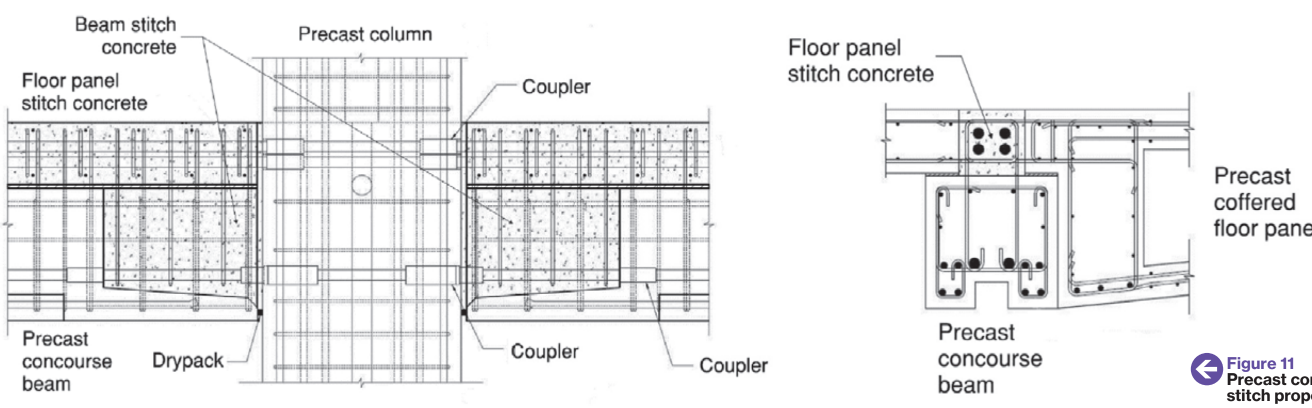

Figure 11
Precast concrete wet stitch proposal

Figure 12
Hybrid mechanical and wet stitch solution for typical beam

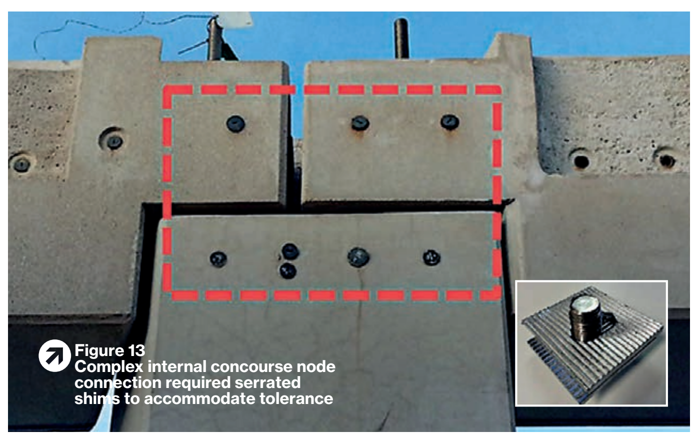

Figure 13
Complex internal concourse node connection required serrated shims to accommodate tolerance

are three or four lines of beams which span between the pile-caps, depending on the width of the platform and the location of the platform drainage channels.

There is a naturally ventilated space below the platform for maintenance of the services running below it (Figure 9). The services include two 11kV cables powering the Elizabeth line, drainage pipes for the roof and concourse drainage, and other electrical and communications services serving the platform.

Main station structure

The concrete superstructure consists of a series of precast reinforced concrete frames, columns, beams and floor units. In section, the beams are either rectangular, 'L' or inverted 'T' shapes depending on the precast floor units they are required to support (Figure 10). All the precast units were designed to be attached together with a hidden *in situ* concrete stitch. The key structural design challenge for Custom House was reconciling the need for the primary precast component connections to be 'invisible', while having sufficient strength and stiffness to transfer the connection forces generated from moment frame action.

The wet stitch proposed by the design

team for this connection (Figure 11) was eventually replaced by mechanical bolted connections (Figures 12 and 13). The advantage of this method was the lack of temporary propping required at the beam ends. The approach satisfied the original architectural intent, except in a few locations where discrete half-joint connections were adopted. This did, however, reap greater benefits in terms of constructability and temporary works.

The main span of the concourse structure consists of precast panels with triangular vaulted soffits (Figure 14) and beams along each long edge, which are supported by east–west primary beams. The profile of the soffit planes was determined primarily by the architectural intent, but also by the prefabrication process in which the release of the finished component from the moulds and the need for void formers within the panel, in order to limit weight, had to be considered.

With the open-plan nature of the station environment, braced or shear wall stability structures were not viable, so the lateral stability of the main station building is derived from a series of portal frames in both directions (Figure 15). Lateral loads are transferred to these stability elements by the concourse and roof structures being wholly or partially stitched together to act as a stiff diaphragm. Wind loading from the north and south is transferred through the structure via the A-frames, and wind loading from the east and west via the precast concrete colonnade.

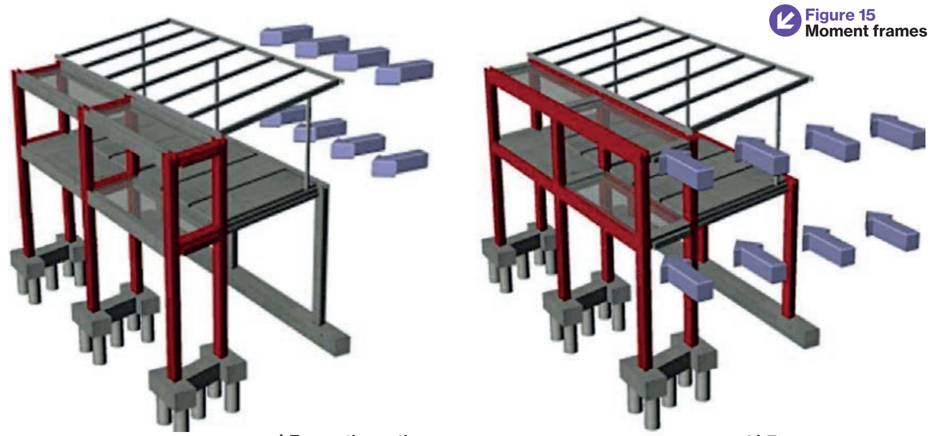

Figure 15 Moment frames
a) For north–south lateral loads
b) For east–west lateral loads

Roof structure
The steel roof structure supporting the ETFE pillows comprises circular columns and fabricated box-section beams (Figure 16). The columns are set back from the edge of the roof structure and are supported at their base by the precast panel beams described above. The roof members are connected by concealed bolted end-plate connections and all visible welds have been ground flush and smooth.

The ETFE pillows are connected on all sides to aluminium extrusions, which also form the watertight seal and gutter. In order to connect these extrusions to the steelwork, discrete L/T-sections are required on top of the steel roof members.

Victoria Dock Road wall
The key challenge in the design of the Victoria Dock Road wall structure and foundations was the limited space available both above and below ground level. With the footpath, and associated services, immediately to the north and the Elizabeth line tracks immediately to the south, the 2000kN and 500kN train collision loads need to be resisted by a 500mm wide linear structure. This was achieved by casting structural steel sections into the precast concrete column/wall units (Figure 17) and having linear pile clusters beneath the columns. As with the precast column bases, the wall units were connected to the cast-in anchor bolts in the *in situ* pile-caps using stainless steel column shoes.

Figure 16 ETFE pillows are supported by steel roof structure with circular columns and fabricated box-section beams

Footbridges
There are three footbridges linking the DLR station, ExCeL arena and the London Borough of Newham to the new Elizabeth line station. All three structures comprise steel fabricated primary box-beams with a composite slab cast on permanent precast concrete planks. Of these bridges, the ExCeL footbridge had the largest span and was the most challenging to construct due to the proximity of the overhead power lines. This was achieved by using two cranes working in tandem (Figure 18).

Construction
Crane decision
Before construction started, a major decision was taken to move from the use of a mobile crawler crane to a gantry crane for erecting the main works (Figure 19). Although common in the shaft and tunnelling world, using a gantry crane is an innovative solution for above-ground station construction.

The primary driver for the gantry crane solution was health and safety. How could the team effectively manage a crawler crane in such a restricted area with so many

Figure 17 Victoria Dock Road column/ wall structures

variable interfaces? The team decided the answer was that it simply could not. Using a gantry crane significantly reduced major risks such as collapse radii and proximity to the 400kV overhead power line. Another major benefit of the gantry crane was its ability to track back over the structure once erected.

Kit of parts in action

With the connection design for the 880 precast units (each weighing up to 34t) completed, all hidden within the structural envelope (Figure 20), the next task was to ensure that the team could achieve the architectural finish. Service void dimensions and shapes, chamfers and drip details, and visible joint arrangements all needed careful detailing for the DfMA process. Prototype units were cast to check the manufacturing process was working as expected and to confirm the finish quality.

Due to the size of the units and the number of handling operations required, both in the factory and on site, the lifting solution needed careful planning. With no space in the top of the units for cast-in lifting points, the team developed a bracket that used the permanent works connections. This ensured the units were adequately protected during the horizontal to vertical pitching process.

Going digital

Modelling all of the components at Custom House in three dimensions (3D) provided an invaluable communication tool to aid conversations between teams around buildability, site inductions, logistics, sequencing and health and safety. The 3D model was also linked to the project programme in Synchro[1] to visualise and plan the complex sequence of installation. This was in turn linked to Laing O'Rourke's factory database.

Figure 18 ExCeL footbridge, largest at Custom House, is lifted into place

Figure 19 Gantry crane was used to reduce risk from station's close proximity to overhead power lines

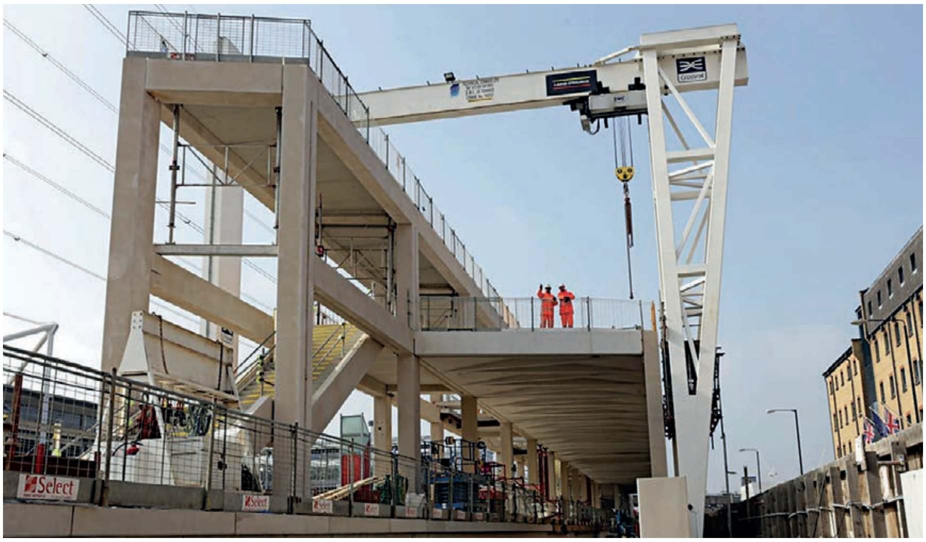

Figure 20
Custom House station used 880 precast units

By using unique QR codes on each component, the team was able to track, plan and record the status of each of the 880 precast components from the design stage through to casting, delivery and installation on site. Once on site, mobile devices scanned the QR code and brought up the necessary quality form to complete. This provided an efficient way to carry out all quality control and health and safety checks, maximising traceability and simplifying the handover process.

Conclusions

Custom House's success resulted in the project winning 'Infrastructure Project of the Year' in the Explore Offsite Awards, and 'Offsite Construction Project of the Year' in the London Construction Awards. Using a 'kit of parts' approach, the close collaboration of Atkins, Laing O'Rourke, Arup and Allies & Morrison developed a solution that was elegant, durable, cost-efficient and safe to erect on the constrained site (Figure 21), with the majority of the fabrication process taking place in a controlled factory environment. The team not only overcame the unique challenges for design and construction presented by the site, but also delivered the iconic 'ambassador' for the new Elizabeth line network they were seeking. A real-life model railway station.

Acknowledgements

The authors express their thanks to Robert Maxwell, who led the architectural team at Allies & Morrison, and also to Cameron Corsby, Laing O'Rourke site engineer for the duration of the project.

Atkins is a part of SNC-Lavalin's Engineering, Design and Project Management business.

Project team

Structural engineer: Atkins
Client: Crossrail Ltd
Building services engineer: Arup
Architect: Allies & Morrison
Main contractor: Laing O'Rourke

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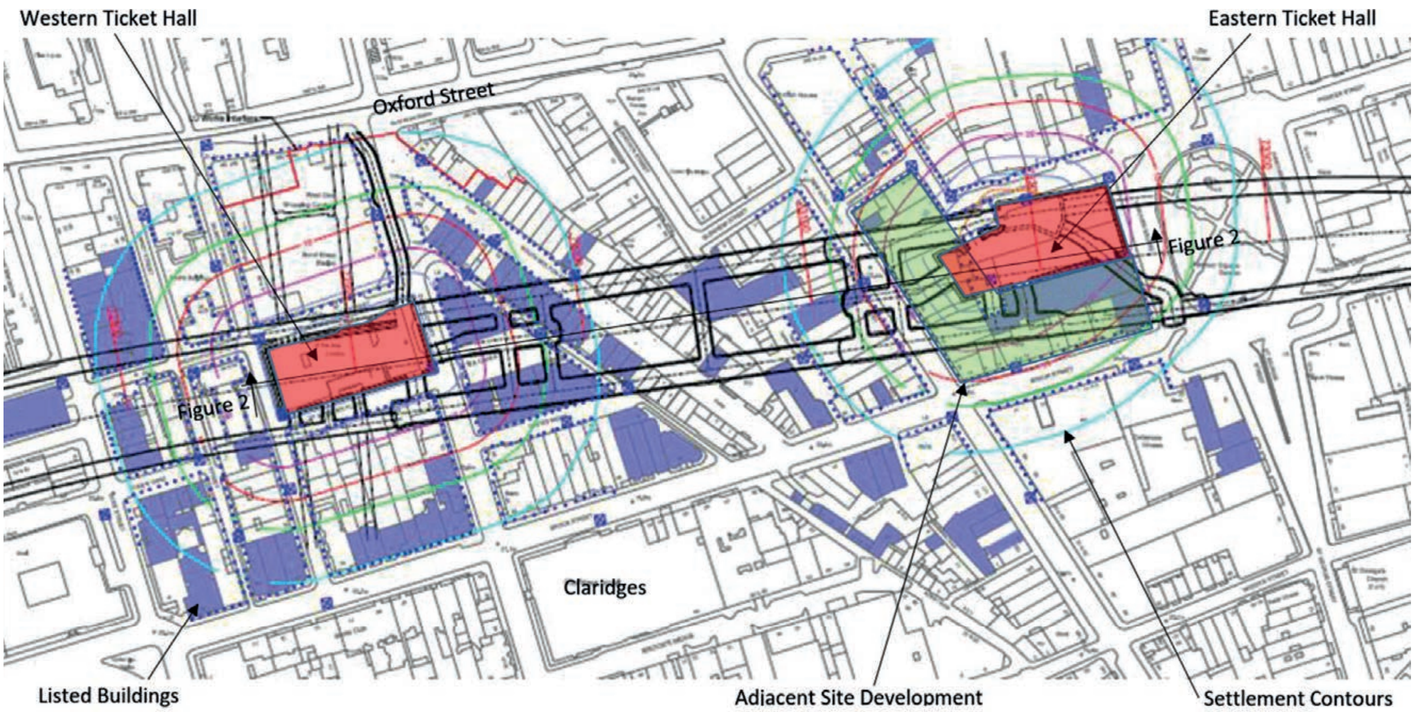
Figure 21
Custom House station in final stages of construction

Evolution of the design for Bond Street Elizabeth line station

Figure 1 Street plan with station overlaid



Rob Paul
BEng (Hons), CEng, FIStructE, FICE
Technical Director, WSP, London, UK

NOTATION

ASD	adjacent site development
CAD	computer-aided design
GFRP	glass fibre-reinforced plastic
GHS	GHS Limited Partnership
MEP	mechanical, electrical and public health
OSD	oversite development

Introduction

The design of Bond Street Elizabeth line station has evolved over 10 years of design and construction work. This article explains how the design has developed over this timeframe and how the independent designs for two clients were successfully delivered on the same site. It will discuss how the site constraints have informed the design, how the station was designed to be constructed and how it was ensured that the design has been assured throughout.

Background

WSP was appointed by Crossrail Ltd in September 2009 as the framework design consultant for Bond Street Elizabeth line station. The station comprises two ticket halls in the centre of Mayfair connected together, some 35m below ground, by two 250m long platform tunnels (Figures 1 and 2). WSP was appointed to carry out the architectural and mechanical, electrical and public health (MEP) design through to RIBA Stage E[1], while developing the civil and structural design through to RIBA Stage F and construction status. As well as WSP, the design team included John McAslan + Partners as the architects and AECOM (then Scott Wilson) as the Category III checker, among others.

WSP was also the designer for the oversite development (OSD) and the adjacent site development (ASD) at the Eastern Ticket Hall. Here it was appointed under a separate contract with an independent design team to develop the design for the GHS Limited Partnership (GHS).

Due to the length of the project, the design has had to evolve to address the changing site and project conditions. Some eight years since the start of the commission, WSP is still employed as part of the site team to help deliver the construction.

Design approach

The design team of around 150 full-time staff developed the design through to the end of 2012. The majority of the team was co-located with Crossrail at its offices and was able to develop the design in a truly collaborative fashion. The results of the team's design development each week would be pinned up on the wall on Friday afternoons for a critique session. These sessions allowed the whole team to understand how each discipline was developing and to comment on the direction the design was taking. This helped build and develop the team and ensured that the solutions developed were shared by the whole team. This collaborative process ensured the development of a robust peer-reviewed design.

Following the delivery of the architectural, structural and MEP design through to RIBA Stage E, the WSP team was retained to develop the civil and structural design further. The civil and structural design needed to

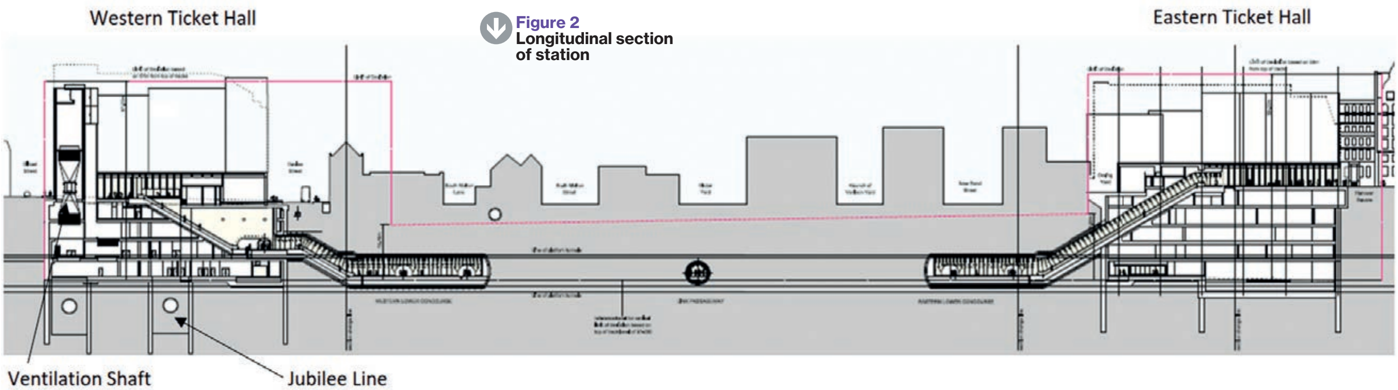


Figure 2
Longitudinal section of station

develop to RIBA Stage F and the production of construction information to allow early construction works to start on site. With the tunnel boring machines heading towards the station, the construction of the basement boxes needed to progress, incorporating early-access shafts for the tunnelling team as they came through the station.

Design constraints

The station design was split into three main sections: the Western Ticket Hall, the Eastern Ticket Hall and the platforms. The two ticket halls both had a number of similarities: each was five-and-a-half storeys below ground to a depth of around 35m, and each also had to incorporate a ventilation shaft to allow the tunnel ventilation system (required to vent smoke in the event of a train fire) to extract above the proposed OSD. This system also allows the venting of air due to the 'piston' effect of the trains passing through the tunnels. Each ticket hall also included a podium deck to the first floor to allow for the siting of a future OSD of up to eight storeys, while providing permanent access from Day 1.

The detailed design of the OSD was to be carried out by the design teams of the developers; however, the programme for delivery of the OSD was considerably behind that of the station design programme. This was to be expected due to the long construction programme required to deliver the stations on site. In order to allow the station design to progress and the OSD to be space-proofed, the station design team developed a scheme design for both OSD sites. This design ensured the correct space provision for access and egress, welfare facilities, service routing, etc. to support the future operation of the OSD.

Western Ticket Hall

Each of the ticket halls had its own specific design constraints which needed to be incorporated into their design. The Western Ticket Hall was constructed within a predominantly residential area and was constructed directly over the running tunnels of the Jubilee line (Fig. 2). It was constructed adjacent to a number of listed structures, each of which had very tight movement trigger levels placed on them as part of the undertakings and assurances process applied at the hybrid bill phase[2] of the Crossrail project. This limited the movement of some of the listed structures to 1–2mm when elsewhere this could have been in the order of 5–10mm for a similar structure.

In order to satisfy these requirements, a diaphragm wall construction was proposed for the external wall, to reduce the vibration arising from the installation. This approach was also possible due to the orthogonal nature of the ticket hall plan, aligned to the panel size of the diaphragm wall machine. This, supplemented with a compensation grouting system, allowed the project to meet the tight movement tolerances required here, for all stages of the construction works.

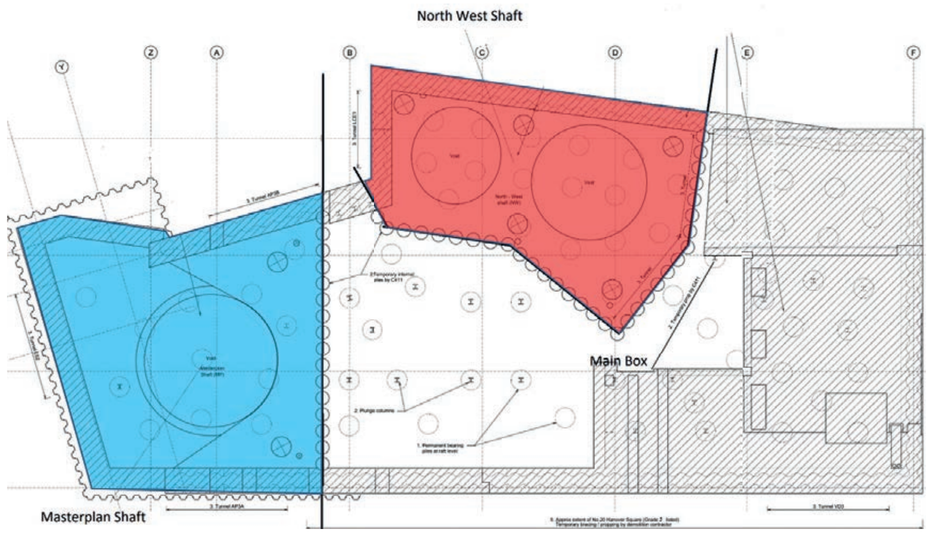


Figure 3
Plan of Eastern Ticket Hall

Eastern Ticket Hall

The Eastern Ticket Hall is surrounded by an area of land owned by GHS which the company was in the process of redeveloping. Together with the development of the OSD to the Eastern Ticket Hall, this would complete a significant regeneration of the area. Two factors reduced the need to limit construction vibration, compared to the Western Ticket Hall, and allowed the design to employ a secant piled wall: i) the proposed development provided a sufficient stand-off distance to the residential properties in the area, and ii) the majority of the listed structures in the zone of influence were owned by GHS and formed part of the proposed development. The more flexible secant piled wall helped with the construction of the less regular perimeter of the basement to the station (Figure 3).

The ASD, which was part of the development by GHS, applied its own constraints on the design of the station basement box. The site which was to be developed consisted of existing masonry buildings of around five storeys in height; these were to be demolished to ground level, while the facades on New Bond Street were retained with significant temporary works. New basements were to be excavated adjacent to the external retaining wall of the station basement, followed by the construction of the new steel-framed and reinforced concrete buildings to form the final development.

The design of the station structure accounted for the top-down construction of two temporary shafts – the North West Shaft and the Masterplan Shaft – followed by the top-down construction of the rest of the station basement. The station also needed to be designed to account for the staged construction of the ASD. As the ASD was being developed to a separate programme, which Crossrail did not want to constrain, the station was designed to accommodate the staged construction of the ASD at any time during the construction of the station.

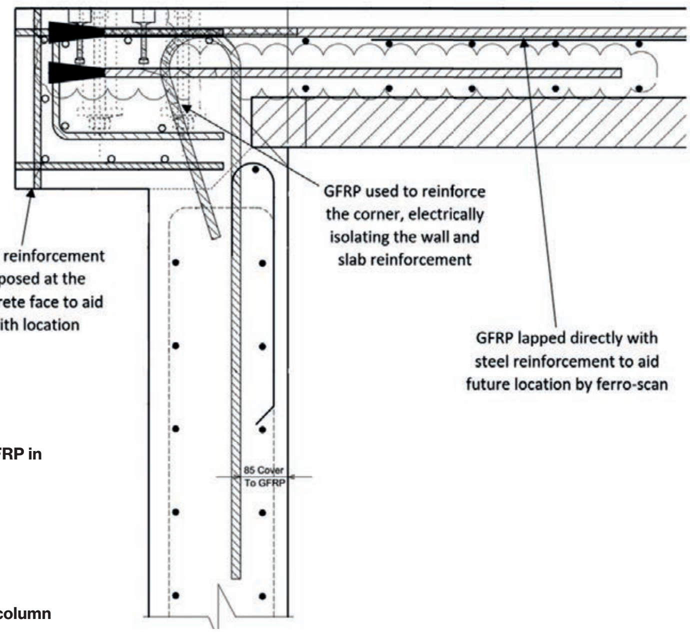


Figure 4
Setting-out of GFRP in platform nosings

Figure 5
Collar to plunge column

Platforms

The platforms had fewer constraints due to the existing site conditions; however, they needed to be designed to accommodate a number of project constraints, such as high point loading to allow for the replacement of large pieces of plant via engineering trains and the use of glass fibre-reinforced polymer (GFRP) reinforcement to the platform nosing to ensure electrical separation from the track and the station earthing systems.

While the design of this was straightforward, utilising design guidance from the *fib* Model Code[3], the detailing of this section needed a lot of attention. Throughout the station the structure was designed for a 120-year design life. Over such a long period, it is reasonable to assume that other elements, such as the platform edge screens, would need replacement and that post-drilling into the platform would be required. The GFRP was therefore detailed in such a way that it could be located on site, even though it would not be picked up in a normal scan for ferrous reinforcement. As set out in Figure 4, sections of GFRP were placed to be exposed on the surface and aligned with the steel reinforcement further back in the span of the slabs.

Design assurance

The design went through a vigorous verification process. Internally, it was regularly peer-reviewed and underwent Category I self-checking within the design team, and Category II checks by an independent WSP office. The design then went through an external Category III check by Scott Wilson (AECOM). This process ensured that the design was independently scrutinised and introduced a number of changes, predominantly in the assessment of the geotechnical parameters and their impact on the loading into the temporary works.

In addition to the verification works carried out by the design team, the design needed to pass through a staged-gate process with Crossrail. This required the provision of evidence to demonstrate that the design had been assured in line with Crossrail's procedures and coordinated with the other disciplines and at design/construction interfaces.

This high level of assurance continued through to the execution of the design on site. While the design-and-build contractor was appointed to be fully self-assured, Crossrail retained a site presence and assurance role with several full-time field engineers responsible for the sign-off elements of the works before they proceeded. This enhanced the quality control of the works constructed and reduced the number of non-conformances that occurred on site.

Independent design teams

Subsequent to its appointment as the framework design consultant for the station development at Bond Street, WSP was appointed to design the OSD and ASD for GHS adjacent to and above the Eastern Ticket Hall. To enable WSP to deliver these works for two clients, two independent design teams were set up. As the majority of the design work for the Eastern Ticket Hall had recently been completed, it was possible to transfer across a number of key designers from the Eastern Ticket Hall team, with good knowledge of how the station design had been developed, to lead the development of the OSD and ASD design. This team was kept independent of the station design team, who were finalising the design and supporting the delivery of the project on site. This independence was important to ensure that no conflict of interest arose between the design teams.

The OSD and ASD design team was able to develop the design for GHS in a sympathetic fashion to the station design. This approach ensured that, while changes were proposed by the OSD to the station design, these changes were minimised and had already been assessed to enhance the likelihood of their acceptability to the station design. The OSD and ASD design was developed and adjusted alongside the finalisation of the station design to allow both schemes to be coordinated. This coordinated design was referred to as the masterplan scheme and provided benefits to both schemes, while adhering to the independence and differing assurance schemes required by both of the clients.

Coordinating design changes

Two major changes to the schemes went through the change control process: the relocation of the ventilation shafts and the revision of the OSD column grids.

Relocation of ventilation shafts

The original design for the station placed the vent within the main station and OSD footprint and acted to reduce its lettable area and efficiency. The ventilation shafts are also a source of noise and vibration, which can require significant mitigation to achieve suitable commercial space. To mitigate this, within the Western Ticket Hall the OSD design allows for isolated connections between the main frame and the ventilation shaft to overcome the noise and vibration transferring across to the main frame.

In the Eastern Ticket Hall, however, the design teams were able to work together with GHS to relocate these shafts from the OSD across to the ASD and to incorporate them within the less critical areas of the ASD accommodation, to mitigate issues with the transfer of noise and vibration. Moving the shafts across to the ASD section also increased the area available within the station basement box to locate one of the early-access shafts for the tunnelling contractor. This shaft was subsequently renamed the Masterplan Shaft to reflect this. The final location is shown in Fig. 3.

The change in the position of the ventilation shafts had significant benefits for both clients. For the OSD, it increased the lettable area of the development and removed the risk that an area of this would be affected by noise and vibration from the ventilation shafts. For the station, it allowed the positioning of the shafts in an area which caused less impact to the development of the station construction works.

The relocation did mean that large sections of each of the designs had to be reworked. For the OSD, the design team had to rearrange the accommodation in the ASD to allow for the positioning of the ventilation shafts, ensuring that the accommodation layout was positioned to suit the adjacency with the ventilation shafts.

For the station, the routing of the extracted air from the tunnel ventilation system needed to be revised to suit the position of the shafts. In this instance, the tunnel ventilation fans were relocated from a vertical position within a vertical ventilation shaft, to be positioned horizontally within the basement box. The two tunnel ventilation fans were positioned above each other on different floors and ventilation routes were spread through the station incorporating sloping sections of slab to transition through the levels.

Revised column grid

The second major change was to the column grid for the OSD. As described above, the station was designed to accommodate an OSD which had been schemed by the station design team. This original design had allowed for the positioning of columns on a 9m grid from the podium level upwards. As the design for the OSD developed, it was requested that the columns move to a 3m grid around the perimeter of the building.

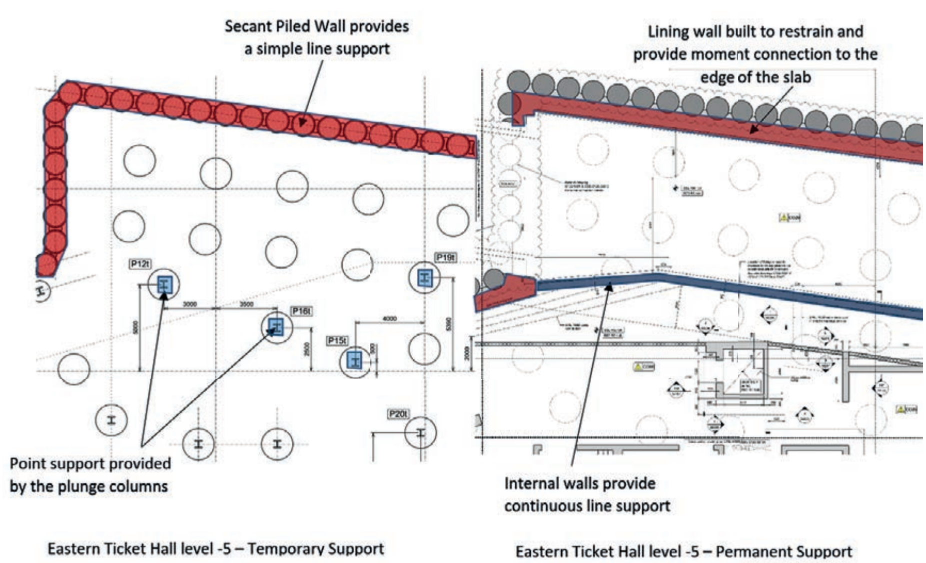
Working independently, the WSP team employed by GHS developed a revised design for the structure above the station podium deck. This design incorporated the revised external appearance of the OSD and allowed the transfer of load such that the load distribution and total load remained similar to that considered in the original OSD scheme design. This careful consideration reduced the amount of redesign required by the station design team.

The station redesign had to validate that the total load and the load paths remained largely unchanged and that there was no effect on the below-ground structure which had already been built. The work by the OSD design team meant that the review required to confirm this was limited. The changes were limited to the edge beams of the first-floor podium deck, which needed to be redesigned to pick up the intermediate point loads from the 3m column grid and to transfer these back to the main column grid at 9m centres.

The design of the reinforced concrete elements within the station was limited to a 0.3mm crack width to satisfy the Crossrail Civil Engineering Design Standards[4]. Two limits were applied for crack control across the project: 0.2mm for water-retaining structures and 0.3mm elsewhere to ensure the quality of the appearance. This design case governed for the design and detailing of these elements, while the ultimate limit state design was not significantly affected.

The revised design was then re-assured through the same rigorous process as the original design. Internal checks were carried out on the revised design prior to this being sent out for a revision of the independent external Category III design and recertification. Once the design was acceptable to both the WSP and AECOM design teams, the changes were presented back to Crossrail under a 'gate impact report'. This impact report was produced to demonstrate that the revised design had undergone the same level of verification as the previous design and that the revisions did not impact on the coordination with any of the other disciplines, which had been demonstrated to Crossrail through the staged-gate review process.

Figure 6
Temporary and permanent connection to secant piled wall



Design to construct

While every structure needs to be designed with construction in mind, the construction of this station in the centre of Mayfair required a thorough understanding of the processes that could be utilised. Construction advisers were an integral part of the design team and were able to deliver a number of solutions to achieve this. These varied from the sequential installation of the precast concrete coffer units for the first floor of the two ticket halls, to achieve the ±3mm architectural tolerance, through to the performance specification of a propping system to be installed in place of the permanent slabs where these sloped between floors.

The station boxes had been designed to be built on constrained sites and in a top-down construction sequence. This sequence required the ground-floor slab to be designed for a number of design loading conditions as a construction deck: firstly, as a staging ground for the piling and diaphragm wall works, followed by the staging for the excavation of the basement box in a top-down sequence. For the Eastern Ticket Hall this was also staged into three elements: the North West Shaft, the Masterplan Shaft and the main box.

To construct the basement boxes in a top-down fashion, the main elements needed to be designed to work in a number of temporary conditions. The Eastern Ticket Hall box was designed to be excavated in sections and supported in a temporary condition with the early installation of the Masterplan Shaft, followed by the early installation of the North West Shaft which was infilled with the permanent structure at the same time as the top-down construction of the main box. The basement box would then be finished with the top-down permanent slab construction in the Masterplan Shaft. These sequences were considered in the development of the original design and were incorporated in the design of the structure.

Top-down construction

The top-down construction was enabled by the installation of plunge columns. These were installed with the piling from ground level: when the piles were concreted to the underside of the lowest-level slab, the plunge columns were lowered through the empty core and founded into the concrete, with regular spacers around the columns to ensure their verticality. The core was backfilled around the plunge columns to ensure that they were stable in the temporary condition, as they were then re-exposed during the top-down excavation works.

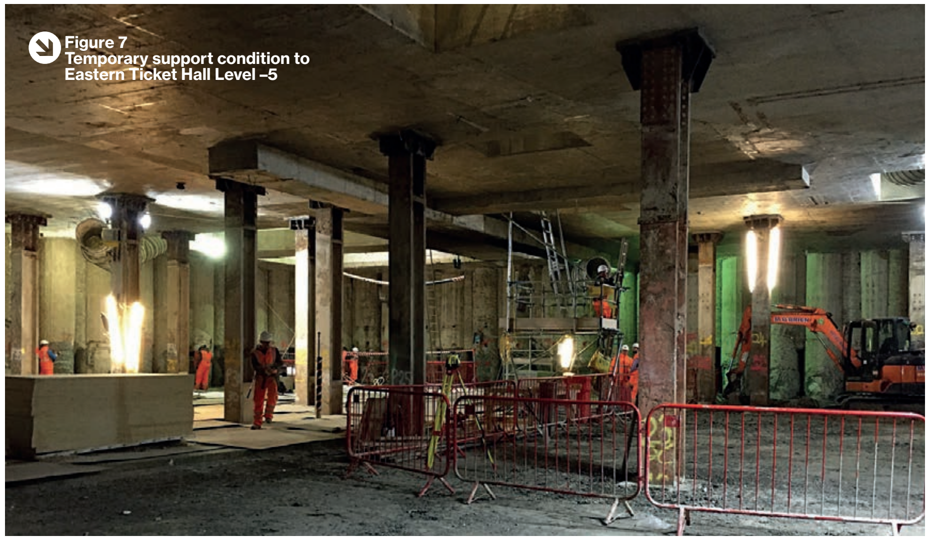


Figure 7
Temporary support condition to Eastern Ticket Hall Level –5

The permanent slabs were constructed on the excavated base at each stage of the excavation works. Collars were installed onto the plunge columns (Figure 5) at each of the floor levels to reduce the impact of punching shear effects at the connections between the floor slabs and the columns. This provided point supports to these slabs internally. The slabs were connected to the external secant piled wall, connected into couplers installed within the piles as they were originally cast. These connections provided a simple support condition to the edge of these slabs. In the temporary condition, a series of mole holes was provided within the slab to allow for construction access through the station box from ground level.

Once the excavation had been completed to the lowest level, it was possible to start the construction of the permanent vertical loadbearing structure in a bottom-up sequence. The permanent loadbearing structure comprised an inner skin and lining

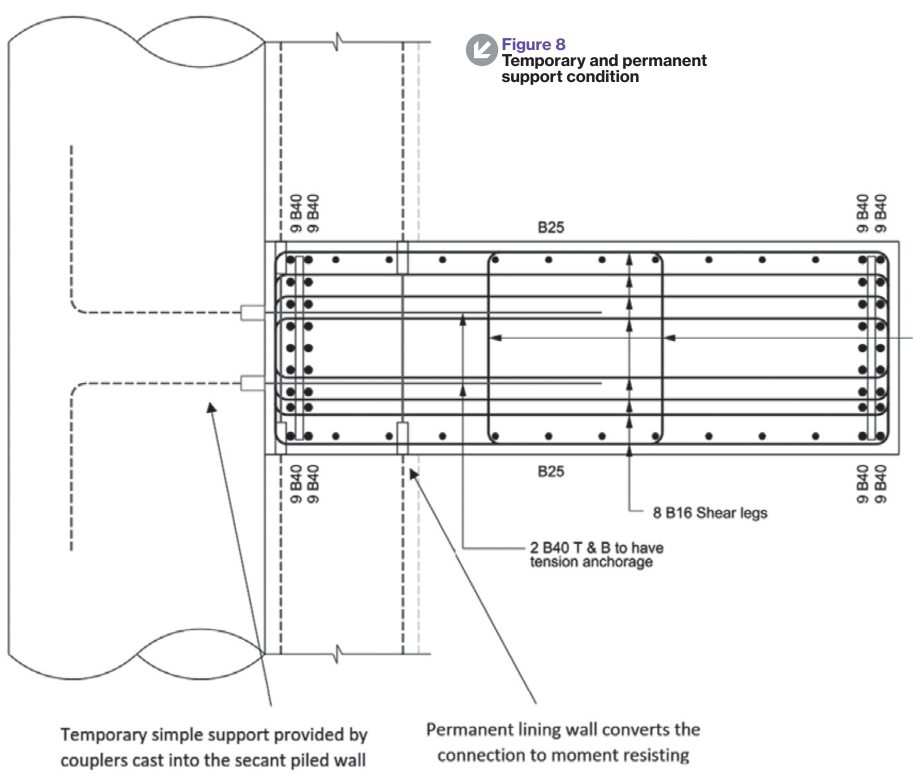


Figure 8
Temporary and permanent support condition

wall to the external piled walls; the lining wall was approx. 700mm thick, plus an allowance for tolerance, and was detailed to be continuous through the structural slabs, which had already been cast, via coupled bars through the slab, installed as part of the top-down sequence.

Internally the structure was supported by loadbearing walls generally, with a small number of columns. This permanent condition changed the support to the structural slabs from localised point supports internally and a simple support to the edge of the slab, through to a series of internal line supports onto reinforced concrete walls and an external moment connection into the lining wall (Figures 6 and 7). This increase in the support condition in the permanent case allowed an increase in the load capacity of the intermediate slabs from around 5kPa in the temporary case to a total of 15–20kPa in the permanent case, as a combination of superimposed dead load and live loading (Figure 8).

Once the permanent vertical loadbearing structure was in place, it was possible to remove the plunge columns. The post-installed collars to the plunge columns were removed and the localised grout packing around the columns at the floor levels was broken out. This allowed the plunge columns to be extracted vertically through the slabs to ground level.


Figure 9 Propping to waling beams on secant piled wall

Temporary works and monitoring

Significant temporary works were required to facilitate the top-down construction sequence. Typically, these employed sections of the permanent works acting in a temporary condition, but this wasn't always possible. In such locations, waling beams were designed into the lining wall construction to allow the secant wall to be propped back to the main structure, typically at grid lines. These props were designed to support loads of up to 11 000kN, as an output of the analysis of the soil–structure interaction (Figure 9).

This interaction was subject to a degree of assumption in the development of the design and the accuracy of this was integral to the stability of the basement box in the temporary condition. As such, it was important to confirm that the assumptions included within the design were correct, or conservative. In order to achieve this, a monitoring regime was specified and installed within the embedded retaining walls to record the actual movements of the structure in operation. This consisted of a series of cast-in inclinometers and discrete monitoring targets, combined with trigger levels set at amber, red and black levels. A breach at any of these levels would trigger progressively more stringent limits on the progression of works, combined with additional monitoring requirements to ensure the safety of the structure.

Monitoring periods were set weekly, increasing in frequency during periods of excavation or de-propping where movement was expected. An example of the output of this monitoring is shown in Figure 10; the amber trigger levels were not breached.

Site support

The appointment as framework design consultant included provision of engineering support on site during the construction phase of the project. WSP provided a site team of up to 15 engineers and CAD technicians to fulfil this role. This team primarily responded to queries raised by the construction team and by the contractor's architectural and MEP designers who were developing their detailed design. This support role involved answering technical queries, as well as revising the design and construction details of the structure to accommodate the changing architectural and MEP design and the preferred construction methods of the contractor. The majority of the design changes involved minor changes to the structure, incorporating builders' work openings and changes to upstands to suit the development of the services distribution and the clarification of cladding details.

A number of more significant changes arose during the construction phase. The most significant of these was the revision of the construction sequence to the Masterplan Shaft. While the permanent slabs in the area of the other early-access shaft, the North West Shaft, were constructed early during the top-down construction of the main box, the Masterplan Shaft was left open to facilitate easy access to the platform level for the main contractor and the other system-wide contractors so that they could complete the platform and track works. This placed the infilling of the Masterplan Shaft with the permanent structure onto the critical path towards the end of the project.

To support the construction of the rooms and service routes to the lower levels of the basement box first, a revised sequence was proposed to infill the permanent structure to the Masterplan Shaft in a bottom-up sequence. The revised sequence imposed a significantly different construction sequence onto the external piled wall within the Masterplan Shaft corner. Originally the permanent slabs would have been installed prior to the demolition of the first of the temporary slabs, serving to enhance the support to the external wall as the works progressed. With the bottom-up sequence, the temporary slabs needed to be demolished prior to the installation of the permanent slabs to ensure that there was a suitable route for the removal of the demolition arisings. To enable this alternative support system to work sufficiently with the existing, as-installed, piled retaining wall, a system of additional temporary works was required to support the piled wall and to ensure that it did not deflect during the infilling of the Masterplan Shaft.

Designed for an evolving design

The architectural and MEP design of the station has been progressing several years behind the civil and structural design. This has meant that key interfaces between the disciplines, such as builders' work openings and secondary support structure for the cladding, have been finalised after much of the structure has been constructed. This was allowed for in the original design by ensuring that there was some scope for later change.

Instead of a 0.99 utilisation ratio, the design was typically carried out with a utilisation ratio of 0.9–0.95 to balance the need for an efficient design with scope for future flexibility. This allowed new builders' work openings to be introduced in most of the areas in which they were requested. Similarly, the partition allowances that the team included as part of the superimposed dead load allowance were sufficient to allow medium-dense blockwork walls to be adjusted to *in situ* reinforced concrete walls, to allow for the omission of a secondary steelwork sub-frame to the cladding.

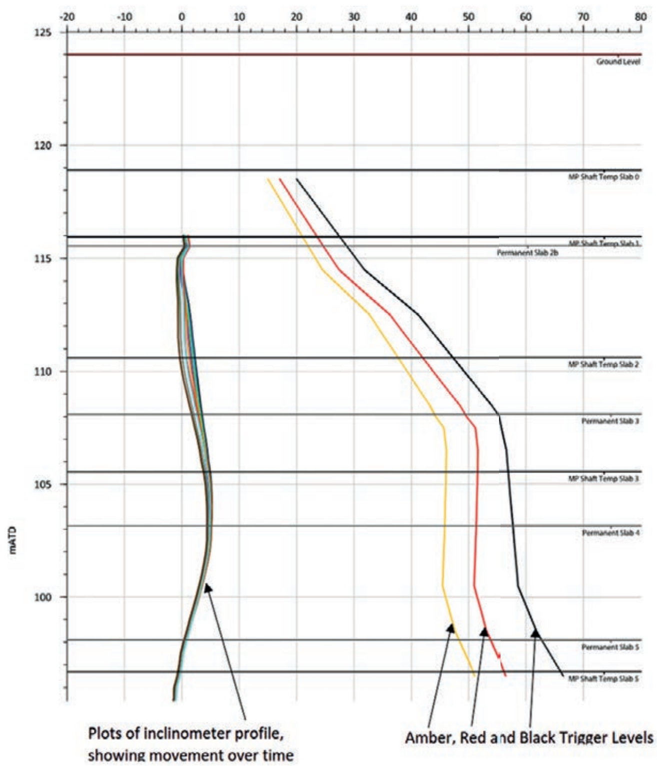
As part of the role on site, WSP has worked closely with Crossrail, the client, and with Costain Skanska Joint Venture, the contractor. The parties collaborated to ensure that the works on site progressed to programme while ensuring the quality of the work would not be affected. Part of this work included ensuring that the site team knew the importance of the constraints placed on the construction sequence. This was achieve through regular meetings with the contractor's engineering and temporary works teams, and lunchtime sessions where a joint team presented the importance of the temporary works systems to the contractor's office and site teams. This briefing programme was coordinated to align with the works on site to keep it relevant.

Summary of key points

A number of lessons were learned by the design team and have been captured by Crossrail to be taken forward to other projects as a learning legacy. Those that particularly stand out on this project are presented below:

- **Design to construct** – everything needs to be built and it is a designer's responsibility to ensure that it is possible to build safely what has been designed. At Bond Street this included the incorporation of elements of temporary works within the permanent structure of the station, reducing the need to install and remove temporary works.
- **Design for change** – with a project lifecycle of more than 10 years it is inevitable that there will be change. It is also unlikely that all the changes over this timespan could be predicted at the outset of the project. Given this and the difficulty in altering a deep basement, designing in spare capacity as part of the structural design was considered more sustainable and cost-effective than designing to the limit and having to rebuild elements later.
- **Focus on interfaces** – interfaces are the locations where misunderstanding or scope gaps are likely to arise. Ensuring that these are agreed and developed in parallel between the design parties across the interface, as early as possible, will save the designers and contractors time and effort further into the project. Across the Crossrail project, interface control documents were used to document the agreements and were kept as live documents that were updated as the design and construction progressed.

Figure 10
Typical plot of movement against trigger levels



Project team

Framework design consultant: WSP
Client (station): Crossrail Ltd
Client (OSD and ASD): GHS Limited Partnership
Architect: John McAslan + Partners
Category III checker: Scott Wilson (AECOM)
Contractor: Costain Skanska Joint Venture

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►3) *fib* (2013) *fib Model Code for Concrete Structures 2010*, Berlin: Ernst & Sohn

►4) Crossrail Ltd (2009) *Civil Engineering Design Standards*

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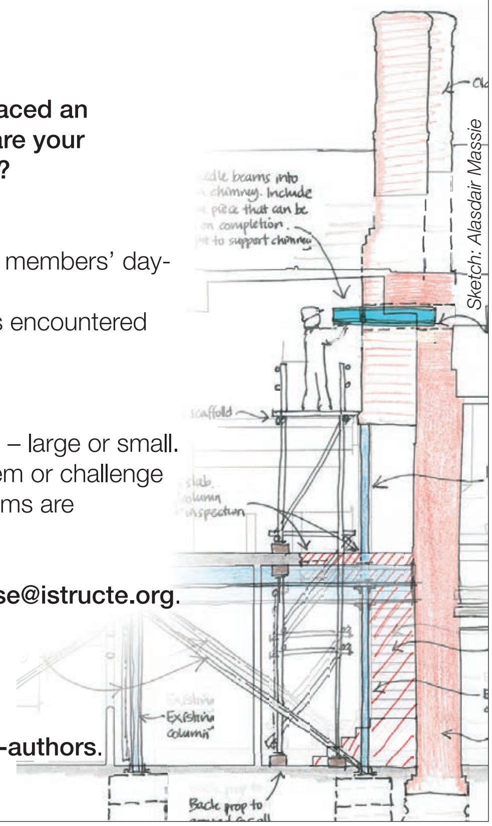

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Civil and structural engineering design for the Elizabeth line station at Tottenham Court Road

Jonathan Plant
BEng, CEng, MIStructE
Senior Engineer, Arup, UK

Stephen Frost
BSc, Eur Ing, FICE, CEng
Director, Geada, UK

Steve Roberts
BEng, CEng, MIStructE
Regional Director, AECOM, UK

Synopsis

The new Elizabeth line station at Tottenham Court Road, delivered by the Crossrail programme, has been an exercise in interface management as well as a feat of engineering.

This paper describes the design carried out by the Arup Atkins Joint Venture (AAJV) under contract C134, principally of the Western Ticket Hall box. Nestled in Soho, this was developed within a dense urban grid and the constraints of a residential oversite development above.

The team worked closely with London Underground Ltd's engineers at the Eastern Entrance, which was delivered as part of London Underground's own station upgrade works.

The tunnel for the eastbound Elizabeth line passes through the Western Ticket Hall box, which also provided construction access for the sprayed concrete-lined platform and concourse tunnels. Access dates to the site meant that there was insufficient time to complete construction of the box before the arrival of the tunnel boring machine (TBM). Consequently, the need to complete the excavation became critical and the team adopted a bottom-up construction sequence for one of the deepest open shafts ever excavated in central London. The box, formed of elements of diaphragm walls and raft, was constructed before the TBM arrived, and the remaining internal elements completed afterwards.

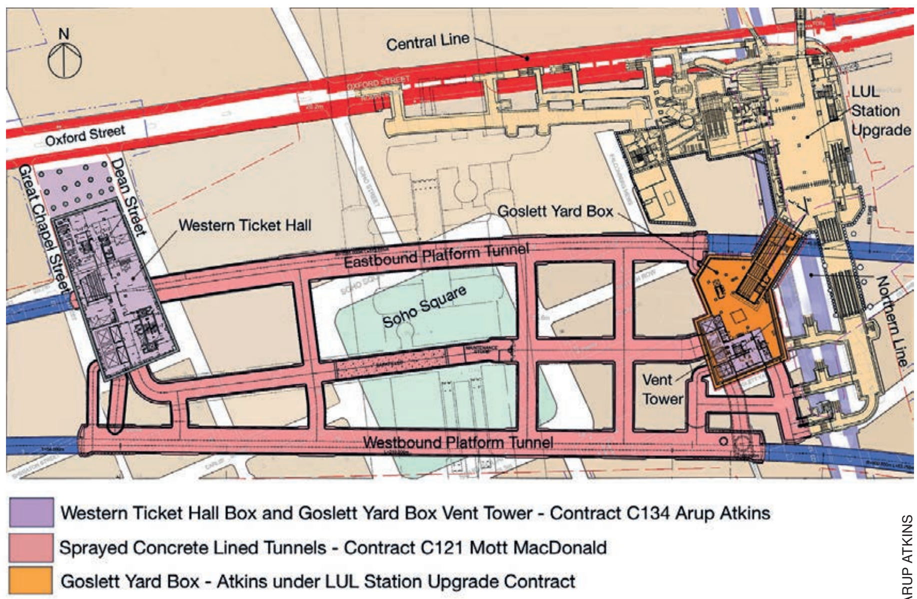

Figure 1
Tottenham Court Road station plan showing division of design contracts

NOTATION	
AAJV	**Arup Atkins Joint Venture**
MEP	**mechanical, electrical and public health**
OSD	**oversite development**
TBM	**tunnel boring machine**

Introduction

Tottenham Court Road Elizabeth line station consists of two entrances housed in box structures at either end of the platforms, both formed within diaphragm walls. These house access and circulation spaces, as well as tunnel ventilation systems and mechanical and electrical services. Between the two boxes, sprayed concrete-lined tunnels house the platforms and concourses, with sub-platform service connections to the boxes.

The structural engineering design of the station was carried out under three separate contracts (Figure 1):

- The below-ground shell of the eastern end of the station, known as the Goslett Yard Box, which connects into the new ticket hall of the London Underground station, was designed and built under London Underground's Tottenham Court Road Upgrade contract, with the design carried out by Atkins.
- The sprayed concrete-lined platform and concourse tunnels were designed under Crossrail Contract C121 by Mott MacDonald.
- The Western Ticket Hall box, ground level and five-storey ventilation tower structures, at the west end of the station, were designed under Crossrail Contract C134 by Arup Atkins Joint Venture (AAJV). This contract also included the internal structures in the platforms and concourse

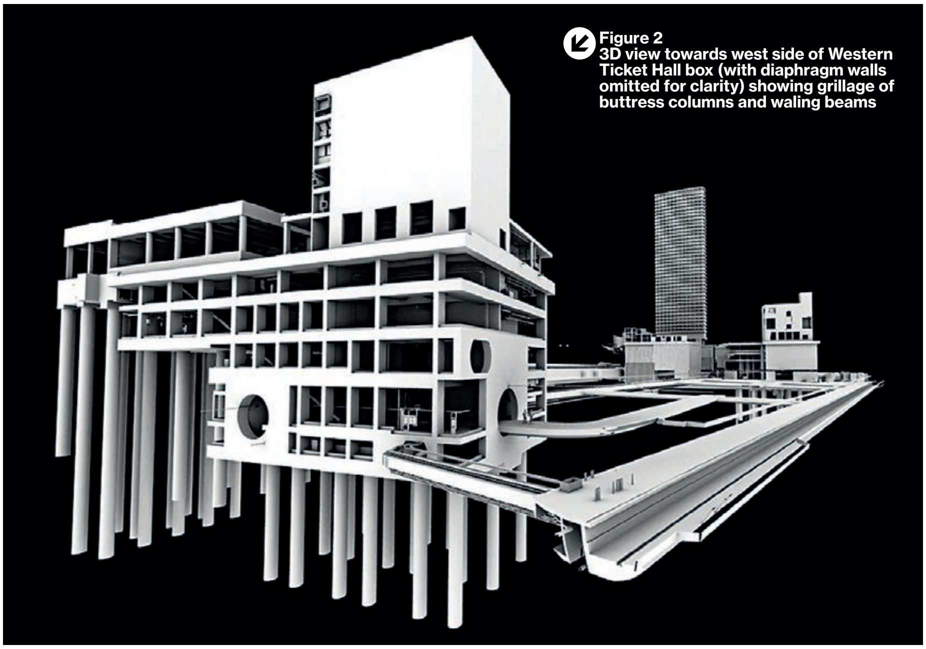
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Figure 2
3D view towards west side of Western Ticket Hall box (with diaphragm walls omitted for clarity) showing grillage of buttress columns and waling beams

tunnels, the internal fit-out structures in the Goslett Yard Box and a five-storey reinforced concrete tower above the Goslett Yard Box housing ventilation and other plant.

Although the structural design responsibilities were split, the overall responsibility for planning, architecture, building services, specialists and constructability for the whole station resided with AAJV, with Hawkins\Brown as architects, under the C134 contract.

Structural arrangement of the Western Ticket Hall box

The Western Ticket Hall site is located on the south side of Oxford Street between Dean Street and Great Chapel Street (Fig. 1). It is approx. 80m × 30m in plan and is split into two blocks by Fareham Street. The north block contains the ticket hall and station entrance at ground level, as well as retail units fronting onto Oxford Street. The ground level of the south block contains an electrical substation and an emergency escape and intervention access. A six-storey tower housing the tunnel ventilation systems occupies part of the site, and the remainder of the south block is given over to retail units.

Above both blocks, provision has been made for future six-storey residential oversite developments (OSDs). The ventilation tower acts as the stability core for the south block of the OSD. A stability core and shear wall provided in the north block are designed to resist lateral loads from both the ticket hall and future OSD. The ticket hall itself features a single central column supporting a grillage of post-tensioned roof beams. These beams support column loads from the OSD and form a primary visual feature of the entrance.

"THE RESULTING GRILLAGE IS STIFF ENOUGH TO FRAME THE ESCALATOR VOIDS WITHOUT THE NEED FOR FLYING PROPS"

Below ground, the site consists of three sections. Adjacent to Oxford Street there is a single-level basement formed within cast *in situ* retaining walls which will house retail units. To the south of this are two sections of diaphragm walled box, 12m and 28m in depth, which house the station. The shallower 12m deep box contains two levels of basement and is located below the ticket hall. The deep box occupies the south end of the site and contains five levels of basement.

The platforms are reached directly from ground level by a dramatic single bank of three escalators. The eastbound line passes through the deep box, allowing direct access to the eastbound platform from a lower concourse at Level –4. Access to the westbound platform is via a 7m diameter concourse tunnel. Draft relief and service tunnels connect from the westbound tunnel into the deep box. There are four cores containing stairs, lifts, building services and ventilation risers.

The box was formed using diaphragm wall panels with a 1.8m deep piled raft in both the shallow and deep sections. There are five levels of 700mm thick propping slabs (including the ground floor). Vertical support is provided by a mixture of columns (including composite concrete-encased steel sections) and core walls. The propping slabs are flat, except at Level –3 where a grillage of drop beams is provided to form the roof of the lower concourse area, mirroring the beams in the roof of the ticket hall.

There are a number of transfer structures (described further below) formed of reinforced concrete deep beams and storey-high concrete-encased steel trusses. Buttress columns around the perimeter of the box provide a direct load path for the perimeter columns of the station above. They are also tied to waling beams that frame the large voids around the escalators and the tunnel ventilation ducts. The western diaphragm wall was positioned as far west towards Great Chapel Street as possible in order to accommodate the waling beams and buttresses sized for this purpose. The resulting grillage is stiff enough to frame the escalator voids without the need for flying props (Figure 2).

The diaphragm walls of the deep box are penetrated at three levels by tunnel openings. At Level –3, an opening was formed on the south wall to connect the draft relief shaft from the westbound running tunnel. At Level –4, there are openings for the concourse tunnel on the south wall, and the eastbound running and platform tunnels on the west and east walls respectively. At Level –5, there is an opening for a service tunnel connection to the westbound running tunnel. At each level, 1200mm thick reinforced concrete lining walls were provided around the openings. The purpose of the lining walls is to transfer vertical loads around the openings. The concourse tunnel opening to the south wall of the box is located immediately below a deep beam supporting the ventilation tower. A heavily reinforced beam strip was provided within the lining wall above the opening to deal with this. Similarly, the opening for the eastbound platform tunnel is located below one of the buttress columns.

The lining walls also act to tie the diaphragm wall panels together where they are cut by the tunnel openings, and contain the termination of the waterproof layer of the sprayed concrete tunnel lining where it extends through the openings into the box. This is described in more detail further on.

Storey-high transfer trusses are provided in three locations. A pair of trusses transfer column loads from the south block across the eastbound platform (Figures 3 and 4). These are located at Level –3. They are formed from 356mm × 406mm × 634mm universal columns. The diagonal members are encased in 850mm × 850mm reinforced concrete sections, while the bottom chords sit within the 1250mm × 1700mm deep drop beams. A third transfer truss (Figure 5) is provided at Level –1 below the highly loaded central column of the ticket hall (Figure 6) in order to spread the load into the piled raft foundation.

The walls of the ventilation tower act as deep beams below ground level to transfer the weight of the tower onto a line of columns at lower concourse level.

A bank of three escalators connects the ticket hall at ground level directly with the platform concourse at Level –4, a level difference of approx. 23.5m, with space provided for a fourth escalator to be installed in the future if required. The escalators are located above a 450mm thick

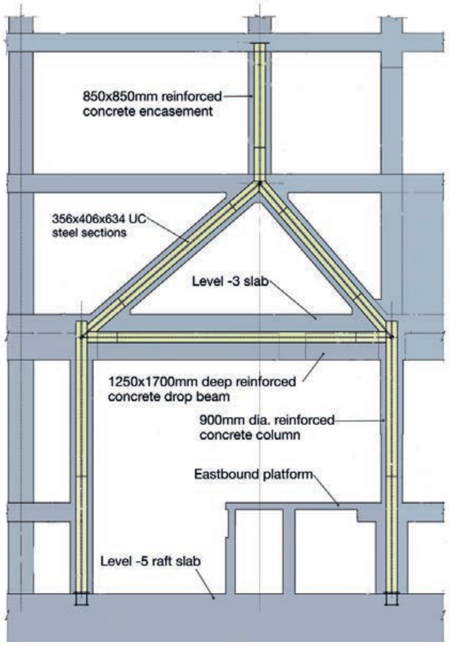

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Figure 3
Concrete-encased steel transfer truss above eastbound platform


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Figure 4
Installation of steel transfer trusses above eastbound platform

inclined slab (Figure 7) which spans between the propping slabs, with the intersections between the inclined slab and the propping slab reinforced as beam strips. The supports for the escalator trusses are located close to these beam strips and plinths are provided on the inclined slab at the support positions. The escalators are designed to be maintained from above, with only a limited clearance between the bottom of the truss and the inclined slab.

The single-level basement for the retail units on the north side of the shallow box has a 350mm thick base slab supported by piles and an arrangement of pile caps and ground beams.

Water tightness throughout the station boxes and the basement to the retail units is ensured by drained cavities in front of the perimeter walls and by an 'egg crate' over-slab drainage system.

Structural arrangement of ticket hall roof

The ticket hall roof (Figure 8) is formed from eight rows of 1000mm × 1550mm deep rib beams, running east–west at 3.75m centres, spanning between edge beams and a 1500mm × 1550mm deep central spine beam and a core wall. The edge beams are supported by perimeter columns at 7.5m centres. Two rows of 750mm × 750mm deep spreader beams run north–south at the mid-span of the rib beams. The roof is required to act as a transfer structure supporting column loads from a future six-storey concrete-framed residential OSD. It was decided at an early stage of the design that the beams forming the roof should be post-tensioned. This was to minimise the size of the beams, which are a key architectural feature of the station entrance, and to control flexural cracking as they are loaded by the OSD.

The design life of the station structure is 120 years, whereas that of the OSD is 50 years. The design of the roof had to allow for the OSD to be demolished in the future. As a result, it was not possible to use a staged stressing of the tendons during construction of the OSD. The design is therefore less efficient than it could otherwise have been. The high compressive

Figure 5
Concrete-encased steel transfer structure supporting main column to ticket hall roof

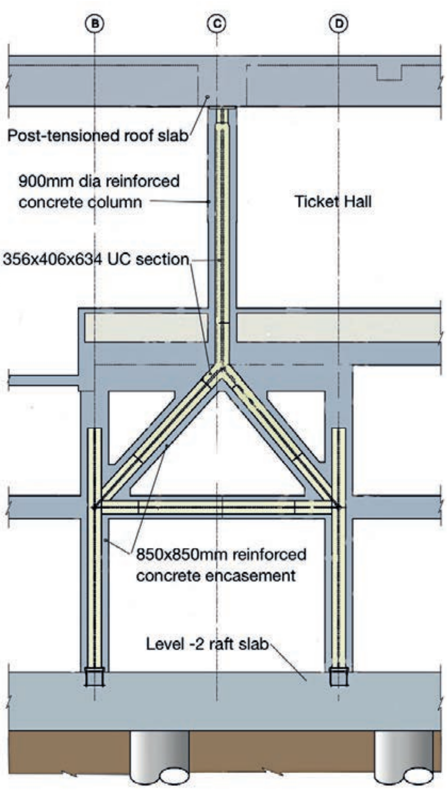

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Figure 6
Central column to ticket hall roof prior to encasement

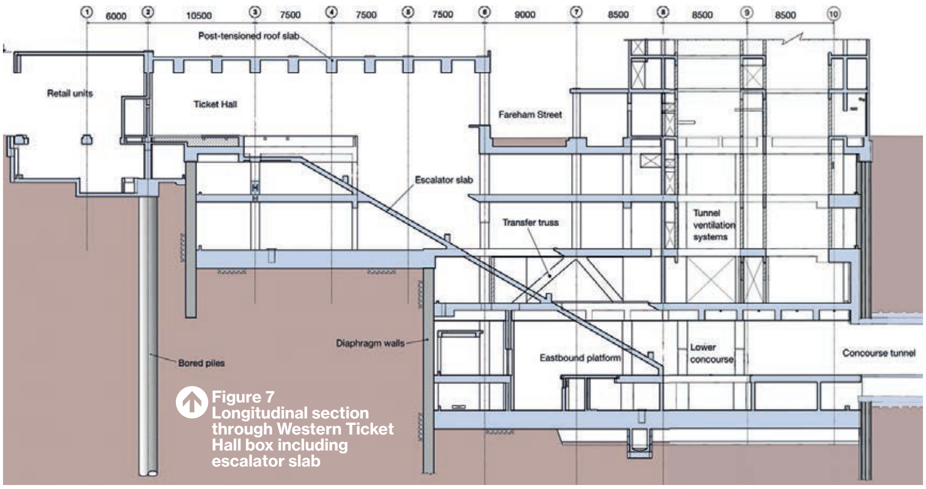

Figure 7
Longitudinal section through Western Ticket Hall box including escalator slab

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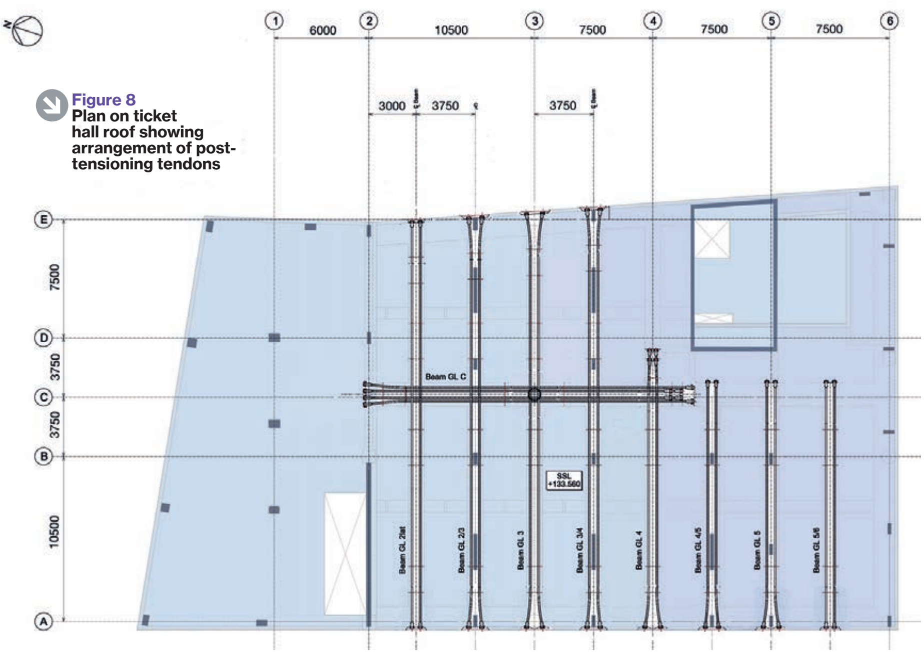

Figure 8
Plan on ticket hall roof showing arrangement of post-tensioning tendons

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stresses resulting from the post-tensioning required a considerable density of bursting steel around the end blocks (Figure 9) and the use of a relatively high-strength C50/60 concrete mix with a 10mm aggregate.

The OSD columns will be located on four of the eight rib beams. Consequently, the arrangement of the ducts varies between beams. The rib beams have arrangements of either four or six ducts with 19 strands or 22 strands per tendon. The central spine beam has eight ducts with 22 strands per tendon (Figures 10 and 11).

The roof slab was cast over a period of approx. eight weeks in the summer of 2016. The stressing of the first tendons (Figure 12) started within a week of the last section of slab being cast, by which time the concrete in the last pour had achieved the required minimum cylinder strength of 35N/mm^2.

The OSD loading for which the roof slab was designed was based on a scheme design developed in 2010. However, during the construction phase the OSD's interior planning was updated to increase its commercial potential, in particular the entrance areas. In the meantime, planned acoustic testing of the completed raft slabs showed that sound and vibration levels transmitted to the OSD from the adjacent Central line (London Underground) might be unacceptable. It was therefore necessary to make provision for acoustic isolation

Figure 9
View of end block to post-tensioned spine beam

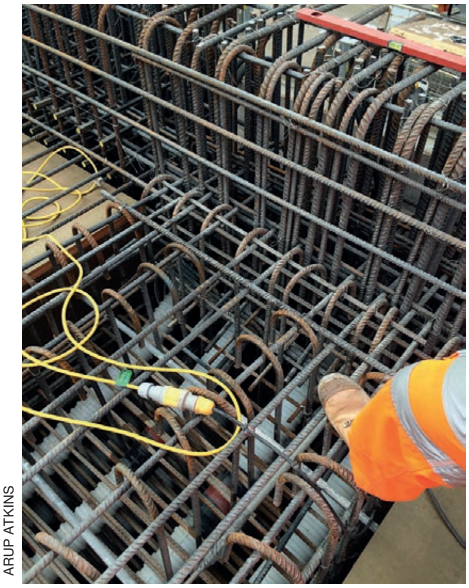

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Figure 10
Section through post-tensioned spine beam showing arrangement of tendons

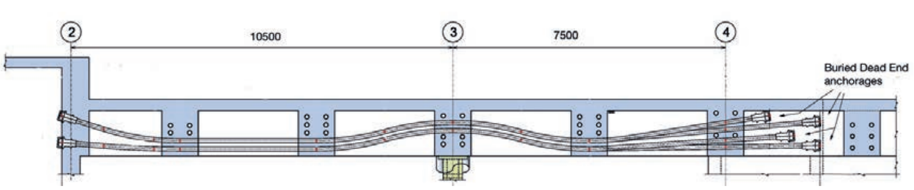

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Figure 11
View of post-tensioned roof beams during installation of tendons

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bearings between the station roof and the OSD. Both these events resulted in design changes to the arrangement of the roof, including the addition of 100mm high plinths to support the isolation bearings. Although the total loading from the updated OSD scheme was broadly in line with the safeguarded loading, changes to the distribution of loads required a detailed check of the roof structure against the original design.

Coordination with architectural design

Architectural design for the C134 contract was led by Hawkins\Brown Architects, who had previously completed the design of the adjacent upgrade works to the London Underground station at Tottenham Court Road. The architectural, structural and building services teams worked closely together to develop a coordinated design for the station. The design includes large areas of visual concrete, including the soffits of the public areas and the columns. Many of these elements were densely reinforced and, in some cases, included cast-in steel sections making the achievement of a high-quality finish extremely challenging. The success of this element of the design was therefore very much dependent on the skill and experience of the contractor.

Separated by platforms over 250m in length, the two entrances emerge in very different areas of central London. While the Eastern Ticket Hall's design sits within the bright commercial modernism of Centre Point at the intersection of Tottenham Court Road and Oxford Street, the design of the Western Ticket Hall reflects the darker, denser urban grid of Soho and the hi-tech industries that have occupied the older buildings in that area.

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Figure 12 Stressing of tendons to roof beams

"THE STRESSING OF THE FIRST TENDONS STARTED WITHIN A WEEK OF THE LAST SECTION OF SLAB BEING CAST"

The concept for the Western Ticket Hall roof reflects this, with a single central column and grillage of unclad ribbed and spine beams. This creates a slightly industrial structural aesthetic which is finished off neatly by bright bespoke stainless steel lighting drums (Figures 13 and 14) which are a signature feature of the station design. The drums were sized to sit neatly within the coffers between ribs to create a flush soffit.

Fortunately, C134 was commissioned for the design work to support a Schedule 7 Planning Application for the OSDs at the Western Entrance concurrently with the design of the ticket hall itself. This enabled the structural grid of the ticket hall roof to be optimised and coordinated to suit the residential planning grid of the OSD above, resulting in an efficient transfer system.

As part of value management undertaken in response to the Coalition Government's Spending Review in 2010, a link tunnel connecting the Western Ticket Hall and the Central Line was omitted. This had a series of knock-on effects on the design which allowed the internal arrangement of the box to be completely re-planned and simplified. The most significant change was the introduction of a single bank of escalators connecting the ticket hall at ground level with a new lower concourse at platform level (Level –4). This allows natural light to penetrate deep into the station, assisting with wayfinding.

The Level –3 propping slab above the lower concourse was formed into a grillage of downstand rib and spine beams, similar to that in the ticket hall. The downstands contained the lower chords of the transfer trusses and the ties of the strut-and-tie system used to transfer the walls of the ventilation tower above. The coffers are

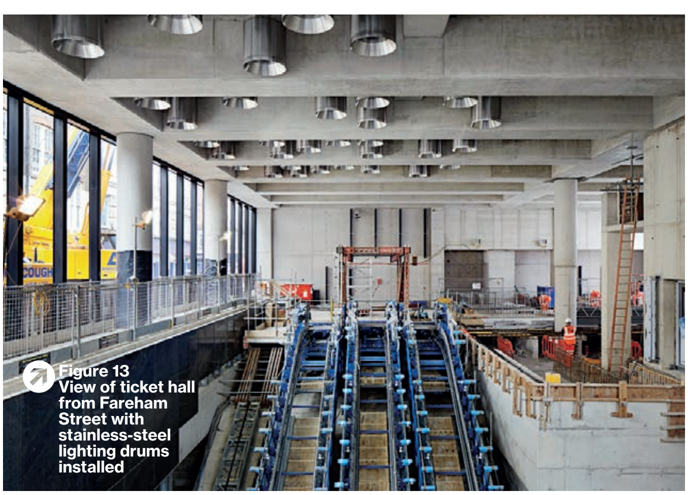

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Figure 13 View of ticket hall from Fareham Street with stainless-steel lighting drums installed

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Figure 14 Close-up view of stainless steel lighting drums between post-tensioned roof beams

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Figure 15 Station entrance and ticket hall

again used to house the lighting drums, unifying the appearance of the public areas of the Western Ticket Hall box (Figures 15 and 16).

Externally the station is split into two distinct blocks separated by Fareham Street, which was completely enclosed within the worksite during construction. It is being reinstated approx. 7m further north from its original position. This creates more space in the south block to accommodate the ventilation tower and electrical substation. The appearance of the station facades is coordinated with the design of the future OSD. The northern block containing the ticket hall is clad in panels of polished black concrete. The southern block is clad with panels of glazed bricks (Figure 17).

The ventilation tower above the Goslett Yard Box will eventually be covered by the OSD, but until that time it will be highly visible from Charing Cross Road. It was therefore specified to have a visual concrete finish with the joints between formwork panels and the tie holes arranged in a regular pattern (Figure 18).

Coordination with MEP services

Public circulation areas are located at ground level, Level –2 and Level –4. The remainder of the station box is largely occupied by mechanical, electrical and public health (MEP) services, with Level –5 given over entirely to distribution of services and ventilation ducts between the platform tunnels and service risers. Coordination between MEP services and the structural

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Figure 16 Image of lower concourse and eastbound platform at Level –4 design benefited significantly from three-dimensional (3D) modelling (Figure 19).

A large volume of the station is taken up by the tunnel ventilation systems. These are housed in ventilation towers at each end of the station which rise from Level –3 at approx. 15m below ground to Level +6 at approx. 20m above ground. The ventilation tower in the Western Ticket Hall box is located directly above the concourse area at Level –4 and is supported by concrete-encased steel-composite columns, with the lower sections of the ventilation tower walls acting as deep beams. Both towers house three fans with an internal diameter of 2.5m located at ground level to allow maintenance access directly from street level (Figure 20).

The draft relief connections to the western ventilation tower are made via a tunnelled connection from the crown of the westbound platform tunnel directly into the ventilation tower at Level –3 and through an opening in the Level –3 slab above the eastbound trainway, which connects to the ventilation tower via a duct. Heat from the trains is extracted through openings in the trackside edge walls below the platforms and ducted underneath the platforms to the Western Ticket Hall and Goslett Yard boxes at Level –5 before joining risers to connect to the ventilation towers at Level –3.

A network of water pipes was cast into the outer cover zone of the diaphragm walls and the internal piles. These were routed through the capping beams to allow the future installation of a ground-source heat pump system serving the OSD.

Figure 17 View north up Dean Street and into Fareham Street showing different treatments of facades to north and south blocks

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Design of diaphragm walls and foundations

Both the western and eastern boxes at Tottenham Court Road are formed with diaphragm walls to similar depths. Inside the boxes there are deep-level large-diameter bored piles to support the internal structure and retain the raft slab against future heave and water pressures. The western box has a split-level foundation with a dividing diaphragm wall (Figure 21).

All diaphragm walls are 1m thick with panel lengths of approx. 3m and are reinforced with conventional reinforcement. The shallower panels have a toe level 18m below existing ground level while the deeper box panels are 41m below ground level. All bored piles are 1.8m diameter. The deepest piles have toe levels 48m below ground level and a cut-off level 27m below ground level within the lower raft slab. The toe level of the shallow bored piles is 38.5m

"A LARGE VOLUME OF THE STATION IS TAKEN UP BY THE TUNNEL VENTILATION SYSTEMS"

below ground level with a cut-off level 12m below ground level within the upper raft slab. Both rafts found on the London clay layer. This is a relatively thin stratum in this area and the lower raft is close to the interface with the Lambeth Group below.

The diaphragm walls have penetrations on most elevations for access routes, ventilation and train ways. Internal lining walls were provided to transfer vertical loads around the openings and support the remaining stubs of diaphragm walls against earth pressures. All openings for tunnels allowed for the provision of waterproofing at the interface, which was ultimately designed by others.

The Western Ticket Hall box was constructed bottom-up. This was dictated by the need to reach founding level early to facilitate the passage of the tunnel boring machine (TBM) drive forming the eastbound alignment through the box. The walls were designed using Oasys Frew[1], a 2D analysis package. More rigorous analysis of soil–structure interaction was undertaken using PLAXIS[2] when a greater understanding of soil movements was required.

Piles were also conditioned by the two extreme load cases of full support of the station structure including the future OSD and the minimum load case of excavated

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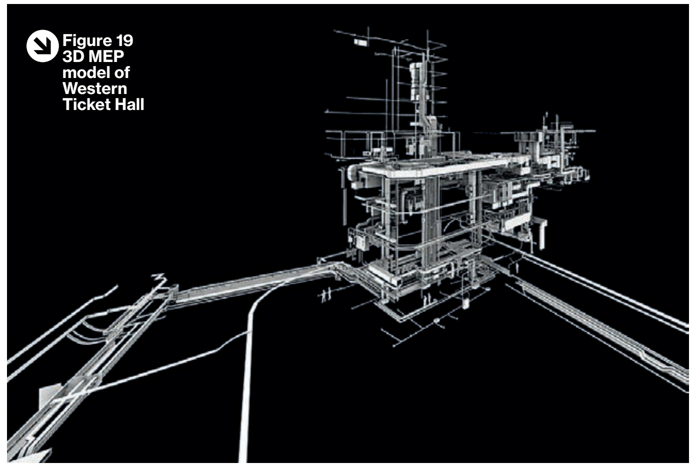

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Figure 19
3D MEP model of Western Ticket Hall

Figure 18
Panelised finish to Goslett Yard Box ventilation tower with expressed formwork joints and tie holes

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Figure 20
Installation of ventilation fans at ground level in Goslett Yard Box ventilation tower

'empty' box with full application of water pressure and heave resulting in tension within the piles.

The raft structure was similarly bounded by these ground conditions, acting both as a conventional ground-bearing raft with piles and a diaphragm transferring water pressures and heave loads to the piles. The raft was analysed using Oasys GSA Raft[3], an iterative software package which allows soil–structure interaction to be modelled using an embedded Oasys Pdisp program[4].

The foundation design evolved through a number of variations to the construction sequence, including having the TBM pass through either before or after excavation of the box. Both top-down and bottom-up excavation sequences were considered, as well as the use of a temporary sprayed-concrete access shaft. As a result, the final design was sufficiently robust to allow subsequent programme saving initiatives by the contractor.

Construction sequence

The design of all structures is conditioned by the application and sequence of loads. A buried deep box structure supporting an OSD is no different. The main difference is that the loads are large. The assumed construction sequence therefore has a significant impact on the design of the structure.

The Western Ticket Hall was initially considered to be a top-down construction, like its neighbour the Goslett Yard Box in the east. The advantage of this method of construction is that the permanent works propping slabs are built as the excavation proceeds, providing stability and full support to the neighbouring ground, resulting in smaller settlements and less likelihood of damage to adjacent buildings. The initial designs were developed on this basis since the Western Ticket Hall sits in the middle of a dense urban environment of mainly old buildings, many of them listed. There was also sufficient time in the programme to substantially complete the box in a top-down sequence before the TBM for the eastbound tunnel drive (christened Phyllis) bored through the diaphragm wall and passed through the box.

However, the redesign of the Western Ticket Hall for the Government's Comprehensive Spending Review in late 2010 left insufficient time to complete the design, tender the work and build the box in a top-down sequence before Phyllis arrived in early 2013. Furthermore, there was not enough time after completing the tunnelling works to construct the whole of the Western Ticket Hall. It was therefore necessary to find a way to build a substantial portion of the Western Ticket Hall before Phyllis arrived.

This led to the adoption of a bottom-up construction sequence. This method utilises temporary propping to support the perimeter diaphragm walls while the excavation to formation level proceeds. The diaphragm walls were quickly redesigned to account for the installation of temporary props and waling beams during the excavation phase. This allowed for an early release of tender information for the diaphragm walls, temporary props, piles and raft while the detailed re-design of the internal propping slabs continued. The relatively unobstructed excavation was completed quickly in only six months, providing sufficient time to construct the raft before Phyllis arrived. The resulting shaft was one of the deepest open excavations ever constructed in central London.

The contract procurement was split into four phases at the Western Ticket Hall site:

- Starting in 2010, Site clearance and demolition was carried out by McGee under Contract C208.
- The foundations, including diaphragm walls panels and piles, were installed between May 2011 and January 2012 under Contract C421 by Balfour Beatty Morgan Vinci Joint Venture.
- The Western Ticket Hall box was excavated with temporary propping installed and the raft slabs cast under Contracts C300 and C410 by Bam Ferrovial Kier Joint Venture. The box was excavated in two stages. The deep section, through which the eastbound TBM passed, was excavated between May and November 2012, while the shallow section was excavated between March and May 2014.
- The internal structure of the station, plus the superstructure and ventilation towers at both eastern and western ends, as well as the building services and architectural fit-out of the station was carried out under Contract C422 by Laing O'Rourke, starting in May 2014 with most of the structure complete by the end of 2016.

The design team supplied an assumed construction sequence and a series of charts illustrating trigger levels for both temporary prop forces and diaphragm wall movements. The C300/410 contract under which the shaft was excavated monitored horizontal movements in the diaphragm walls using shape arrays and prop loads using strain gauges. Both monitoring methods gave 24/7 data gathered by data loggers which were reviewed at intervals throughout the day. The data were compared to the designer's charts to monitor behaviour of the walls and props.

Before excavation began, proposals were discussed between Contract C134, the contractor and Crossrail for the use of the Observational Method[5] to minimise the use of temporary propping. When the excavation sequence reached below –3 propping level, it was evident that the ground movements were less than predicted. C134's

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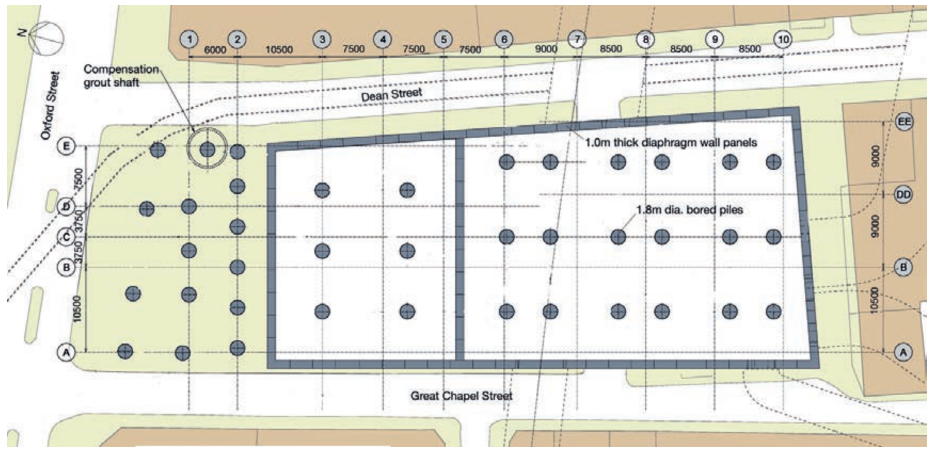

Figure 21
Plan of Western Ticket Hall foundation piles and diaphragm wall panels

"THE RELATIVELY UNOBSTRUCTED EXCAVATION WAS COMPLETED QUICKLY"

geotechnical engineers undertook a back analysis of the measured movements and produced updated charts for the remaining excavation stages, eliminating the entire lowest level of props[6].

This proposal ensured that the base raft sequence was completed in time, eliminating the risk of delay to the TBM. Works for reception of the TBM included stitch drilling of openings in the diaphragm walls. The openings were temporarily filled with domed sprayed concrete-lined headwalls to form 'soft eyes'. The TBM's route between the two sides of the box was temporarily filled with foamed concrete so that the TBM could traverse across the box without the need for jacking frames and other works. Junk segments were erected through the station which remained until the TBM had completed its drive as the conveyors and ventilation were maintained.

The design of the temporary propping (Figure 22) was the responsibility of the C300/410 contractor; however, C134's design team looked at two indicative propping scenarios, an open transverse solution spanning across the entire excavation and a slender system utilising king piles at mid-span with bracing. The two indicative solutions provided a range of prop stiffnesses to the diaphragm walls. The indicative upper and lower stiffnesses were modelled using Frew to provide a range of prop forces for the contractor's temporary works design.

The indicative propping layout was distributed vertically to facilitate excavation of the box, removal of the props and construction of the permanent works in a bottom-up sequence. The design loads considered the construction loading during the build phase of works. These included a nominal impact load on a prop and the loss of any prop at any level. The prop-loss case had the greatest impact on the sizing of the waling beams, the purpose being to protect against progressive collapse of the excavation.

The C300/410 contractor elected to use the indicative open transverse propping arrangement which included muck-out access at each end of the site. Its design was modified to reflect the size of available tubing, lifting equipment and support arrangements.

Prior to the props taking up any load, selected members were identified for instrumentation; this included four strain gauges around the circumference of the tubing, at each end and at mid-span. The gauges were distributed at 45°, 135°, 225° and 315° around the circumference, rather than the more conventional 0°, 90°, 180° and 270°; while the gauges were protected, the selected distribution provided further protection against glancing blows. The gauges were calibrated, and initial strains checked when the erection support was removed. Thereafter, the gauges transmitted data 24/7 for review. The data were converted to axial loads and bending moments to check that the props performed as per the C134 designers' monitoring charts.

The waling beams were supported against a prepared surface on the diaphragm wall. The contractor elected to use shelf brackets rather than conventional gallows

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Figure 22 View of temporary propping and raft slab construction

brackets. The waling beam support was later developed into an embedded U-bar confined in the packing concrete between the waling and wall face.

The C134 designers' temporary works sequence had been included in the Category III independent check. While the design allowed for versatility in construction, the process and turnaround of changes added a restriction and risk to some of the contractor's proposals. An example of this constraint was a restriction on dig levels to comply with the check. The contractor wanted to excavate approx. 1m lower at the perimeter of the excavation to facilitate the installation of the shelf brackets. The restriction confined these reduced dig levels to pockets at the shelf location.

As mentioned earlier, the Observational Method was used to eliminate the lowest level of props. A consequence of this was that the excavation to formation for the raft required sequential excavation using hit-and-miss sequences with berms and a thicker blinding layer designed to act as a prop.

Interfaces with tunnels

There are four types of interface with tunnels at the Western Ticket Hall: running tunnel, platform tunnel, concourse tunnels and service tunnels (for MEP and ventilation).

As the Western Ticket Hall box existed before the tunnels, the design of waterproofing connections was generally the responsibility of the tunnel designer. Where a tunnel penetrates a diaphragm wall, the penetration is surrounded by an internal lining wall. The purpose of the lining walls is to transfer vertical loads around the penetration. They are all designed against hydraulic pressure should the diaphragm wall joints leak. Water tightness is provided by a drained cavity throughout.

The connection between the running tunnel and the lining wall is provided by embedding the tunnel segments into the lining wall. The joint is then reinforced with two hydrophilic strips around the circumference of the tunnel segments and a re-injectable grout tube to act as a fallback should the first line of defence be compromised. The boxes for the re-injectable tubes are located to be accessible from back-of-house areas.

The interface with the sprayed concrete-lined platform and concourse tunnels (Figure 23) is similar. The secondary sprayed concrete lining of the tunnels penetrates into the box to interface with the box lining walls. The tunnel designer developed a series of barriers of re-injectable grout tubes at the interface with the sprayed concrete lining and lining wall and around the annulus between the first-stage and second-stage concrete. The box designer introduced a confined flexible tape secreted inside a notch to provide a flexible waterproof membrane where differential movements are encouraged to occur. The tunnels are expected to become squat in time and will therefore move relative to the diaphragm walls. The notch introduces a crack inducer that is protected by the confined flexible tape, which in turn is supported by the secondary lining. Connections between tunnels and structures are notorious weak points and are therefore also provided with a drainage management layer which directs any future leakage to the station drainage system.

Back-of-house connections between sprayed concrete-lined tunnels and the box for tunnel ventilation and MEP service routes also have re-injectable grout tubes and drainage management systems.

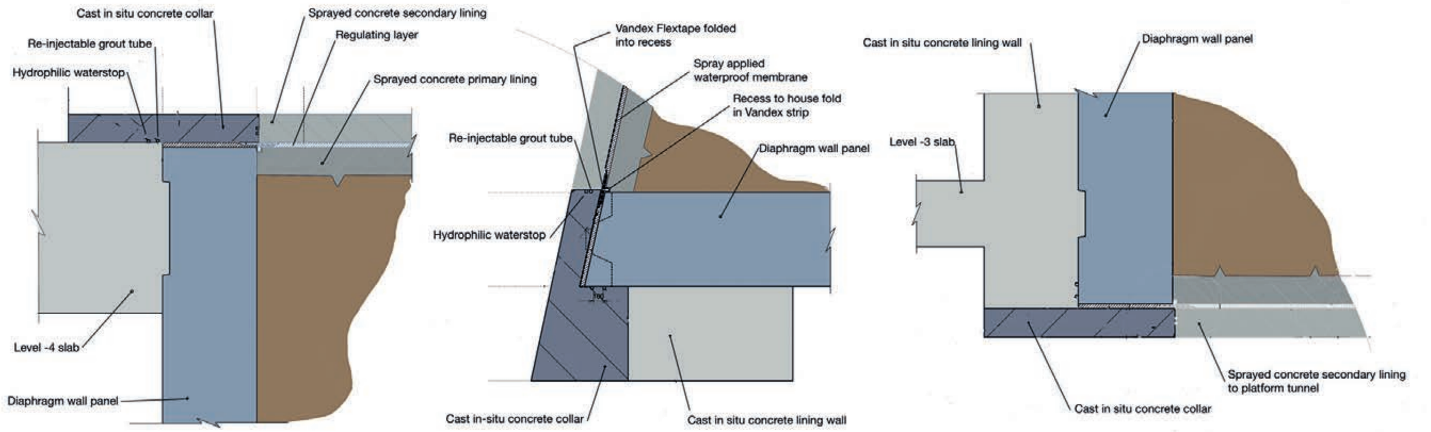
The design of connections is based on the assumption that the connections between tunnels and the box will most probably leak during the lifetime of the station; they are therefore provided with multiple layers of protection. Public areas have additional protection because water ingress into these areas would be poorly perceived, could damage architectural finishes and the access arrangements to these zones for remediation are more restricted.

The connections between tunnels and the station box are all within the London clay and large quantities of water are not expected from this source. However, when it does materialise it will have high pressure. The likely source of water will be down the external face of the diaphragm walls, which are within the surface deposit terrace gravels at high level. The connections are detailed to provide protection against water from either source.

Interfaces with Goslett Yard Box

The primary structure of the Goslett Yard Box was designed by Atkins under the London Underground contract for the upgrade of Tottenham Court Road underground station. However, the design of the ventilation tower from ground level

Figure 23
Section through connection between eastbound platform tunnel and Western Ticket Hall box at tunnel axis level



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a) Invert level of platform tunnel

b) Axis level of platform tunnel

c) Crown level of platform tunnel

upwards remained part of Crossrail Contract C134. The design of the Goslett Yard Box, including the ventilation tower, had to be safeguarded for the construction of a future OSD. In this case, the planned development is for a mixed theatre and office building by Derwent London with Arup as structural engineers. It was necessary to agree a set of interface parameters between the three teams of structural designers at an early stage. These included agreeing a column grid, stability core and maximum loading. It was originally intended that stability of the OSD should be independent of the station. However, this proved impractical and it was decided that the ventilation tower would need to act as a stability core.

The OSD loadings were agreed in 2010 to allow the design and construction of the Goslett Yard Box to proceed in advance of the rest of the Elizabeth line station. In 2012, following design development, changes were made to the OSD loading requirements and positions. These required the introduction of additional transfer beams on the roof of the ventilation tower and additional analysis of the structure to ensure that the loading from the ventilation tower into the Goslett Yard Box, which by then was already under construction, was not exceeded.

Conclusions

The station has been successfully delivered to programme. At the time of writing, it is in the final stages of fit-out and is due to open in December 2018. The original vision for the design has been retained throughout the construction phase.

Coordination of the structural design with both the building services design and OSD continued during the construction phase.

A notable achievement during the construction phase of the Western Ticket Hall box was the use of the Observational Method to eliminate the lowest level of temporary props. This was made possible by the high quality of instrumentation and monitoring employed on the project.

Perhaps the most impressive achievement of the design was the degree of collaborative working between so many designers and contractors across so many interfaces. This proved especially challenging when dealing with contracts outside the Crossrail programme. Crossrail Ltd recognised very early that the success of the project depended on this working well. Its approach to co-locating design and construction teams, ease of communication between parties and well-controlled access to information made this possible.

Project team

Client: Crossrail Ltd
Project delivery partner: Bechtel, Halcrow, Systra
Civil and structural engineer: Arup Atkins Joint Venture
MEP engineer: Arup Atkins Joint Venture
Architect: Hawkins\Brown
Foundations contractor: Balfour Beatty Morgan Vinci Joint Venture
Excavation and tunnelling contractor: Bam Ferrovial Kier Joint Venture
Main structural works and station fit-out contractor: Laing O'Rourke

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►St. John A., Barker J., Frost S. and Harris D. (2017) 'Crossrail project: a deep-mined station on the Elizabeth line, London', *Proc. ICE Civ. Eng.*, 170 (5), pp. 47–56

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Construction of the ventilation towers at Tottenham Court Road Elizabeth line station

Joseph Maltezos
MEng, CEng, MICE
Project Engineer, Laing O'Rourke, UK

Tim Harman
MEng, CEng, MICE
Principal Engineer, Laing O'Rourke, UK

NOTATION

FE	finite element
GGBS	ground-granulated blast-furnace slag
GYB	Goslett Yard Box
WTH	Western Ticket Hall

Introduction

Tottenham Court Road Elizabeth line station is a new station located in the heart of London, adjacent to Oxford Street, being delivered as part of the Crossrail programme. The station is expected to accommodate more than 200 000 passengers every day when it becomes operational in December 2018. The station is of paramount importance due to its strategic location, interchange with the London Underground and future link to Crossrail 2.

Tottenham Court Road consists of two entrances on the east (Goslett Yard Box) and west (Western Ticket Hall) sides of Soho Square, each of which has a ventilation tower equipped to ventilate the 250m long new platforms and running tunnels located 25m below ground (Figure 1). The ventilation towers are two of the largest overground structures on the entire Crossrail project, and presented a vast array of challenges due to their locations as well as their technical complexity.

Following the award of the contract to Laing O'Rourke, discussions were held about changing the design of the superstructures to precast concrete. However, due to the design process that would have been required to alter the concept, and lead time for bespoke precast elements, there was not sufficient time to alter the construction methodology. Therefore, a traditional *in situ* concrete approach was used. In fact, due to the precise planning and coordination of this approach, it resulted in a more economical solution than the precast option.

Eastern ventilation tower (Goslett Yard Box)

The ventilation tower in the Goslett Yard Box is part of a five-storey *in situ* reinforced concrete superstructure and five-storey substructure (Figure 2). This will connect the existing London Underground station with the Elizabeth line platforms, and will also be used for services and ventilation.

It was realised at an early stage that to facilitate the successful completion of the entire Tottenham Court Road scheme, significant changes were needed to the construction sequence of the Goslett Yard Box. One of the main programme drivers at the station was to ensure the tunnel fit-out programme was not adversely affected due to its position on the critical path.

There were only two access points into the tunnels and one of these was through the Goslett Yard Box. This access point was known as the Goslett Yard Box mole hole. It was therefore imperative for the scheme's success that this was left open for as long as possible.

The construction of the superstructure was split into two phases (Figure 3). The major challenges in the construction of the Goslett Yard Box superstructure were the interface around other construction activities, architectural finished concrete and strict programme requirements.

Construction phasing

One of the main project milestones was the handover of rooms within the structure to the fit-out contractor, known as Key Date 3. This served as a sectional completion for the project. Splitting the structure into two phases created two major benefits:

- The mole hole (Figure 4) allowed access to

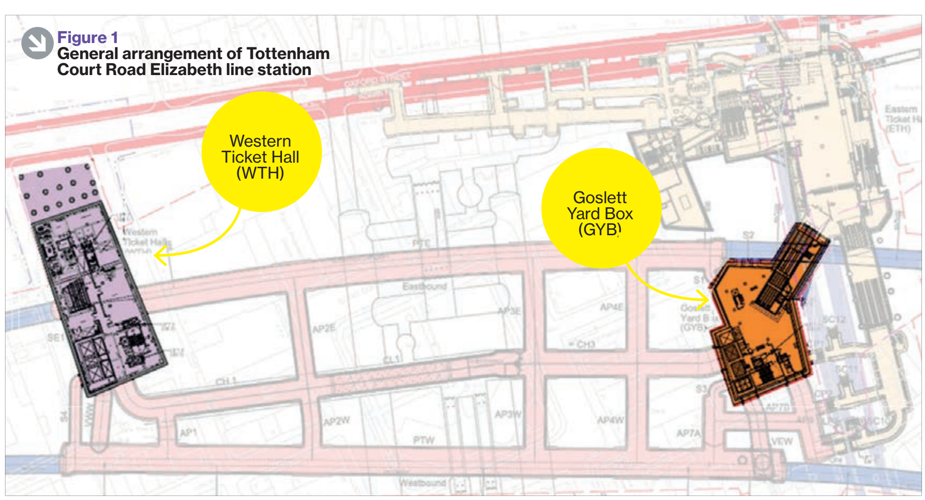


Figure 1 General arrangement of Tottenham Court Road Elizabeth line station

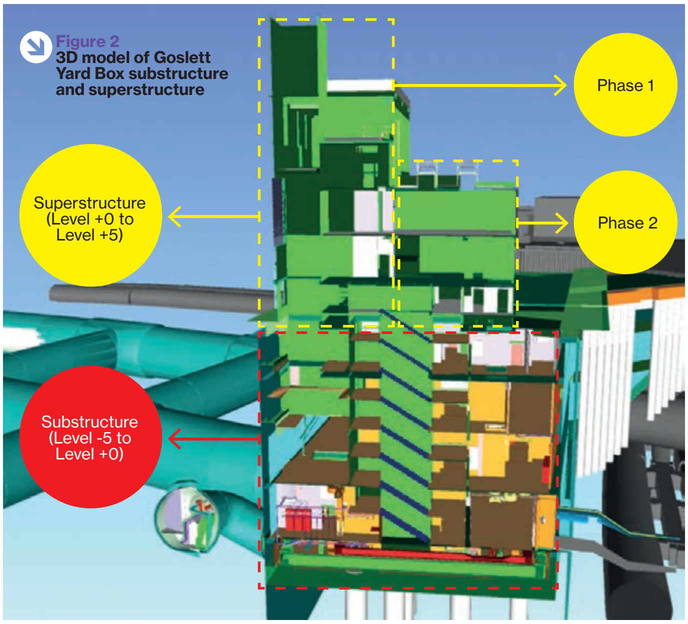


Figure 2
3D model of Goslett Yard Box substructure and superstructure

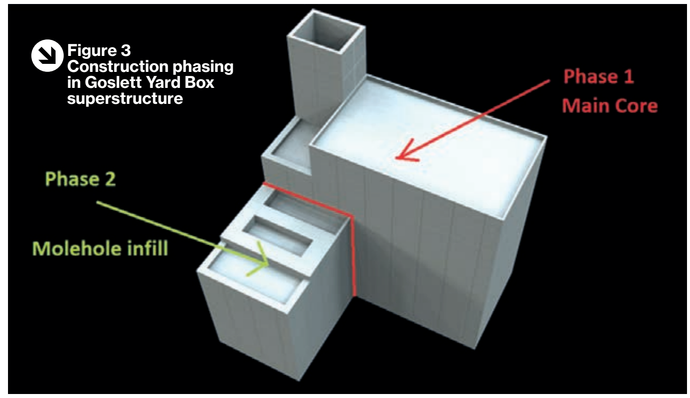


Figure 3
Construction phasing in Goslett Yard Box superstructure

the tunnels, which could then be left open for another five months.

- All the major rooms, above ground, required for handover at Key Date 3 were within phase one.

Architectural finished concrete

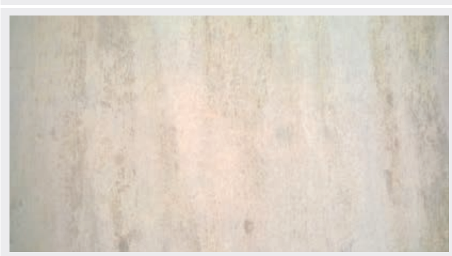
At the Goslett Yard Box the designer had specified that the external walls would adhere to a strict architectural specification for the concrete finish. This requirement was set by Westminster City Council as part of the planning permission, due to uncertainty around the construction of the oversite development in the near future, and was specified within the Works Information.

A series of trials with various concrete mixes was undertaken to agree a benchmark (Table 1). Working closely with the Laing O'Rourke materials laboratory and the concrete supplier, an innovative concrete mix was developed, achieving both the structural and architectural specifications (Mix 3F). This particular mix included a higher amount of limestone filler, which increased workability, reduced the number and size of blow holes, and achieved the desired finish. It was subsequently approved by the architect.

A major concern when working adjacent to Oxford Street was the regularity of the concrete supply. Significant time was invested during initial trials to ensure that the concrete had a four-hour working-life window. The extra effort spent at this stage reassured the site teams when completing concrete pours that cold joints were significantly less likely as a result of traffic congestion in the area around the site.

To meet the Key Date, a climbing formwork system was used. To achieve the rigorous architectural requirements, which included a regular tie-hole pattern arrangement and specified construction joints and ply lines, a bespoke climbing shutter was used (Figures 5 and 6), with *the National Structural Concrete Specification* 4th edition (NSCS4)[1] used as a guide. Due to site constraints, the shutters were made off site and delivered to site ready for the pours. In addition, the formwork was designed to withstand the concrete pressures imposed by a fast rate of rise (in accordance with CIRIA Report 108[2]) to facilitate concrete pumping. To minimise construction joints, additional anti-crack bars were installed to facilitate larger concrete pours by limiting thermal cracking.

TABLE 1: DEVELOPMENT OF CONCRETE MIX TO ACHIEVE ARCHITECTURAL REQUIREMENTS

	Trial No. 1	Trial No. 2
Mix	Original Concrete Mix 3B	Concrete Mix 3F
Additional information	Concrete mix for slabs and walls in Tottenham Court Road with 55% GGBS	Special mix for architectural finish concrete with 55% GGBS and 60kg/m³ more limestone filler than Mix 3B
Strength	C32/40	C32/40
Finished surface	Excessive blow holes in struck concrete surface	Reduced number and size of blowholes
Workability	Moderate workability	Improved workability
Photographs of struck surface		

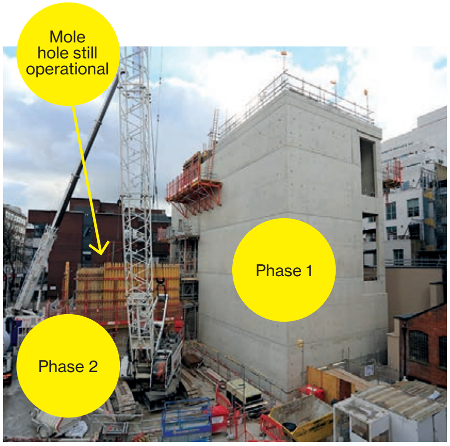
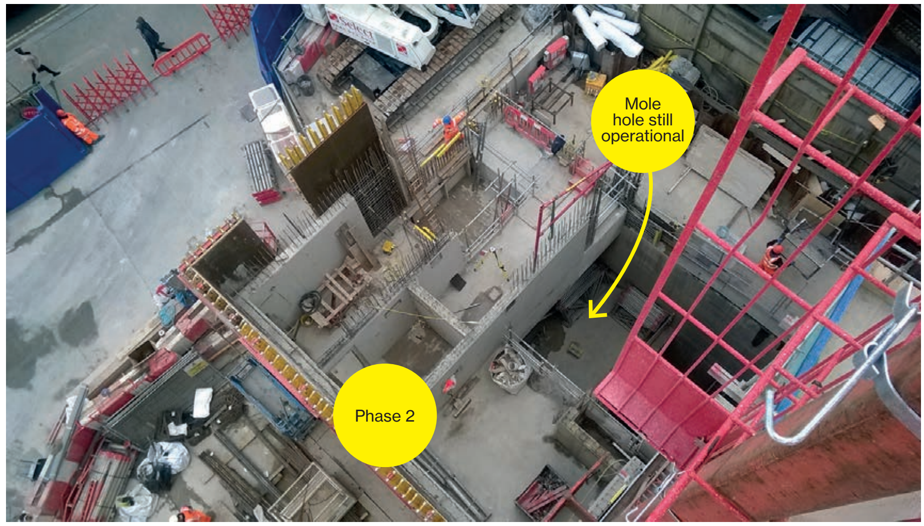


Figure 4
Phase 1 and 2 under construction with mole hole operational



Installation of the climbing formwork was a high-risk operation as it required working at height. A detailed visual risk assessment and method statement were written so that everyone could be briefed and to facilitate greater understanding. In addition, the area was classified as restricted access and a specific safety induction was required for all visitors to ensure their health and safety.

The major engineering challenge of the climbing formwork was its temporary support in the large openings in the external walls (up to 5m × 6.5m). Two options were considered to overcome this problem: the use of temporary reinforced concrete beams; or the use of temporary steel I-beams. After reviewing both options, it was decided to use steel I-beams fixed on either side of the openings with vertical supports at the anchor points (Figure 7), as this was considered an easier system to install with fewer repairs likely to be required after the beams' removal.

"THE MOLE HOLE ALLOWED ACCESS TO THE TUNNELS, WHICH COULD THEN BE LEFT OPEN FOR ANOTHER FIVE MONTHS"

After reviewing the original temporary works scheme, it was observed that there was a potential area for significant deflections due to the localised forces applied at the external flanges. The team suggested that additional stiffeners should be welded at the anchor points. The temporary works designer then verified this visual assessment by using a finite-element (FE) model and approved the installation of stiffeners.

By spending time in the planning stage thinking about intricate details such as where construction joints would be located, pour lengths and heights, shutter design and temporary works, it allowed the focus on site to be on ensuring the plan was implemented. This focused approach allowed the sectional completion date to be successfully met at the Goslett Yard Box due to a full understanding of the plan and the key roles everyone played within the team (Figure 8).

Western ventilation tower (Western Ticket Hall)

The Western Ticket Hall superstructure is a seven-storey *in situ* reinforced concrete structure, supported by a five-storey substructure, which will accommodate services and ventilation equipment (Figure 9). The construction programme for this structure was limited to just 20 weeks, before it was handed over to the system-wide contractors for installation of the ventilation fans, mechanical and electrical

Figure 5
Architectural concrete finish in Goslett Yard Box superstructure

a) Finished structure

b) Architectural elevation

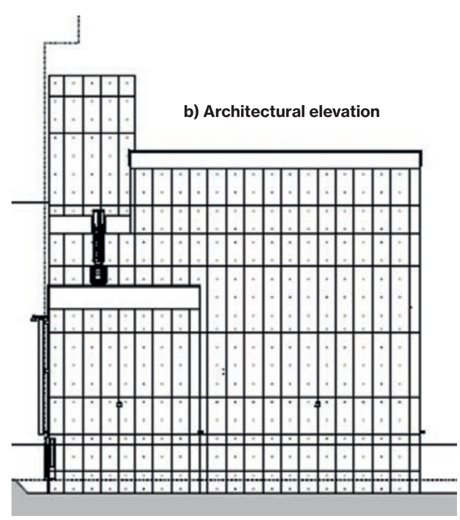
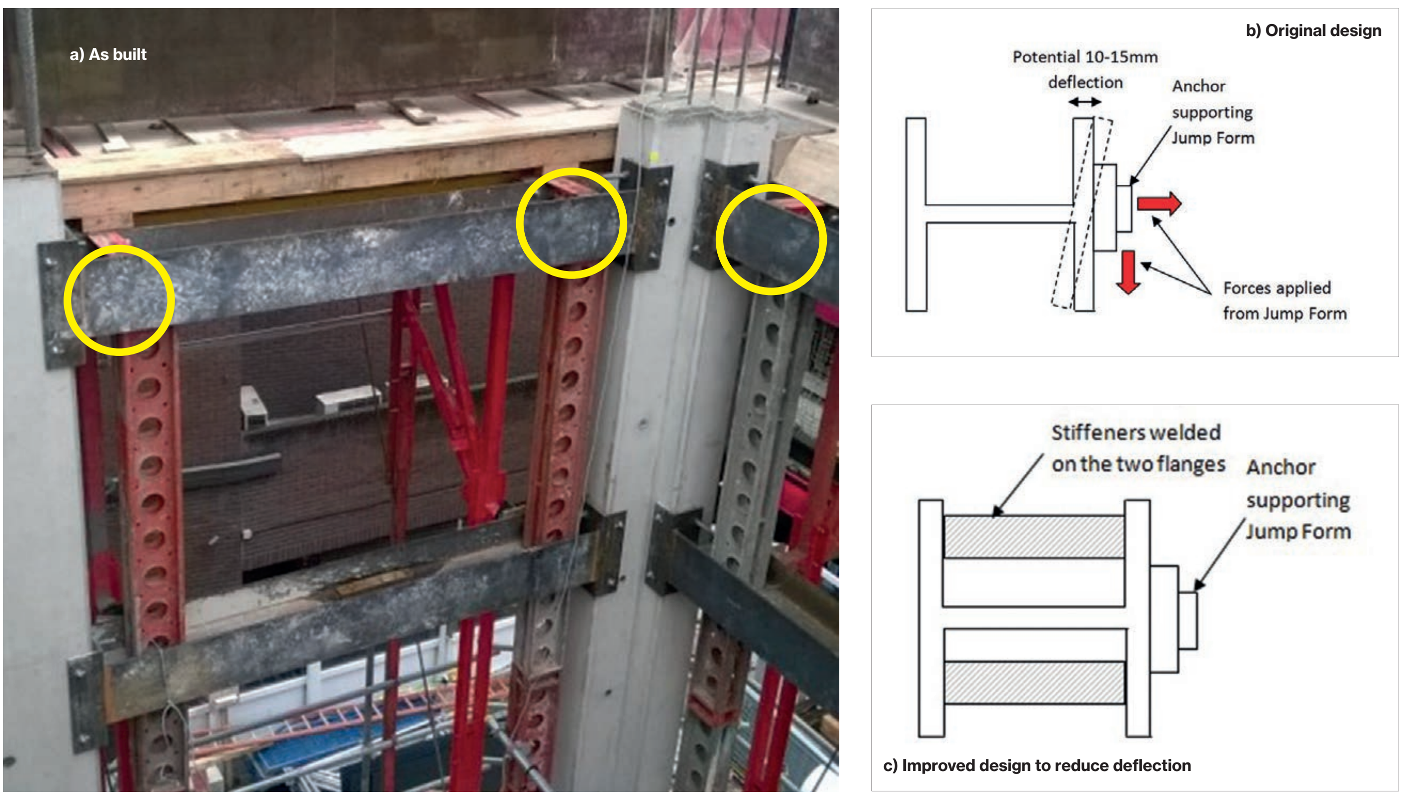


Figure 6
Temporary support of climbing formwork in wall openings

Figure 7
Stiffeners welded in temporary I-beams to reduce deflections



equipment. As with the Goslett Yard Box, the Western Ticket Hall superstructure had architectural finished concrete specifications, technical challenges, logistical restrictions and strict programme requirements.

Construction sequence

The construction methodology and sequence were fundamental to achieving the successful completion of the Western Ticket Hall on time and under the strict programme requirements. The selected sequence of works included the construction of the primary walls up to a certain height (using the jump-form system) before the construction of the slabs could commence. This would enable the construction of the walls and the slabs to occur concurrently and significantly reduce the programme of the works.

For this sequence to be followed, an FE model was developed (Figure 10) to examine the height to which the walls could be constructed prior to construction of the slabs, before excessive stresses and deflections were imposed on the structure in its temporary stages. Following this analysis, a height of 7.2m was specified for the walls before the slab construction could commence.

The selected sequence had an additional benefit: the reduction of the noise impact on local stakeholders. Due to the walls being constructed first, a physical barrier was in place between the construction of the slabs and the surrounding buildings, which significantly reduced the noise impact during the construction phase.

Technical challenges

The construction of the walls ahead of the slabs imposed a major technical challenge: the provision of continuity in the reinforcement between the walls and the beams. The original design specified L-shaped couplers cast into the wall (Figure 11). Due to the significant bending moments

Figure 8
Team who delivered scheme

and shear forces, one of the beams required B32 bars cast into a 250mm wide wall, which caused clashes with the jump-form system.

Following extensive research, the Laing O'Rourke engineering team proposed the use of terminators. These are headed bars which provide the same anchorage as L-shaped bars by activating the shear cone and placing the concrete into compression. To prevent concrete pulling-out, which is one of the failure mechanisms of this system, additional reinforcement was added horizontally along the terminators. The advantage of the terminators over the L-shaped bars was a reduction in the anchorage length (due to the headed end) which suited the thickness of the wall and the construction methodology.

Figure 9
Western Ticket Hall superstructure with climbing formwork

This proposal was supported by research papers[3,4] and industry specialists in mechanical anchorage. It was accepted by the structural engineers and provided a solution to a technical problem that could potentially have hampered the construction sequence and put the construction of this ventilation tower at risk.

Innovative technologies

In the Western Ticket Hall superstructure, the early concrete strength was monitored using wireless thermocouples – a pioneering technology. The wireless thermocouples remotely record the temperature of the concrete in real time during curing. The concrete strength is then determined using maturity graphs (cores were taken and crushed to gain compressive strength during trials to ascertain concrete strength). The use of thermocouples was necessary at Tottenham Court Road for the following reasons:

- The concrete mix contained 55% ground-granulated blast-furnace slag (GGBS) to comply with the project specification for lower embodied CO_2. The GGBS is used as a cement replacement and results in slower strength gain.
- To comply with the architectural concrete specifications, striking was permitted:
  - for concrete strength greater than 5N/mm² for walls
  - when the temperature difference between the concrete and the surrounding air was less than 20°C, in order to avoid cracking from thermal shock.
- According to the climbing formwork design,

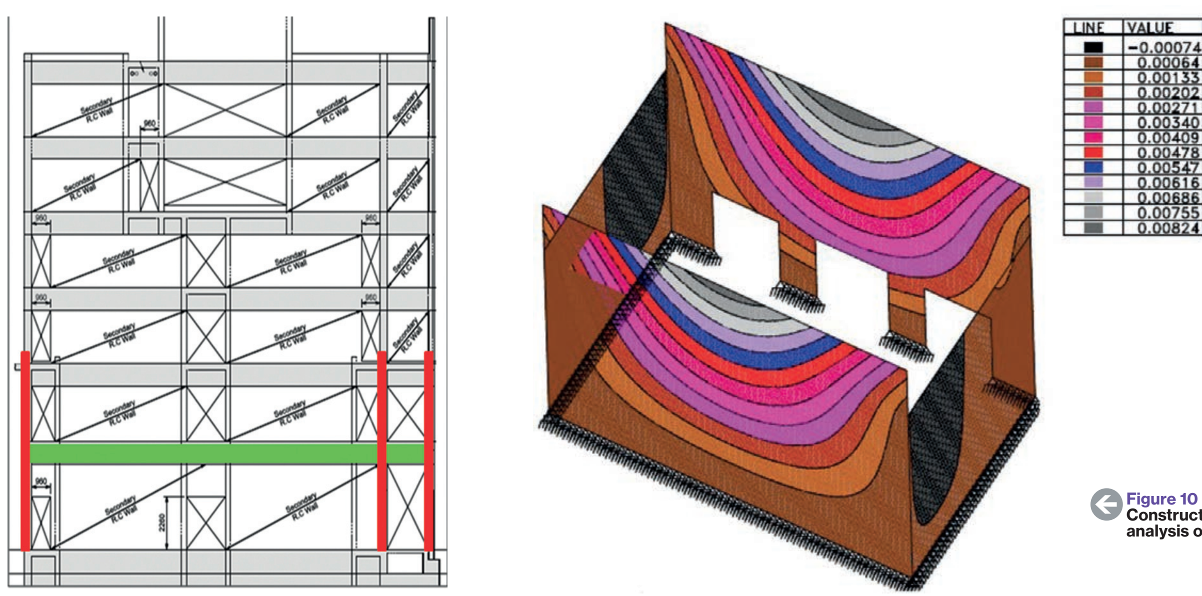


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Figure 10
Construction sequence and FE analysis of temporary structure

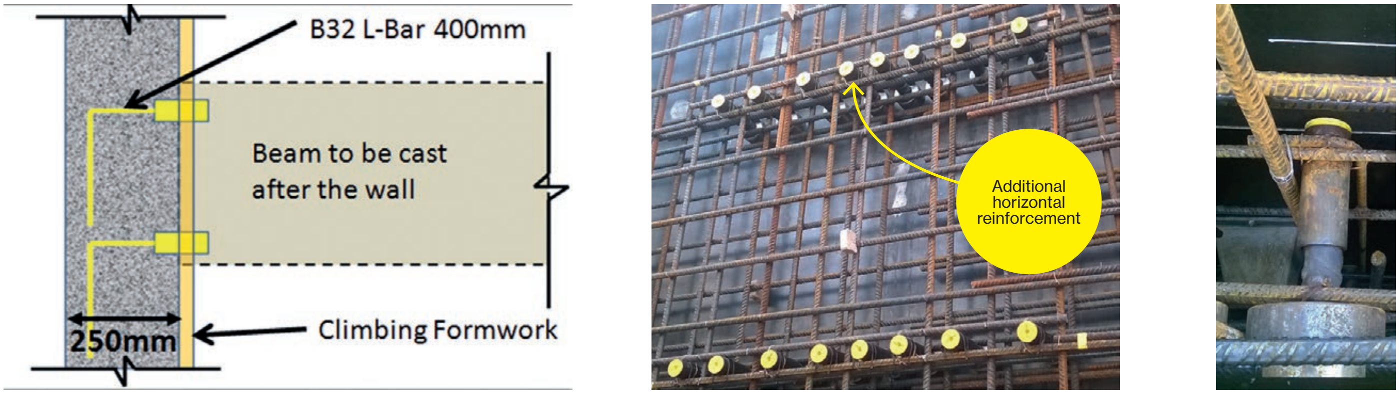

Figure 11
Use of terminator for wall–beam connection

the concrete strength had to reach 20N/mm² before the platforms could be lifted to the next level.

With the wireless thermocouple, the strength of the concrete could be viewed in real time in a phone application or on the website (Figure 12), instead of a more time-consuming operation using traditional thermocouples. The use of this innovative technology enabled striking of the formwork just in time, ensuring compliance with the specifications and bringing significant programme savings.

Logistics and public interface

The central location of the project adjacent to the busiest high street in Europe, with more than 300 000 pedestrians every day and more than 100 buses every hour, imposed a logistical challenge. To overcome this, the construction team employed the latest technology in booking and tracking vehicles to plan and coordinate just-in-time deliveries.

All the deliveries were booked using a logistics management system (Juggler) and discussed in a daily coordination meeting between all the trades working on the project. This was to ensure that adequate space was available on site, as well as crane availability to offload the deliveries. The vehicles were also tracked using GPS technology to ensure the precise delivery time was known and to prevent clashes with other deliveries or other works on site.

Due to the sequence of works, the internal formwork panels in the Goslett Yard Box had to be sent off site during the construction of the slabs. Instead of sending them to the supplier's yard, they were temporarily stored in a nearby yard owned by Laing O'Rourke. This initiative contributed to a significant reduction of CO_2 emissions (Table 2).

The day-to-day interface with the public and the numerous businesses and

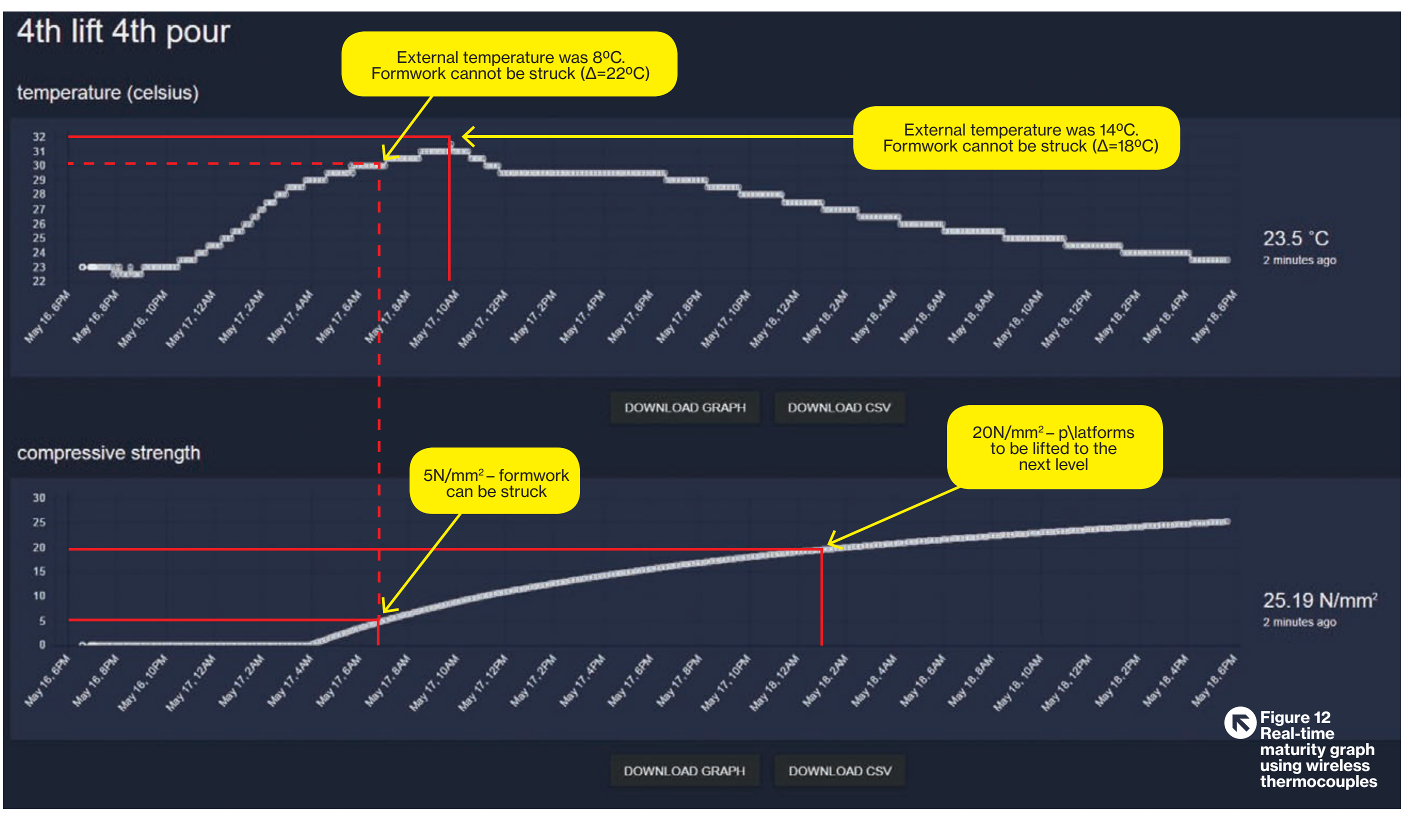

Figure 12
Real-time maturity graph using wireless thermocouples

TABLE 2: SAVINGS IN CO_2 EMISSIONS FROM USE OF LOCAL YARD FOR STORAGE OF FORMWORK PANELS

	Storage of formwork panels in supplier's yard in Rugby	Storage of formwork panels in Laing O'Rourke's yard in Westferry Road
Total distance for return trip	270km	22km
CO_2 emissions per km*	1.852kg/km	1.852kg/km
CO_2 emissions per return trip	500.04kg	40.74kg
Total number of trips for GYB construction	21	21
Total CO_2 emissions	10 500kg	856kg
Total savings in CO_2 emissions	**9644kg**	

* Data from Environment Agency's Carbon Calculator based on average 15t load per trip

stakeholders around Tottenham Court Road caused both logistical issues and operational challenges during the construction works. Generation of noise and vibration is an unavoidable outcome of the construction works for a complex infrastructure project. To minimise the impact on the local stakeholders, continuous liaison, early engagement and mutual understanding of each other's needs was fundamental.

For example, adjacent to the Western Ticket Hall site there is a school and a recording studio which could potentially have been impacted by noise and vibration during exam periods and music recordings, respectively. The construction team and stakeholders worked closely together so that exam dates and recording sessions were embedded in the construction programme, ensuring that noise and vibration during those times were kept to a minimum. An example of this was all demolition work below ground ceasing during recording sessions, as the vibration generated by the activity interfered with the recording equipment.

Conclusions

The construction of the two ventilation towers at Tottenham Court Road posed myriad technical and non-technical challenges. The scale of these towers, their onerous architectural specifications, in conjunction with their central location and the strict programme requirements, made these structures complicated projects in themselves. However, with a commitment to succeed and to resolve the technical challenges, the Laing O'Rourke engineering team (Figure 13) worked closely with the client and designers to implement new ways of doing things to improve safety, efficiency and productivity.

Figure 13 Lang O'Rourke and Crossrail team celebrating completion of Western Ticket Hall ventilation tower

The interface with the public and the local population imposed logistical and operational challenges to the site. These were overcome through continuous engagement with the stakeholders and the team's aspiration to minimise the impact on them and their businesses.

The experience gained from the construction of these ventilation towers, which is shared in this paper, will be valuable on other projects across the industry and on the future extension of Tottenham Court Road station to accommodate Crossrail 2.

"EXAM DATES AND RECORDING SESSIONS WERE EMBEDDED IN THE CONSTRUCTION PROGRAMME"

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St Giles Circus: meeting the challenges of building above the Elizabeth line

Clive Fussell
MEng, CEng, MIStructE
Director, Engenuiti, London, UK

NOTATION

BIM	Building Information Modelling
NLEB	Northern line Escalator Box
OLE	overhead line equipment
OSD	oversite development
RC	reinforced concrete

Introduction

The St Giles Circus project involves the £150M redevelopment of a central London site adjacent to Tottenham Court Road Station (Figure 1). The structural and civil engineering design was undertaken by Engenuiti, with geotechnical advice from Donaldson Associates. The development includes leisure, retail, commercial, residential accommodation and a boutique hotel. Above ground, the building incorporates an immersive multimedia urban gallery hung from two-storey steel trusses cantilevered over Tottenham Court Road Station. Below ground, a 2000-person basement venue for live music and events is being built 6m above the eastbound Elizabeth line tunnel, adjacent to the London Underground Northern line platform tunnels and above the inclined cut-and-cover concrete box housing the escalators that provide public access to the Northern line.

There have been significant engineering challenges with the construction of the deep basement over the Elizabeth line and limiting the movement and deformation of the tunnel. A scheme was designed and approved to build the basement above in sections, using deep piles and tunnelled 'adit' beams to create 50m deep piled 'staples' to hold the tunnel down.

Collaboration with Arup brought to the table previous experience of excavating down to the Elizabeth line tunnel to form the Paddington Station box. The St Giles scheme was calibrated against the significant monitoring data collected by Arup from the Paddington Station project, leading to a revised construction scheme which was quicker, safer and provided a £7M saving. Through collaboration, knowledge sharing and innovative design of the foundations and basement, the team has maximised the value brought to the client within this constrained site.

From an early stage, the client brief for the development was quite clear: maximise the size of the basement to maximise the value of the site; and provide a flexible basement performance space with a large clear height that can be used for music events, product launches, music rehearsals and club nights.

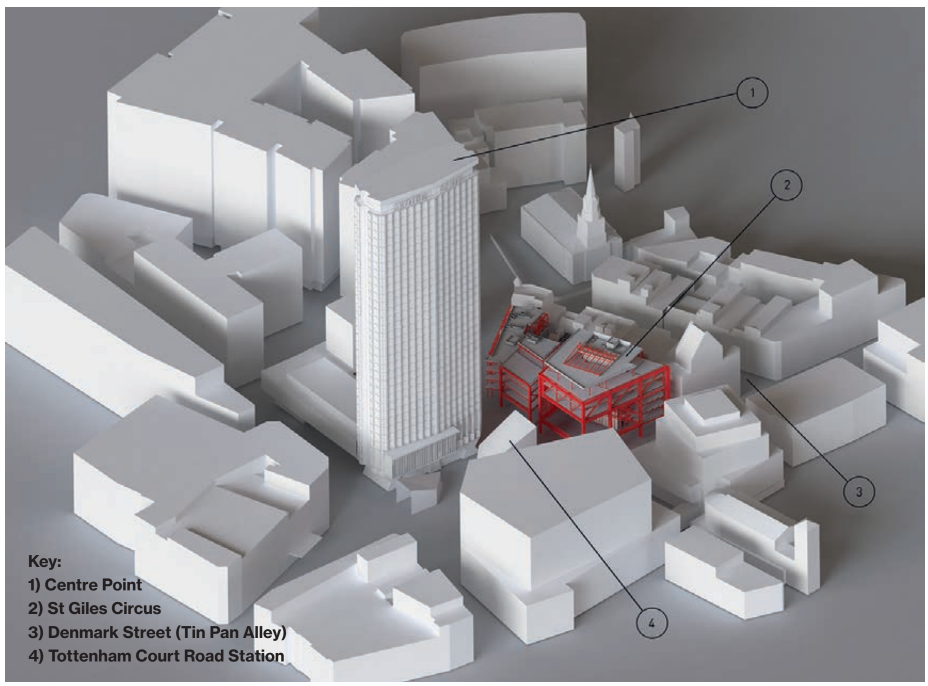


Figure 1
St Giles Circus development adjacent to Tottenham Court Road Station and Centre Point

Site constraints

The St Giles Circus site is in the London Borough of Camden and is bounded by Charing Cross Road to the west, the former Andrew Borde Street and the Centre Point development to the north, St Giles High Street to the east and Denmark Street to the south. The site lies in a Conservation Area and contains a significant number of listed buildings, particularly on the Denmark Street frontage. Denmark Street is known as 'Tin Pan Alley' through its long association with the British music industry, particularly in the 1960s and 1970s, and retains a significant number of independent guitar and music shops.

A key requirement of the project brief was to maintain the character of Denmark Street and keep the independent shops trading

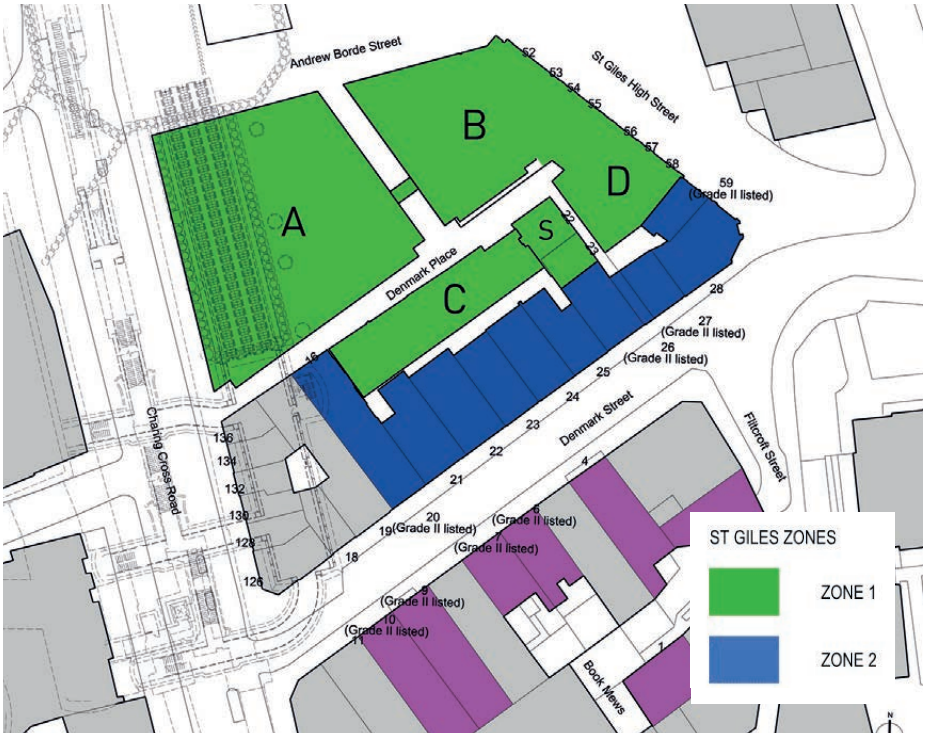


Figure 2
Site plan with listed buildings and extent of Zone 1 and Zone 2

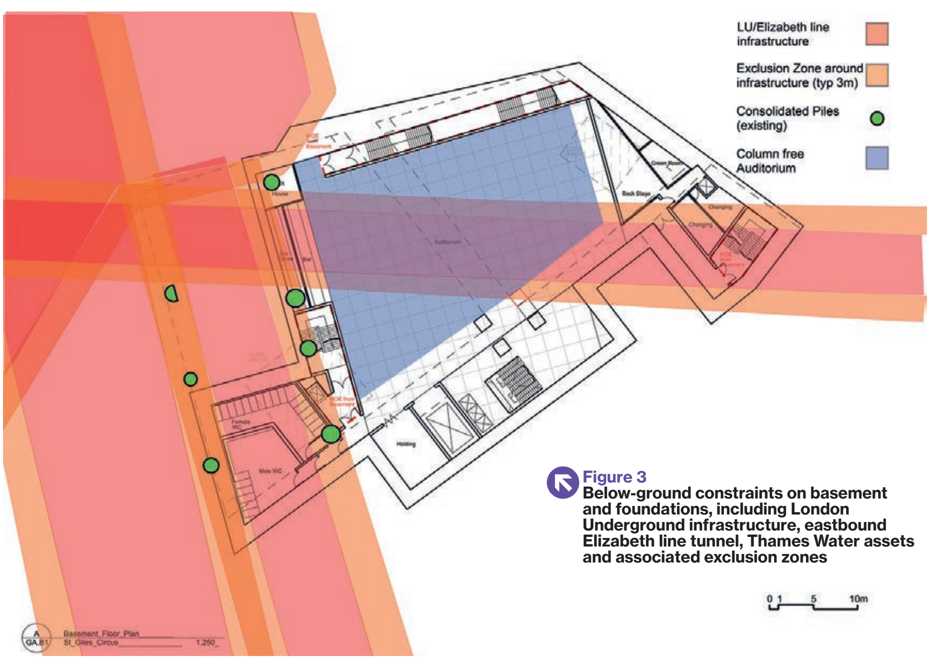


Figure 3
Below-ground constraints on basement and foundations, including London Underground infrastructure, eastbound Elizabeth line tunnel, Thames Water assets and associated exclusion zones

before, during and after the development. For this reason, the development was split into two parts – the new buildings and basement construction in the north and west of the site, known as 'Zone 1'; and the retained and refurbished buildings on Denmark Street and St Giles High Street, known as 'Zone 2'. Figure 2 identifies the site plan, the listed buildings and the extents of Zones 1 and 2.

The footprint of the new development was constrained by the street plan and the existing buildings at ground level, but the size and depth of the new basement – submitted for planning in 2012 – was informed by the underground infrastructure, both existing and proposed.

The site lies immediately to the southeast of Tottenham Court Road underground station, with the Northern line station platforms running under Charing Cross Road immediately to the west of the site. The eastbound Elizabeth line tunnel was due to pass under the site in 2013. As part of the first phase of the Tottenham Court Road Station upgrade works constructed between 2010 and January 2015, a new ticket hall was built adjacent to the site with an escalator connection (known as the Northern line Escalator Box or NLEB) to the Northern line platforms passing under the site. A 2010 Development Agreement between the client and London Underground allowed the latter to lease part of the site during the upgrade works to Tottenham Court Road Station, demolish the buildings over the footprint of the NLEB, divert Charing Cross Road across the site to improve the construction sequencing of the new ticket hall at Tottenham Court Road Station, and construct the NLEB as a cut-and-cover tunnel as opposed to the originally proposed sprayed concrete-lined tunnel.

As part of the Development Agreement, seven deep piles designed by the author while at BuroHappold, were installed around the proposed NLEB to support a future oversite development (OSD). The seven piles – known as the 'Consolidated Piles' after the client – were designed to be at least 1.0m clear from the proposed eastbound Elizabeth line tunnel, and isolated from the NLEB so that the NLEB and the adjacent Northern line London Underground infrastructure did not rely on the piles, and the piles did not rely on the Elizabeth line or London Underground infrastructure. This was important, as any future OSD would require an independent foundation structure if the client was to retain the freehold for the land above the NLEB. The design and construction of the Consolidated Piles, in particular of the 'D-pile', was detailed in the construction press at the time[1,2].

Figure 3 identifies the underground constraints on the site, including the Northern line platform tunnels, the NLEB, the eastbound Elizabeth line tunnel and the Victorian sewers of the surrounding roads. The Consolidated Piles were installed by Bauer in 2010, with the NLEB temporary works secant piled wall, as part of the Vinci BAM Nuttall upgrade works to Tottenham Court Road Station.

Development of planning application

Engenuiti was appointed in 2011 by Consolidated Developments to prepare structural and civil engineering designs

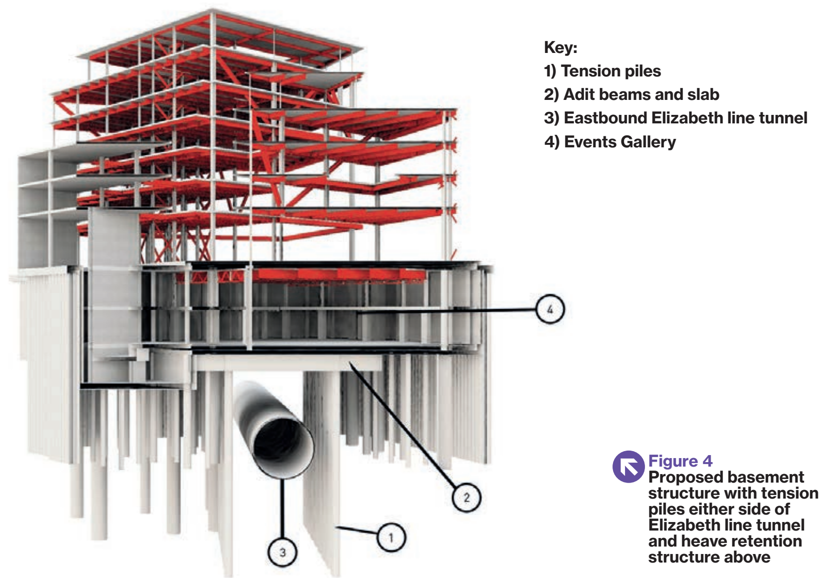


Figure 4 Proposed basement structure with tension piles either side of Elizabeth line tunnel and heave retention structure above

and technical submissions in support of a planning application for the St Giles Circus development. With the client's brief for as large a basement as possible, it was clear that this could have a significant interface with the eastbound Elizabeth line tunnel that was due to be constructed under the site. The depth of the basement would be limited by the Development Agreement with Crossrail Ltd which required 6.0m clearance between the tunnel crown and any basement or foundation structure above. This would limit the depth of the basement above the Elizabeth line tunnel to approx. 11m.

Removal of almost two-thirds of the overburden above the proposed tunnel would result in significant heave of the tunnel. Early liaison with Crossrail's Third Party Development Manager confirmed that the movement limits for the tunnel would be tight to control track distortion, opening up of the joints in the segmental concrete tunnel lining and, in particular, 'egging' or 'squatting' of the tunnel which, if excessive, had potential to exceed the kinematic allowances between the rolling stock and the overhead line equipment (OLE) for the Elizabeth line. Details of the constraints were provided by the guidance in Crossrail's 'Information for Developers Pack'[3]. Engenuiti engaged Donaldson Associates (now part of COWI) to provide additional geotechnical services, including ground movement modelling, to the design team. Early two-dimensional (2D) analysis showed that simply removing the overburden above the tunnel could result in 40mm of heave at the tunnel crown and 20mm of egging of the tunnel, significantly beyond the 10mm limit identified in the Crossrail guidance.

In order to control the predicted movements, the team developed a heave retention system to effectively 'staple' the tunnel down. This would use a combination of deep tension piles either side of the tunnel to resist the heave, and a system of beams and slabs between the tension piles to transfer the heave forces to the tension piles. This arrangement is shown in Figure 4.

The heave retention system was modelled in Plaxis[4] and results compared with a simple hand calculation assuming uniform skin friction on the tension pile, axial pile stiffness based on a cracked pile section (i.e. tension reinforcement only) and bending theory for the beams and slabs to transfer a heave load equivalent to the weight of overburden removed. Both systems showed that the predicted heave movements could be more than halved by the proposed retention system, depending on the parameters such as pile depth and diameter, amount of tension reinforcement, spacing between the piles and stiffness of the beams and slab. So, the movement criteria could be met in theory, but could the system be constructed without having to remove the overburden first to construct the retention slab?

A 'brain storming' workshop was held with Hilary Skinner and Vicky Potts from Donaldson Associates in early 2012. Hilary recalled a technique used during the construction of the Westminster Station upgrade as part of the Jubilee line works. The District and Circle lines needed to remain operational during construction of the new station box below, but there was very limited headroom available to construct a new transfer structure to support the lines above the box. A series of headings supported by timber props ('timber headings') were tunnelled under the District and Circle lines in a hit-and-miss arrangement to enable the insertion of a new transfer structure below the operational railway. By using a similar technique at St Giles Circus to construct the heave retention slab above the Elizabeth line tunnel, it would be possible to insert the slab while maintaining the overburden above.

It was agreed that the tension piles could be installed from ground level and form a cut-off wall either side of the tunnel. This would isolate the area above the Elizabeth line tunnel, allowing the basement construction to be split into three distinct areas: north of the tunnel, south of the tunnel and above the tunnel. The basement areas north and south of the tunnel could then be excavated in advance of the area above the tunnel.

A system of horizontal props and waling beams would be used around the perimeter of each basement to control lateral deflection of the wall and the Elizabeth line tunnel. Timber headings or 'adits' would then be advanced from the formation level through coordinated soft spots in the tension piled wall, reinforcement would be placed in the adits and concrete cast to form 'adit beams' anchored to the tension piles that would be capable of limiting the heave that would result from excavating the overburden above the adit beams. Figure 5 shows the proposed construction sequence and arrangement of the adit beams.

A key consideration in the design of the adit beams was the access requirements for the miners that would advance the timber headings; advice was taken from Joseph Gallagher Ltd who specialise in this type of work, often for London Underground. An adit beam size of 1200mm wide × 1800mm high was decided upon, with a concrete blinded base to minimise any softening of the base during construction.

During Stage 2, a parametric study considered the effect of varying the span and spacing of the adit beams along the tunnel. It was found that the tunnel was more sensitive to the span of the adit beams than the spacing, as the effect of the span was not only influenced by the deflection of the adit beams, but also the interaction between the tension piles and the tunnel: the shorter the distance between the tunnel and the tension piles, the more restraint that the tension piles would provide to the tunnel. The results of the analysis were regularly discussed with Crossrail, as constructing the piles closer to the tunnel presented its own risks of movement or ground failure during pile construction.

The depth of the basement excavation, large event space in the basement and modest size of the above-ground development meant that removal of the overburden above the tunnel was much more significant than any gravity loads that could be applied by the new structure.

The Development Agreement between the client and London Underground allowed for piles to be constructed just 1.0m clear from the proposed Elizabeth line tunnel adjacent to the NLEB, as the position of the tunnel would be constrained by the NLEB, the Northern line platform tunnels and access shafts to the Northern line[5]. The Development Agreement allowed for the clearance to taper from 1.0m to 3.0m as the tunnel progressed east across the site and moved away from the constraints of the NLEB, in order to give Crossrail some flexibility in the final alignment.

As it was now clear that the Elizabeth line tunnel would be constructed before the St Giles Circus development, Crossrail imposed a limitation that the piles must be at least one pile diameter clear of the tunnels and that the detailed geotechnical site investigation must investigate the risk of sand lenses in the London clay around the tunnel, as these could cause collapse of the pile bore during construction.

A conceptual design statement was prepared for Crossrail and submitted with the planning application in December 2012. The engineering risk assessment in the statement concluded that the best balance between extent of tunnel movement, risk of pile bore collapse and cost of construction would be met by constructing piles up to 1200mm diameter under bentonite at a distance of 2.0m plus construction tolerance. An adit spacing of 3000mm was selected so that the remaining overburden could 'arch' between the adit beams in the temporary case. This incorporated 2D assessments of ground movements and their effect on the Crossrail tunnel, and preliminary structural calculations for the heave retention system. A basement impact assessment was also prepared for the planning submission to the London Borough of Camden.

With the basement only 6m above the eastbound Elizabeth line tunnel and only slightly further away from the Northern line platform tunnels at Tottenham Court Road, the acoustic performance of the basement space was also a key consideration. The main performance space or 'Events Gallery' in the basement will be used for a wide variety of functions, from musical rehearsals and performances, corporate functions and product launches, to club nights and gigs. As a result, there was a need to provide acoustic separation from train noise and vibration breaking into the Events Gallery, but also from performance noise breaking out to other spaces within the development.

From an early stage, it was agreed that a 'box-in-box' solution would be the most effective method of isolating the Events Gallery from other sources of noise and vibration. This would entail a double structure around the Events Gallery, with the outer walls, floors and roof structures supported by the building's foundations and an inner structure separated from the outer structure by an air void at least 70mm wide.

While the walls and roof structures could be completely separate, the weight of the inner box structure would need to be supported by acoustic bearings sitting on the main basement floor level. Engenuiti worked with the acoustics team at BuroHappold to develop the performance requirements of the acoustic bearings so that they would tune out the dominant vibration frequencies from the trains passing around the site, but would have a natural frequency distinct from possible sources of excitation such as footsteps and dancing. A natural frequency of approx. 9.5–11.5Hz was selected for the bearings.

As a result of feedback from the planners, there was a requirement to re-site a significant amount of building services plant that had previously been located at roof level. This resulted in a need for more basement space to house the plant. As the site footprint had already been maximised, site constraints meant that the only way to do this would be to excavate deeper either side of the Elizabeth line tunnel to form a new level of basement below the main Events Gallery, effectively forming a saddle over the Elizabeth line tunnel. This basement space would improve construction access to the proposed adit beams.

Camden's planning committee approved the St Giles Circus scheme in November 2013; the design team was then appointed to move the scheme forward in early 2014.

Collaborative approach to value engineering

The design of the heave retention structure had been developed to support the planning application in advance of the Elizabeth line tunnel construction. In the time between the submission of the planning application and the end of the Stage 4 design, the Elizabeth line tunnels had been built from Paddington

Figure 5
Proposed basement construction sequence with adit beams to restrain heave during construction

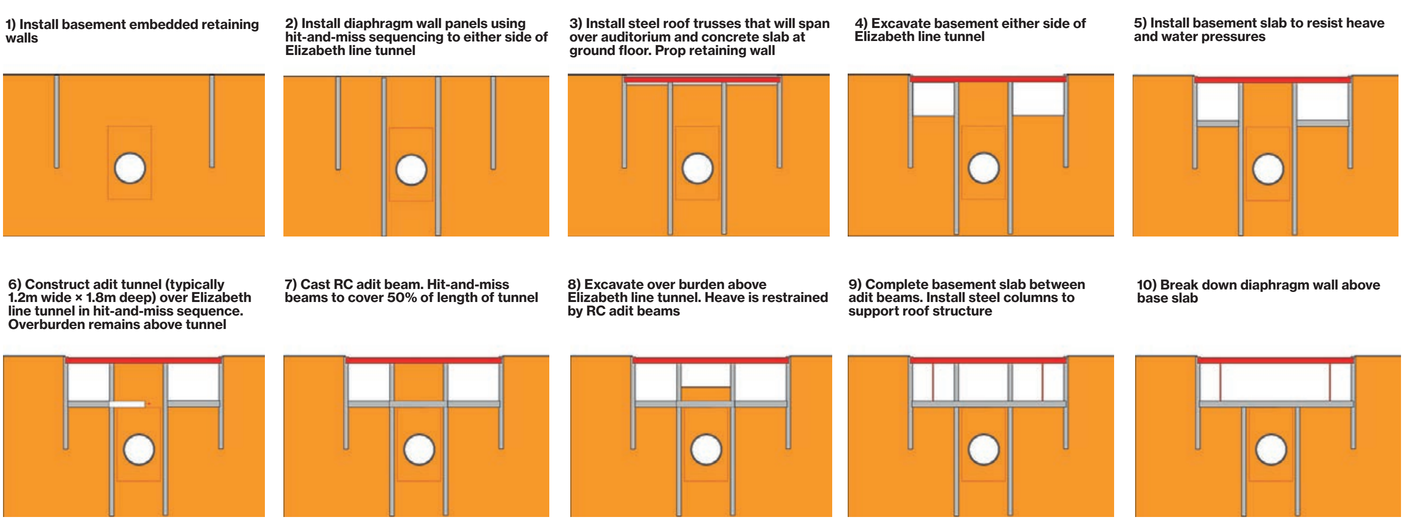


TABLE 1: COMPARISON OF PREDICTED TUNNEL MOVEMENTS

Location	Short-term movements (mm)				Long-term movements (mm)			
	Original scheme	Arup 2D feasibility	Donaldson 2D verification	Donaldson 3D verification	Original scheme	Arup 2D feasibility	Donaldson 2D verification	Donaldson 3D verification
Tunnel invert	6	14	13	10	8	22	19	14
Tunnel crown	10	24	21	19	12	36	30	25
Ovalisation	4	10	8	9	4	14	11	11
Adit slab/beam	Not available	Not available	38	39	Not available	Not available	46	47

to Stratford and the station box excavated above and around the Elizabeth line tunnel at Paddington.

In many ways, the excavation of the station box at Paddington had similarities to the proposed construction at St Giles Circus. On both sites the Elizabeth line tunnels were constructed in advance of the new basement structures; both basements would be constructed using top-down construction within embedded retaining walls (diaphragm walls at Paddington, secant piled walls at St Giles).

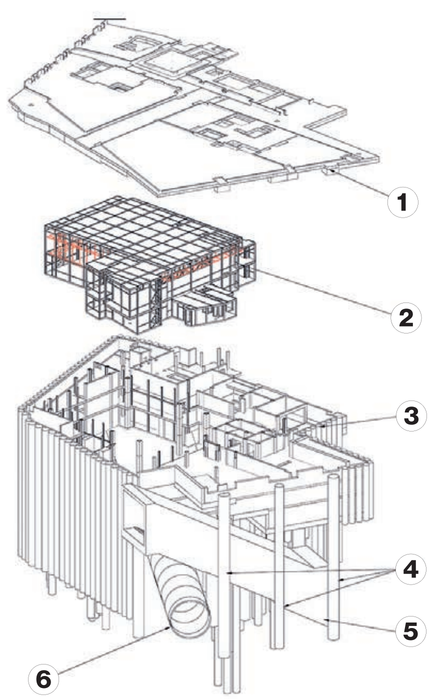
The tunnel crown was approximately the same depth below ground level on both sites; the tunnel construction was the same on both sites with the tunnel remaining unpropped during the excavation; both sites proposed to use tension piles to control the heave; and the geology on both sites was similar, with made ground overlying river terrace deposits, overlying the London clay through which both tunnels passed.

At Paddington, the excavation then continued to expose the tunnel crown and break out the bored tunnels to form the station box; therefore, no adit slab was required as the tunnel no longer existed.

Collaboration with Arup, the structural and geotechnical engineers for the Elizabeth line tunnel and the station box at Paddington, enabled the learning and real movement data from constructing and excavating the station box at Paddington to be used to inform the design and construction methodology for the St Giles Circus development. Results from the excavation at Paddington showed that the short-term tunnel movements for an excavation of similar depth to that proposed at St Giles Circus would be in the order of 15mm at the crown and 5mm at the invert, resulting in a net ovalisation of approx. 10mm.

Although the basement depths, tunnel depths and ground conditions at Paddington and St Giles Circus were not exactly the same, if similar movements could be shown to apply at St Giles Circus, it might be possible to amend the construction

Figure 6
Isometric of basement arrangement showing interfaces with Elizabeth line and London Underground infrastructure



Key:
1) Ground-floor reinforced concrete slab
2) Box-in-box structure steel frame with metal deck composite concrete floor and roof slab
3) Main basement box reinforced concrete frame
4) Consolidated Piles
5) NLEB
6) Eastbound Elizabeth line tunnel

sequence to a conventional top-down methodology removing the adit beam construction, generating both cost and programme savings, and crucially removing a significant health and safety risk of excavating an extensive adit system in confined spaces.

Arup was instructed to undertake a feasibility study that post-rationalised the predicted movements of the Paddington Station box, so that confidence could be gained that the proposed analysis techniques for the St Giles Circus development would, when applied to the Paddington Station box, correlate with the actual movement recorded at Paddington. Once this had been done, Arup developed a number of value engineering options using Plaxis 2D that considered the effect of varying the stiffness of the adit slab and heave retention piles at St Giles Circus, with the aim of limiting the movement of the track slab and ovalisation of the tunnel to criteria agreed with Crossrail.

The alternative construction sequence was presented to Crossrail and agreed in principle. The 2D analysis results were then verified by Donaldson Associates using Plaxis 3D analysis[6] so that the effect of the complicated basement form on the tunnel movements could be assessed. Table 1 compares the predicted tunnel movements for the original scheme with the Arup feasibility study and the Donaldson 3D analysis.

In view of the magnitude of the predicted movements of the adit slab, the decision was taken to increase the thickness of the slab from 1.0m to 1.1m after the Arup feasibility study. This accounts for the slightly smaller movements predicted by Donaldson.

Given the close proximity of the basement development to the Elizabeth line tunnel, Crossrail required that an independent Category III check was undertaken to verify the movement and structural capacity of the tunnel. This was undertaken by A-squared Studio using the Plaxis 3D finite-element modelling package to replicate the construction sequence. Similar results were obtained to the Donaldson models.

The resulting basement arrangement is shown in Figure 6.

The Smithy

At the rear of the listed building at 26 Denmark Street were two smaller buildings: a three-storey Victorian warehouse known as 23 Denmark Place and a single-storey 18th century former blacksmith's forge known as the 'Smithy'. As a rare survivor of this type of building in central London, the

Smithy is of local historic significance and adds character to the area.

These buildings are located above the Elizabeth line tunnel and the original plan was to leave the buildings in place so that the basement would not encroach on the footprint of the buildings. This presented a challenge, because the proposed Elizabeth line heave retention scheme was to pass under the Smithy in order to maintain a consistent interface with the tunnel over the length of the site. During the Stage 3 design phase, ground movement analysis showed that the original arrangement would potentially result in significant damage to the existing Smithy building. Alternative options were therefore considered, assessed and presented to Crossrail (Table 2).

It was agreed that the most favourable option for both protection of the Elizabeth line tunnel and safely constructing the works was to move the Smithy. By de-coupling the Smithy from the other buildings at 26 Denmark Street, the existing structures would either be entirely supported on the new piled basement structure or on their existing spread footings, reducing the risk of differential movement between the piled basement and the existing footings adversely affecting the buildings. The building at 23 Denmark Place was considered to be of limited architectural merit in the context of St Giles, and efforts were therefore focused on finding a way to safeguard the Smithy building. As this would require the demolition of 23 Denmark Place, approval was obtained from the conservation officer at Camden Council and Historic England before progressing with the

TABLE 2: ALTERNATIVE OPTIONS CONSIDERED FOR SMITHY

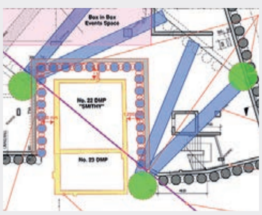
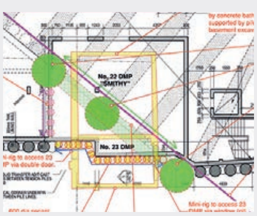
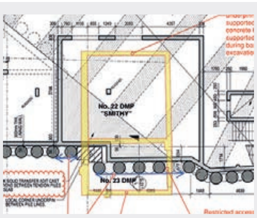
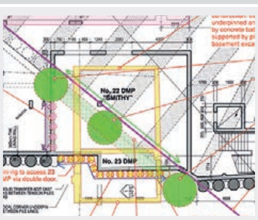
Options	Construction implications	Effect on Elizabeth line	Health and safety implications	Effect on Smithy
Piling around Smithy, no basement under				
	Simple conventional construction, but complicated by skewed adit beams	Poor as heave retention system would be incomplete around footprint of Smithy	Potential movement of Elizabeth line tunnel	Ground movement assessment showed potential for structural damage to Smithy due to both pile installation and basement excavation
Pile within Smithy footprint to maintain consistent heave retention system, either no basement under or pile within 23 Denmark Place to extend basement				
	Would require either small caisson construction or specialist piling techniques (e.g. Martello) to work in tight space of Smithy – slow and expensive. Adit beams would need to be constructed from one end	Good as heave retention system would be more consistent over site footprint	Working in tight spaces with large machinery potentially dangerous. Risk of collapse of Smithy due to impact or vibration. Small caisson construction has significant safety implications	Risk of damage to Smithy during construction works. Smithy roof likely to require temporary removal during pile/ caisson construction
Pile within 23 Denmark Place and construct 'super adit' to get similar heave retention system to rest of site. Support Smithy on piles, extend basement under Smithy				
	Would either require demolition of 23 Denmark Place or piling within the existing building. Smithy would need to sit on transfer structure. Requires additional adit construction	More consistent heave retention structure, but longer spans result in some additional movement	Working in tight spaces with large machinery potentially dangerous unless No. 23 demolished. Additional adit construction undesirable	Reduced risk of damage to Smithy as it will be supported on piles, but likely to require demolition of No. 23 either to facilitate pile access or as result of differential movement between basement and spread footings
Move Smithy to enable piling either side of Elizabeth line tunnel				
	New transfer structure required under Smithy to enable move. Large crane for move. Conventional piling and adit beam construction. Programme and logistics of move need to be carefully planned	Good as heave retention system consistent across site footprint	Conventional underpinning and large lift, but no working in confined spaces or increased risk of tunnel movement	Requires 23 Denmark Place to be dismantled, but reduced risk of damage to Smithy fabric than other options

Figure 7
Smithy move in August 2016 by H. Smith and Abbey Pynford as part of enabling works

structural option preferred by Crossrail.

Once the principle of moving the Smithy was agreed, options for the logistics of the move were developed, such as: lift with crane or slide on skate; support on concrete base slab or steel grillage. In order to minimise the programme implications of the move, it would be beneficial to undertake the first move as part of the demolition and enabling works package that was let to H. Smith. The logistics of the move were developed with input from Skanska and specialist subcontractor Abbey Pynford so that the agreed solution would have minimal impact on both the enabling works and the permanent works.

Sliding the Smithy was quickly discounted as it would require significant excavation and temporary works to move the Smithy horizontally without the need for a vertical lift, and any crane required for a vertical lift would be able to move the Smithy the short distance required to install the piles. Initially, inserting a steel grillage below the Smithy was the favoured lifting option as it would reduce the total lifting load compared to a concrete base slab, but there were concerns about the stiffness of the steel grillage and transfer of the Smithy from its current foundations onto the grillage and again from the grillage onto the permanent works.

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Figure 8
Smithy returned to original position in October 2017 by Skanska and Mammoet

The concrete base slab could be integrated into the ground-floor permanent works, avoiding a double transfer. Abbey Pynford's assessment of the total lifting load showed that it was viable to lift the Smithy and the base slab together and (just) fit the crane on the site, so the concrete option was adopted. Figure 7 shows the Smithy move from its original location in August 2016 and Figure 8 shows the return trip by Mammoet in October 2017.

Construction methodology

As can be seen from the preceding sections, the key to the success of the St Giles Circus project was the control of ground movements and the protection of adjacent infrastructure during the basement construction phase.

Top-down construction was adopted for the basement with the ground floor and basement mezzanine slabs providing horizontal props to the retaining wall during the basement excavation. The key benefit of top-down construction was the reduced risk of ground movements adversely affecting the adjacent buildings on Denmark Street, the London Underground infrastructure and the Thames Water infrastructure. However, the constrained nature of the site and the complicated basement construction meant that top-down construction also provided more lay-down and working space at ground-floor level, improving the logistics of deliveries to the site and muck away, particularly from the Charing Cross Road entrance. Top-down construction would also release the superstructure construction earlier, which had major programme benefits as the duration of the basement excavation, construction and box-in-box installation was of similar duration to the above-ground works.

The adoption of the top-down method required significant temporary works in order to support the construction over both the Elizabeth line tunnel and the NLEB. With limited locations available on the site footprint for deep foundations, it was not possible to install a regular grillage of plunge columns. Long spans over the Events Gallery required significant permanent steel transfer beams at ground-floor level to support the structures above. These were supplemented by temporary transfer beams over the NLEB and below building D (Fig. 2) to support the ground floor until the permanent vertical support provided by the Consolidated Piles around the NLEB and the raft slab below building D could be mobilised.

Construction advice from Skanska required the load capacity of the ground-floor structure to be enhanced to a minimum of 20kPa to support the logistics of deliveries and muck away; in some areas this would need to be enhanced further to 110kPa to support mobile cranes required for the erection of the steel-framed superstructure.

The steel transfer beams and ground-floor slab below building D will be supported by the basement perimeter piled wall during excavation down to basement mezzanine level; however, the piled wall in this location lies above the Elizabeth line tunnel, with the pile toes at least 6m above the tunnel crown. Therefore, an alternative method of support is required before the basement excavation progresses below basement mezzanine level and exposes the lower part of the piled wall. Early construction of the liner walls between basement mezzanine and ground-floor level enables the lining walls to act as deep transfer beams that support the ground-floor and basement mezzanine slabs over the Elizabeth line tunnel until the main basement raft slab above the tunnel is completed. For this reason, the superstructure works for building D will not be able to commence until the main basement raft slab is completed.

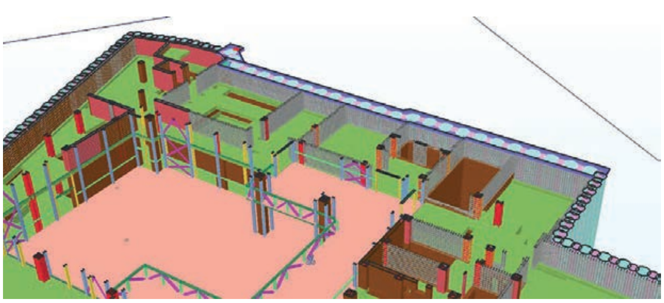

Figure 9
BIM model for production of reinforcement detailing for basement

A key hold point for the basement construction is the completion of the basement mezzanine slab and the associated horizontal propping that is required to restrain the perimeter piled wall. Until this is complete, the excavation may not progress below basement mezzanine level. The majority of the propping at this level is provided by the permanent concrete slab; however, the need to maximise the footprint of the Events Gallery in the permanent condition resulted in some pinch points where insufficient slab was available to act as a complete whaling or ring beam around the perimeter of the excavation.

In these locations, Skanska's temporary works engineers designed a system of temporary props to complete the ring beam in the temporary case. The stiffness of the steel/concrete propping system was independently assessed by both Skanska and Engenuiti, with figures agreed prior to submission to Cementation, which was responsible for the final design of the piles and piled wall. The complex form of the basement resulted in the need for 11 different retaining wall analysis models, each with either a different section or construction sequence. The upper and lower bound results from the retaining wall analysis were then fed back into the propping models to check the stiffness assumptions, deflections and stresses in the propping system.

The complexity of the basement and superstructure arrangements led Engenuiti to adopt 3D modelling for the project from the start of Stage 3 design in 2014. This was invaluable for explaining the project to Crossrail and the other stakeholders and was key to the tenderers understanding the construction sequence. Building Information Modelling (BIM) Level 2 was then adopted for the project after the appointment of Skanska as main contractor and has proved invaluable in the development and coordination of the subcontractor design information, as shown in the production of the reinforcement detailing for the basement (Figure 9).

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Figure 11
Site photograph taken on 18 April 2018 showing progress on site

Figure 10
Model image of St Giles Circus basement and structure

Conclusions

St Giles Circus was already a complex and constrained central London site even before the infrastructure improvements associated with the Tottenham Court Road Station upgrade and Elizabeth line construction began. However, without the infrastructure capacity improvements that the Elizabeth line brings to the area, the planners would not have permitted new developments that increase the number of people working and socialising in this part of London, and the area would gradually stagnate.

By adopting a proactive and collaborative approach with the infrastructure protection team at Crossrail, it was possible to develop and agree technical solutions that maximised the potential development and value on the site, provide solutions that were mutually beneficial, and protect the heritage of the site. Continually questioning assumptions, seeking knowledge and experience in the supply chain, through precedents and through specialist consultants, was fundamental to the success of the project.

The project would not be in its current form without the knowledge of Donaldson Associates and the experiences from the construction of the Paddington Station box that Arup shared. The input of Joseph Gallagher helped to secure planning approval, Abbey Pynford facilitated the Smithy lift and Cementation fine-tuned the construction methodology of the deep bored piles and plunge columns (Figure 10). Underpinning all of this was the early involvement of the main contractor, Skanska, through a two-stage design-and-build procurement route and the continuity of the professional structural engineering team from planning through detailed design, procurement, contractor design and construction.

At the time of writing (April 2018), the project is progressing well on site (Figure 11): the plunge columns and piled foundations have been installed; the Smithy has been moved back to its original position and is supported by the temporary works steel beams; the ground-floor slab is almost complete; preparations are being made to commence the bulk dig. The next few months promise to be an interesting and exciting time.

Project team

Structural and civil engineer: Engenuiti
Client: Consolidated Developments
Architect: ORMS
Building services engineer and acoustician: BuroHappold
Project manager: GVA Second London Wall
Geotechnical engineer: Donaldson Associates (now part of COWI)
Peer review: Arup
Category III checking engineer: A2 Studio
Demolition and enabling works contractor: H Smith
Main contractor: Skanska
Piling subcontractor: Cementation
Basement and concrete subcontractor: Carey's
Steel fabricator: Severfield

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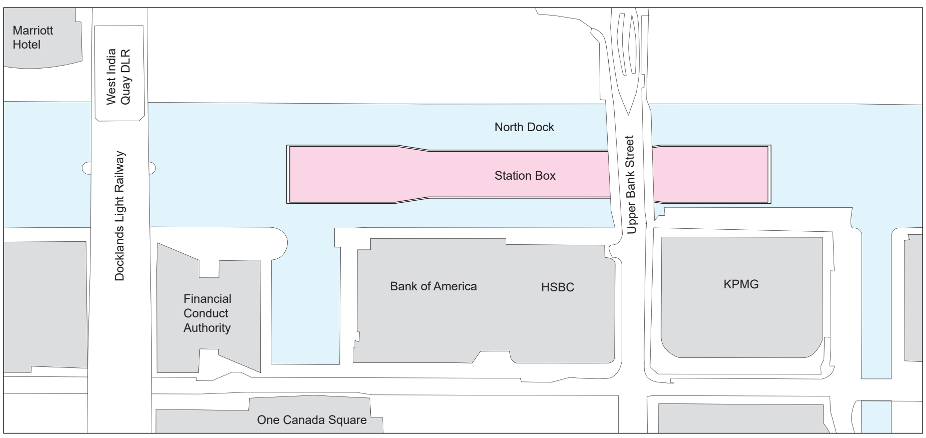
Design of Canary Wharf Elizabeth line station and Crossrail Place oversite development

Tim Worsfold
MEng, ACGI, CEng, MICE
Associate Director, Arup, London, UK

Michael Bryant
BSc, CEng, MICE
Operations Executive, Canary Wharf Group, London, UK

John Crack
BSc, CEng, MICE, MIStructE, MSM, MBA
Associate Director, Canary Wharf Group, London, UK



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NOTATION

ETFE	**ethylene tetrafluoroethylene**
MEWP	**mobile elevating work platform**
OSD	**oversite development**

Figure 1
Overall plan showing proximity of Canary Wharf station to major neighbouring buildings

Figure 2
View from West India Quay station of almost-complete station and retail superstructure, showing proximity of tall office buildings

Introduction

Canary Wharf was the first station on the Elizabeth line to be constructed, and the first to be let as a design-and-build contract, with developer Canary Wharf Group. Innovative design and construction techniques enabled the station box to be completed four months ahead of the development programme. Construction of the Crossrail Place retail and leisure oversite development (OSD) proceeded concurrently with that of station. The OSD included a number of features aimed at increasing future flexibility for the developer and tenants. A timber gridshell roof completes the development, partially covering a large roof garden that is open to the public. The OSD opened in May 2015, nearly four years ahead of the planned station opening.

Canary Wharf station

The station site is located on the north side of the Isle of Dogs in the London Borough of Tower Hamlets, and is within the West India Dock (Figure 1). The dock was decommissioned in the 1970s and is no longer

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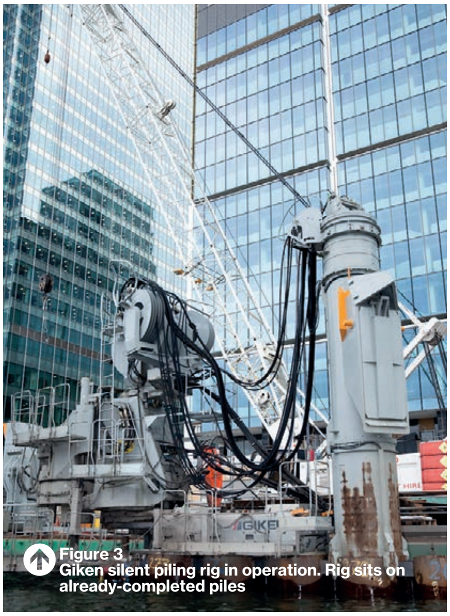


Figure 3
Giken silent piling rig in operation. Rig sits on already-completed piles

ARUP

Figure 5
Level –4 being prepared for reinforcement placement and concreting. This is first top-down level, formed by moling beneath Level –3 slab above

ARUP

in use as a commercial port. The station box is 260m long and 25–30m wide. The dock water is 9m deep and the station base slab is approx. 18m below dock bed level. The station box sits within 10m of four existing office buildings which are up to 40 storeys high (Figure 2).

Station box design

The developer was a leading advocate for the introduction of the southeast spur of the Elizabeth line, which included a station at Canary Wharf. The developer and designer worked strenuously to minimise the environmental impact of the station construction on the Canary Wharf area and maximise the value the project would bring.

The early reference designs developed by Crossrail Ltd had a station box well over 300m long and involved fully or partially filling the dock.

With over 20 years of local knowledge in the design and construction of more than 30 buildings in Canary Wharf, the developer and designer were able to develop several schemes to construct the station box within a drained coffer dam without the need to import material to fill or partially fill the dock.

Relative to the reference design, the depth of the station box was reduced to minimise impact on the adjacent buildings and the centreline was relocated to mimise effects on the Docklands Light Railway viaduct (Fig. 1). The developer and its technical advisers worked to reduce the cost of the station while maintaining functionality: this included reducing the length of the station box to approx. 260m; the width of the box was also reduced over the central section. Access to future developments to the north and existing developments to the south was explored. The space above the box was also developed as a retail area.

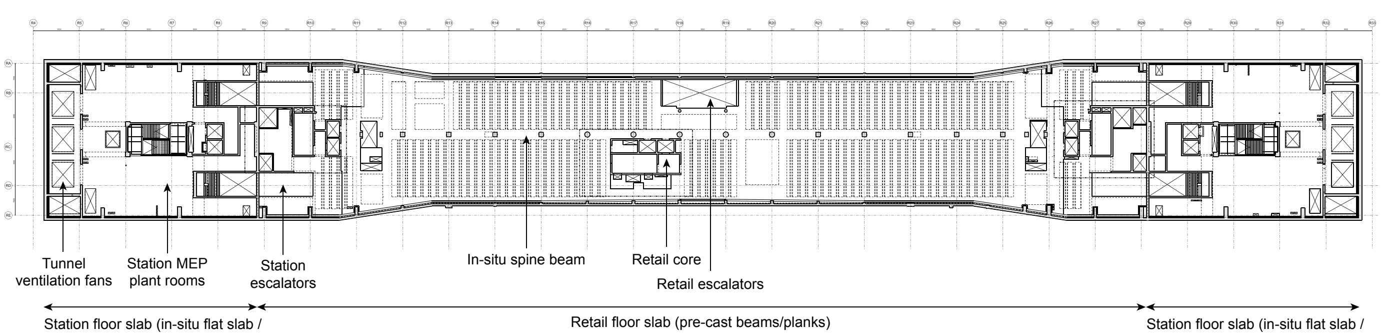
Station configuration

The north, east and west retaining walls were formed from 310 Giken tubular piles of 1214mm diameter, installed using two Giken silent pilers (Figure 3). Reinforced concrete piles were formed by boring through the bottom of the tubular piles and the whole pile was then reinforced and concreted: the detailed analysis, construction methodology and monitoring is explained in Yeow *et al*[1]. The southern station retaining wall was formed by a line of 1180mm diameter hard-firm secant piles at 875mm centres from the dock bed. This saved a year on the construction programme from Crossrail's original proposal. To allow for flexibility in the future OSD, there are 105 tension piles of 1200mm diameter along the station; these are spread across the base slab footprint and are for the permanent groundwater case only. In addition, 30 plunge-column piles of 2100mm diameter facilitated the top-down construction.

Station structure

The station structure is a mixture of *in situ* reinforced concrete and precast concrete (Figure 4). The station features an offset central line of columns which extend through

Figure 4
Example of below-ground floor plan



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the building and are founded on the base slab at Level –6. These columns support a central beam running the full length of the station. At Level –4 this beam is designed to span twice its normal span under accidental load conditions, in case a column below should be lost due to a train derailment. Longitudinal beams also run along the north and south sides of each floor.

Level –3 was the first level to be cast as part of the top-down sequence, and a beam and slab floor structure was adopted, with moling holes incorporated for excavation access below. Level –4 is a solid slab spanning onto the central spine beam (Figure 5).

At Levels –2 and –1, connecting the three longitudinal beams are precast beams at approx. 1.5m centres. Precast lattice planks span between the beams. An *in situ* concrete topping connects the precast elements and completes each floor diaphragm. This arrangement reduced the amount of formwork needed.

Level –1 completes the box structure, with horizontal forces from earth and water pressure being resolved through this level. In some ways, the building can be thought of as a concrete ship, with the Level –1 to –6 substructure being the 'hull' and Levels 0 to +3 being the 'superstructure' (Figure 6). Without the superstructure, the high surrounding water levels and relatively large air volumes inside the substructure would mean that tension piles are needed to hold down the box in the dock.

The longitudinal elevation of the station shows that, in common with tunnelled/mined Elizabeth line stations, the station areas are confined to the platform spaces, plant rooms at each end, and station entrances. Considerable space is available below water level for retail at Levels –2 and –3, as well as at Level –1 between the entrances (Figure 7).

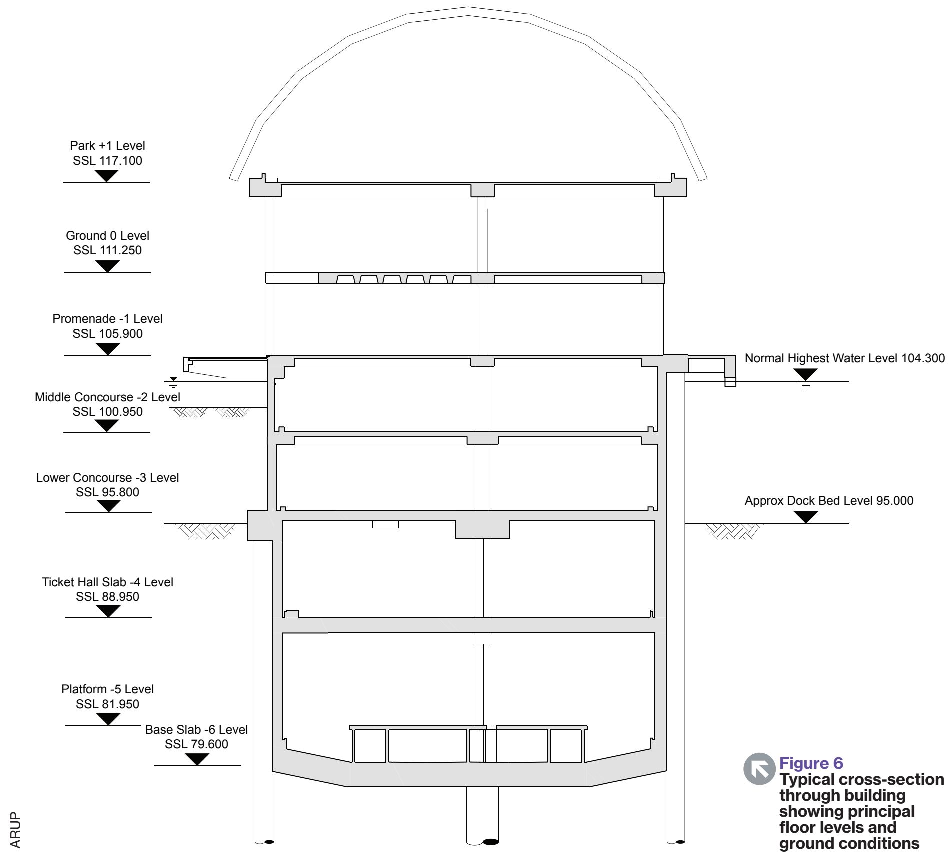

Figure 6
Typical cross-section through building showing principal floor levels and ground conditions

Relationship between station and OSD

The development agreement between Canary Wharf Group, the Secretary of State and Crossrail required Canary Wharf to construct the station within a stated time frame. However, it also allowed the developer the freedom to develop the OSD at any time to suit market conditions. The long programme duration for Crossrail, coupled with the market uncertainty that followed the 2008 financial crisis, meant that a solution had to be developed whereby the station design and construction could proceed without Canary Wharf having to commit to a particular OSD scheme or, for that matter, any scheme at all.

The station design and construction therefore proceeded on the basis of:

- only station entrances and vent shafts being constructed above water level; below-water retail areas being mothballed; various concepts being developed for what to place between the entrances; structure and services being arranged to enable the OSD at an unknown time in the future
- the OSD being developed concurrently with the station box (a retail scheme received planning consent from the London Borough of Tower Hamlets).

Figure 7
Longitudinal section through station and retail development

The station box structure featured concrete buttress walls up to 1.7m long at regular intervals along each longitudinal wall, upon which the superstructure columns are supported. During design development of the OSD, and after the station structure was substantially complete, the location of these columns was adjusted to enable up to 30% more retail floor area to be accommodated.

This revised scheme required a revised planning application to be submitted; therefore, the station structure and starter bars at Level –1 were configured and positioned to enable any of the 'no OSD', 'original OSD' and 'revised OSD' schemes to proceed.

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Figure 8
Completed park level with clearly visible roof openings to admit air and light to garden. Path is formed on concrete slab that floats over soil beneath, allowing plant roots to extend across roof

Crossrail Place

The constructed OSD is known as Crossrail Place and consists of retail space within the station box at Levels –3 and –2 and on top of the station box at Levels –1, 0, +1 and +2, with plant rooms at Levels +2 and +3. There is a large roof garden at Level +1. The entire building is capped with a timber gridshell and an ethylene tetrafluoroethylene (ETFE) pillow roof which is 310m long (Figure 8).

The OSD is supported by the station structure below. Building services are entirely independent, enabling each to operate with or without the other.

Structurally, the OSD consists primarily of a reinforced concrete frame with a grid of up to 13.5m × 9m. While the station box structure is continuous, the superstructure is split into three separate buildings via movement joints at 90m centres. However, the roof is continuous. Stability is provided by reinforced concrete cores that transfer horizontal loads to the box below; these cores terminate at the lowest retail level (Level –3), where they are transferred vertically and horizontally.

The structure was designed to provide as much flexibility as possible for both tenants and the developer, while also being cost-effective. At the time of design, there was a programme and cost incentive to construct the OSD concurrently with the station box, and this required the optimised (wider grid) OSD design to be developed as quickly as possible.

Various floor configurations were studied and it was found that the following scheme achieved the best mixture of flexibility (Figure 9), design/construction speed and cost-effectiveness:

- An *in situ* reinforced concrete structure was used for Levels 0 and +1, featuring longitudinal primary beams and a transverse ribbed-slab structure.
- Removable panels were incorporated into the floor in various places to enable future escalators to be added without major structural work being needed. These panels consisted of precast planks supported by nibs.
- Extensive 'soft spot' zones were incorporated into the floor design to enable future slab removal to create a double-height space, or lift/escalator openings for multistorey combined retail units.
- Most retail entrances were provided with double-height space by default; however, the tenant had the option to infill the space. Steel cast-in plates to the surrounding columns were provided to enable this to be done without needing major structural work.
- The primary structure was designed to work without the presence of surrounding Level 0 floor areas for restraint, enabling large areas of Level 0 to be removed to create a double-height space if desired.

The mixture of retail or restaurant (or other) tenants was not known at the design stage. While this had relatively limited implications for the structure, it made building services design more difficult due to the much more onerous mechanical and public health requirements for restaurant and leisure use in particular. To address this, a retail/restaurant mix assumption was agreed with Canary Wharf Group. The design did not cater for restaurant use below Level –1.

During tenant occupation and fit-out, restaurant tenants in particular took advantage of the flexibility offered by the building to create the space they wanted. Perhaps surprisingly, although the floor-to-soffit height at Level –1 is 4.6m, many of the restaurant tenants opted to create a mezzanine within their demise. Although this had not been explicitly catered for in design, the floor loading allowances were sufficient to enable this.

Overall, the building has proved to be versatile and has accommodated a wider range of tenants than originally envisaged. As well as more conventional retail units, the building has accommodated 10 restaurants, a number of double-height units (including one involving an unexpected staircase through the top of the box down to Level –1), a three-screen cinema, and even a medical centre.

Roof garden

The roof garden provides a new welcoming public space that works to unite the residential neighbourhood of Poplar and the business district of Canary Wharf.

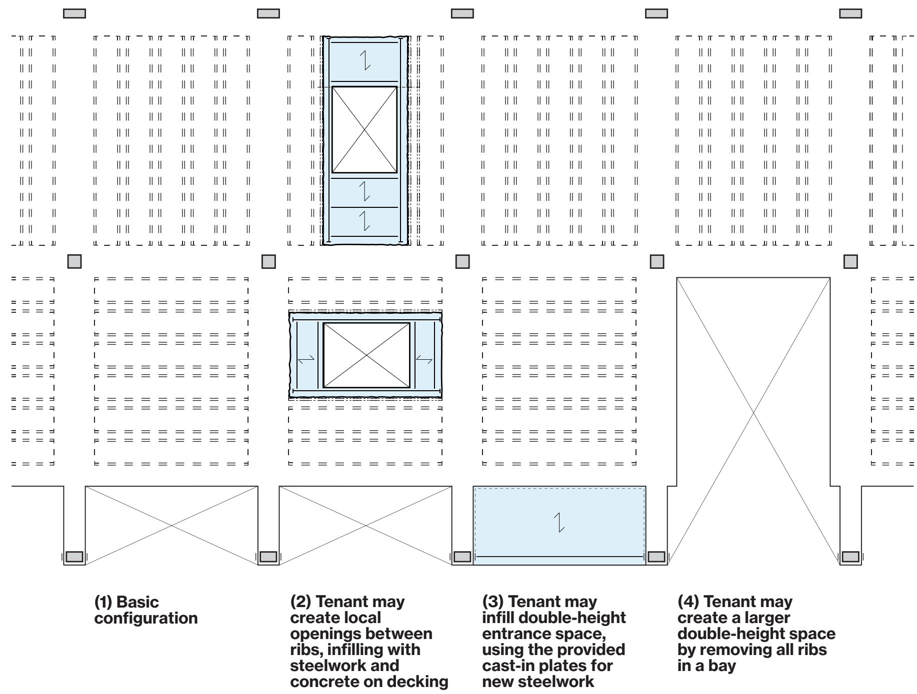
The landscape architect envisaged this space to be completely different from the rest of Canary Wharf, and to be a glowing beacon among the high-rise buildings at night[2]. The building's ship analogy is extended, with the garden being a reminder of the North Dock's maritime past. Hundreds of plants collectively represent and showcase the many countries visited by ships of the West India Dock Company, which unloaded their wares where the station now sits.

The positioning of the garden above a station provided a number of physical constraints and challenges: shallow substrate depth, weight of planting and an overhead roof structure. These challenges were embraced by constructing the garden over a slab that acted as a wide tray containing enough soil to support mature trees and plants.

To allow for root growth and drainage of the trees and plants, footpaths had to be elevated on lightweight supporting structures. The structural loading constrains the soil depth to 1.2m generally: deeper zones were created by placing polystyrene void formers beneath the soil. Where the garden spans over Bank Street, a 27m span steel I-beam structure was adopted, with composite floor on metal decking. In this area, soil depth is restricted to 0.6m.

All structures in the roof garden, including the retaining walls, amphitheatre structure, roof access plinths, etc., are designed to be 'free standing' and do not penetrate the waterproof membrane or insulation on top of the Level +1 slab. This enables the garden to be reconfigured (if needed) without disturbing retail tenants below.

Figure 9
Summary showing flexibility built into retail design for tenant modifications



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Gridshell roof

The park and the rest of the building are enclosed by a distinctive roof, which wraps around the building like a protective shell (Figures 10 and 11). This 310m long timber lattice roof is open in the centre to draw in light and rain for natural irrigation. Timber was an appropriate material to enclose the park – it is organic in nature and appearance, strong, adaptable and is sustainably sourced.

During design development, many studies were undertaken to rationalise the roof geometry and produce a form that maintained an aesthetically satisfying curved shape while minimising the number of elements (reducing crane time) and the number of steel nodes that connect each element (reducing cost). Eventually, a form was derived that is effectively a barrel arch structure, tied across the Level +1 slab via the steel embedment plates connecting the roof to the floor.

The roof consists of 1418 beams and 564 nodes. The beam lengths are typically 6m, although this varies around the roof, with the geometry subtly adjusted in places to accommodate constraints such as the connecting bridges between Crossrail Place and Canada Square. Over mechanical plant areas, steel beams are used in order to avoid continuously exposing timber elements to damp air.

Structurally the roof is continuous, with moment-resisting connections at each node. This makes the structure highly indeterminate, and considerable effort was paid to understanding the sensitivity of the roof to varying stiffness assumptions given the relatively low ductility of the high-strength screws used in the connections. Load tests of connections were undertaken to assess connection strength and ductility.

A second consideration was ensuring that the roof would be sufficiently durable to give maximum lifespan for reasonable cost. The roof is made of European softwood (spruce) and is protected from rain by anodised aluminium flashings. Additional screws have been placed in various connections with the intention of extracting them at regular intervals in the building's life to assess corrosion, given the building's location in a brackish dock.

The roof shape naturally requires elements to twist around the roof, and in the final design the twist was taken out at each node. While this made the nodes relatively complicated, it enabled each timber piece to be entirely straight and square-ended, minimising timber wastage – only four timber pieces had to be

Figure 10
Timber roof under construction. Galvanised tubes that will supply air to pillows are visible from this angle, but were carefully positioned so they cannot be seen from eye level. Concrete floor forms roof to retail units below, prior to receiving waterproof membrane and soil for garden

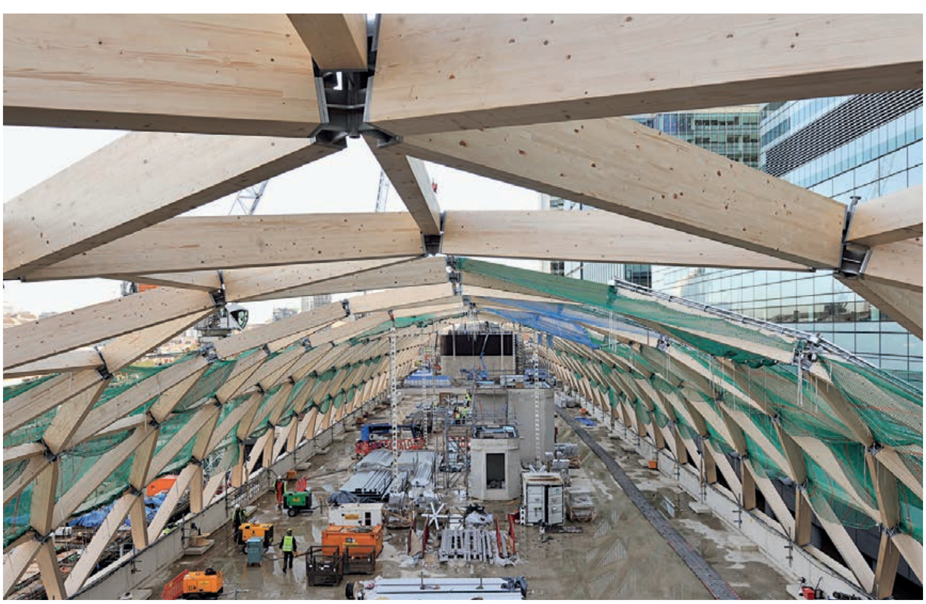
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Figure 11
View from Adams Place showing completed roof and Adams Place footbridge

curved. Three visual grades of timber were used, with the lower visual grades used at greater distances from eye level. The nodes consist of welded steel plates, 364 of which are of unique geometry. All nodes were hot-dip galvanised to maximise their durability.

In contrast to some other large lattice roofs, the roof at Crossrail Place was designed to minimise the amount of propping required during construction. This was achieved by arranging the structure into a series of arches that cross the building diagonally on plan and, when linked together, form a relatively short length of roof that is stable in itself. In addition, the timber beams' moment connections were sized to enable each beam to cantilever off one node in the temporary case, thus avoiding further propping. Once the central section of the roof had been erected, further beams were added progressively towards each end.

The gridshell roof is supported by the Level +1 structure at 6m centres along the building. Typically, Level +1 consists of relatively stiff 1m deep reinforced concrete beams; however, where Level +1 spans over Bank Street, the roof connects to 27m span steel beams which are much less stiff. Analysis showed that the steelwork and roof diagrid combine structurally to form, in effect, a three-dimensional truss. This was a potential problem because it meant that when the roof garden soil load was added afterwards, a significant proportion of the load would end up in the timber diagrid rather than being confined to the steelwork. This would have resulted in substantially larger timber member sizes in this region.

To avoid this, 300t of kentledge was placed on the steelwork before the roof was erected, this weight being similar to that of the future soil. The roof was then erected and the kentledge removed, with the structure deflecting upwards as a result. When the roof garden was built, the added soil returned the roof to the originally erected level, with very little net load in the timber members.

The construction programme for the timber gridshell was approx. six months. Following erection of the main gridshell, the flashings were added along with the ETFE pillows and air supply systems (Fig. 10). The pillows require a continuous supply of air, although the system is designed to generally operate at a lower pressure when possible, with higher pressure only being required for high loading (such as snow load).

Access to the roof is via mobile elevating work platform (MEWP) from the underside, with the MEWP connection pads discretely hidden in the garden planting. At the 30m cantilever ends, access is via cleaning cradles suspended from rails. Access above the roof is via abseiling points which are connected to the nodes.

Sections of the roof can be removed for plant replacement below – an important design consideration was the effects of load redistribution in the continuous structure once a section is removed. The roof has been designed for robustness beyond that required by the Building Regulations, which is relatively easy to achieve given the load redistribution potential of a diagrid structure.

Summary

Crossrail Place opened on 1 May 2015, nearly four years ahead of the Elizabeth line station beneath it. It adds 100 000sq.ft of retail and leisure facilities to Canary Wharf Group's estate, which now incorporates over 300 shops and restaurants (Figure 12). Crossrail Place is a unique addition to Canary Wharf's social and business community.

Project team

Client: Canary Wharf Contractors Ltd
Architect: Foster + Partners; Adamson Associates International; Tony Meadows Associates
Landscape architect: Gillespies
Civil and structural engineer: Arup
MEP engineer: Arup
Concrete contractor: Expanded
Timber roof contractor: Seele-Wiehag jv

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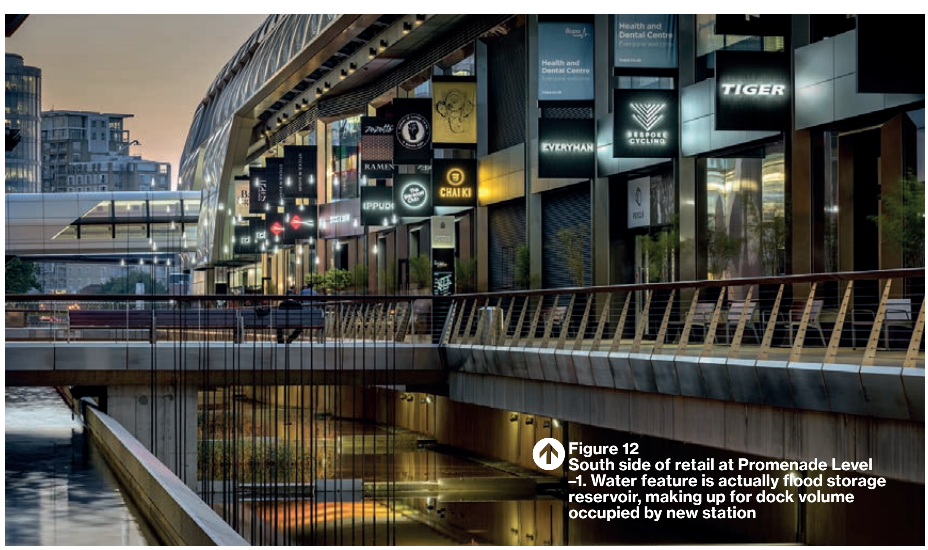

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Figure 12
South side of retail at Promenade Level –1. Water feature is actually flood storage reservoir, making up for dock volume occupied by new station

Engineering design of the platform edge screens for the Elizabeth line's tunnel stations

Julian Birbeck
CEng, MIStructE
Associate, Atkins, UK

Rosemary Smiley
MEng
Structural Engineer, Atkins, UK

George Stowell
RIBA, MRIAI
C100 Framework Contract Manager, Atkins, UK

Ed Newman-Sanders
CEng, MICE
Technical Director, Atkins, UK

Synopsis

This paper covers the engineering design of the platform edge screens for five Elizabeth line tunnel stations in central London. Full-height platform edge screens are a signature feature of the Elizabeth line's station platforms, and their design presented many challenges. To gain maximum uniformity, the edge screens were developed as a common reference design, which was then issued to each of the station contractors.

The paper describes the technical challenges from the point of view of a structural engineer, but in doing so, it draws in interfaces with disciplines as diverse as tunnel ventilation, electrical engineering, and rolling stock procurement. The reference design approach allowed unique features of the platform edge screens to be prototyped and tested before construction.

Figure 1
New landmark – illustration of length of single PES set against London skyline

ISTOCK

Introduction

The term platform edge screen (PES), as used in this paper, describes a complete assembly, comprising screen doors, an upper 'service wall' supporting lighting, communications and cabling, plus a smoke-extraction duct positioned over the track. The term PES-frame is used to describe the structural frame that supports all these elements. This PES-frame design applies to five stations: Bond Street, Tottenham Court Road, Farringdon, Liverpool Street and Whitechapel. These are the Elizabeth line stations with platforms in tunnel bores, as opposed to stations with platforms in basement boxes, such as Paddington and Canary Wharf. The PES-frame design for the latter presented a different set of constraints and is not covered in this paper.

From street level, the scale of Elizabeth line platforms and the associated screens may not be apparent. The length of a single Elizabeth line PES is up to twice that of an existing London tube platform. Underground platforms on this scale will provide a dramatic new experience for London commuters and, as such, the screens can claim to have as significant an impact on London's cityscape as a new skyscraper (Figure 1).

Nine-car Elizabeth line trains are over 200m long, and each platform has some additional publicly accessible length to allow for longer trains in future. Consequently, there is over 0.5km of screen required at each station and the PES-frames described in this paper extend for over 2.5km in total. The PES-frame is designed around a 3m module, so there are over 830 of these modules across the network.

Contractual set-up

The PES was developed as a cross-station design package (see McClements, 2015 and Moxon and Atherton, 2015 for a more detailed discussion of the benefits of this approach)[1,2]. The design was undertaken as part of the Crossrail Architectural Component Design contract (designated 'C100'), comprising Atkins, Grimshaw, GIA Equation and Maynard, who designed and prototyped the line-wide sub-surface station fit-out, and surface station visual identity.

Components were drawn, performance-specified, mocked up, prototyped and tested as generic solutions. Once approved, these common solutions were passed on for station-specific design, manufacture and installation.

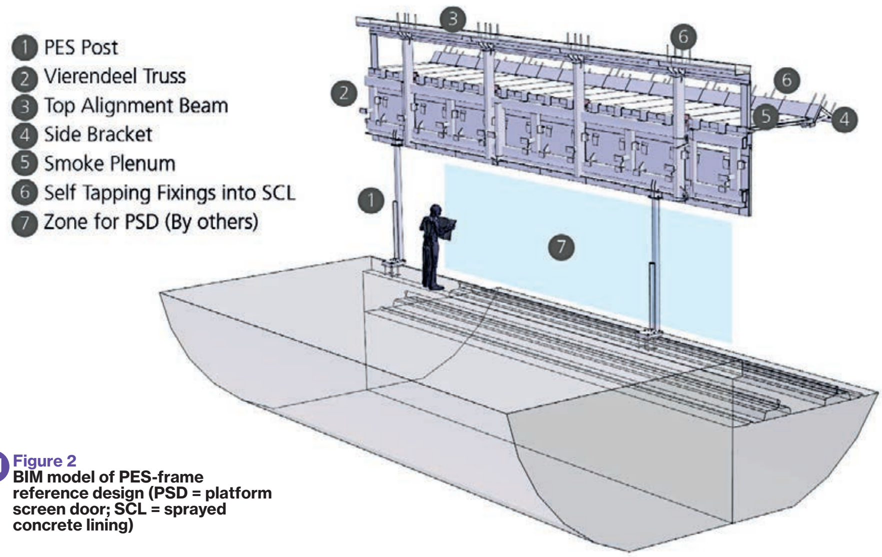

Figure 2
BIM model of PES-frame reference design (PSD = platform screen door; SCL = sprayed concrete lining)

"THE PLATFORM EDGE SCREEN WAS DEVELOPED AS A CROSS-STATION DESIGN PACKAGE"

The platform screen doors, including the glazed infill panels, are specialised mechanical components and were delivered by a separate contract (Figure 2).

A part of the overall strategy was to undertake stakeholder engagement with the organisations operating the stations and railway. This involved reviewing design drawings, three-dimensional (3D) building information models (BIM), and physical prototypes. Thereafter, a coordinated access and maintenance strategy was issued to station contractors alongside the RIBA Stage F1 design (according to the RIBA Plan of Work 2007): the objective being to create a harmonised maintenance strategy.

Purpose of platform edge screens

Platform edge structures are uncommon in the UK, being first used on the Jubilee line extension in 1999. Nonetheless, such structures can be found on many metro systems and fall into three categories (Figure 3):

- **Platform edge doors** are balustrade-height edge structures with automatic doors aligned to the train doors. Their sole function is to prevent passengers falling onto the track.
- **Platform screen doors** are doorway-height structures with automatic doors aligned to the train. They have the same safety function as platform edge doors, also providing a degree of screening to passengers from air movement.
- **Platform edge screens** are platform-to-ceiling structures providing more extensive screening than platform screen doors. They also separate the platform and track environments.

Figure 3
Classification of platform edge structures

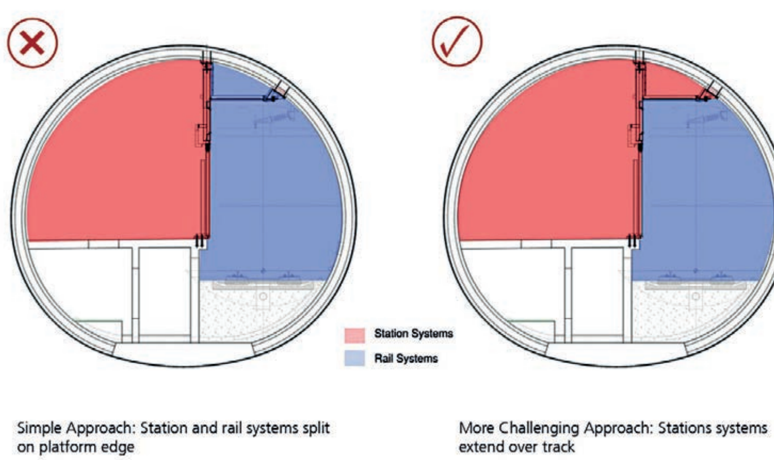

Figure 4
Platform tunnel space planning: station and rail zones

For the Elizabeth line, the early decision to use a PES transformed the tunnel ventilation strategy. Since air leakage through stations was effectively eliminated, the need for six additional ventilation shafts and head-houses in central London was removed.

The simplest approach to design the PES would be to collect all station systems on the platform side and all rail systems on the trackside. Stations require extraction ducting for the full length of the platform – both for the day-to-day managing of platform ventilation, but critically also to provide smoke extraction in the event of a fire.

The decision was taken to place the extraction duct over the track (Figure 4), allowing the platform space to take on a unique character and resulting in a very different passenger experience. The tunnel cladding curves over the passengers' heads to the tunnel apex, with all lighting, signage, public address systems and associated cabling then located on the vertical face of the PES above the screen doors. Light from the lightboxes reflects off the tunnel cladding to create a soft, diffuse ambience (Figure 5).

Consequently, the structural engineer is in a pivotal position: designing a structure which is not only critical to the overall master-planning and land-take of the railway, but which also underpins the platform architectural concept and the passengers' experience.

Development of structural diagram

There are two alternative approaches to designing a PES-frame: it can either be suspended from the tunnel crown above; or propped from the platform edge below. The choice between these two approaches is not straightforward, due to conflicting design constraints. Firstly, during fit-out, the system-wide contract passes through each platform with track-laying plant requiring an unobstructed zone along the platform edge. Secondly, Elizabeth line stations differ from existing underground stations in their use of sprayed concrete lining reinforced with fibre[3] for profile stabilisation. The sprayed lining has limited capacity for point loads.

Sprayed linings are not suited to the accurate placing of reinforcement. Hence, rebar is used sparingly, only at critical junctions. Furthermore, multistage lining build-up raises a risk of delamination between layers under radial point loads. Overall, it was judged that the concrete lining could not be designed with sufficient long-term capacity to support a suspended PES-frame in its entirety. The alternative was to support the PES from below with posts placed onto the platform edge; however, posts passing down to platform level would need to coordinate with the door positions on the screen doors. At the time of design and initial tunnel construction, the rolling stock had not been ordered; moreover, for competitive tender, train bids were placed with multiple suppliers, each with different door configurations. Even when this procurement sequencing issue was resolved, there remained a need to provide future flexibility in the PES, including the option to extend trains in the future with different door configurations.

The chosen structural diagram was therefore an adaptable hybrid, which could function in a suspended or propped configuration in the temporary and permanent conditions, respectively. A sequence of fixings is placed into the tunnel crown at 3m centres; these fixings provide vertical and horizontal restraint temporarily, but revert to horizontal restraint only in the permanent condition, in which the PES-frame is supported from the platform. The propped configuration needs to include sufficient articulation to allow for deformations in the tunnel cross-section, known as tunnel 'squat' (Figure 6).

The crown fixings support a continuous Vierendeel truss with a 1.5m module. The truss is designed so that it may be supported at any point with pin-ended posts onto the platform edge below (subject to some basic setting-out constraints). Horizontal props on a 3m module provide lateral restraint. To complete the system, the smoke duct soffit is formed in precast planks, spanning from the top of the truss onto a continuous side-bracket, fixed to the sprayed concrete lining on the track side.

This structural arrangement provides alternative load paths, giving the PES-frame an inherent robustness. Should a PES-post be accidentally removed, the slotted-hole connections at the crown would reach their limit of travel and the frame would revert (short term) to a hanging structure.

Given the length of the PES, longitudinal movement was an additional consideration. Expansion joints are provided at 15m centres along the platform. Between these movement joints, it was necessary to provide moment-splices in the continuous truss, allowing it to be installed in 3m, 6m or 9m lengths (Figure 7). Setting-out rules were devised

Figure 5 Elizabeth line platform environment

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"THE CHOSEN STRUCTURAL DIAGRAM WAS THEREFORE AN ADAPTABLE HYBRID"

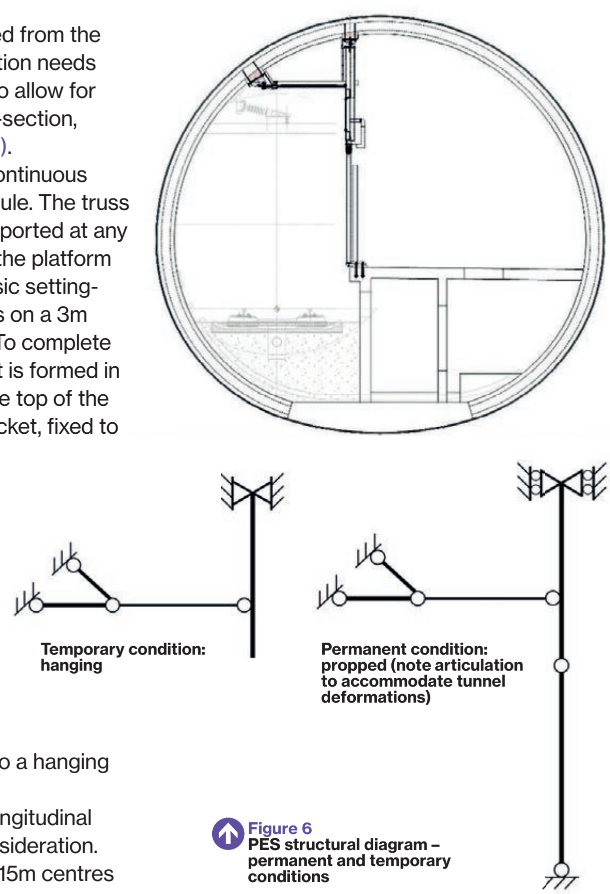


Figure 6 PES structural diagram – permanent and temporary conditions

Figure 7
6m length of PES Vierendeel truss being lifted into place at Farringdon Station (note top and side alignment beams already in position)

PATRYK TOKAREK/YOUTUBE

allowing the reference design to be adapted to any permutation of rolling stock and train stopping position.

The PES is not fire-rated to withstand a full train fire, but it does need to function in the event of a small baggage fire. To this end, computational fluid dynamic analysis was undertaken, confirming a need for the PES to resist smoke temperatures of 200°C for up to one hour. This temperature does not critically affect steel strength, but does create a significant degree of thermal expansion, which needs to be accommodated at the movement joints.

Tolerances

The sprayed concrete lining has a large construction tolerance envelope of 100mm, whereas the screen doors and the cladding fixtures have an installation tolerance of ±5mm. It was thus clear that due to multiple interfaces between the structural frame and the supported elements, this too would need to be erected to cladding tolerances. Consequently, the connection into the sprayed concrete lining was required to take up the major portion of the tolerance.

The solution was to use grout infills running longitudinally along the tunnel at the two upper connections into the sprayed concrete lining: the top connection at the tunnel crown, and the side connection at the trackside edge of the smoke plenum. A folded-plate alignment beam was placed at the apex of the tunnel, and a similar folded-plate detail was used for the side bracket (Figure 8). In this way, the erector was required to line and level these elements, which were delivered to site in 6m or 3m lengths. Once positioned within tolerance, grout infills were poured and the required tolerances were locked in for the subsequent frame erection.

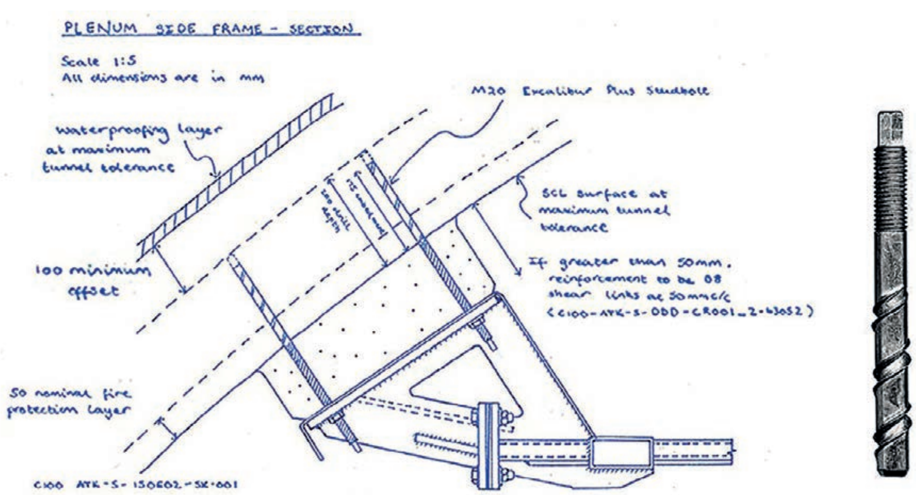


Figure 8
Design development sketch showing grout infill, stud anchors and self-tapping threaded M20 stud-anchor

As there are large tolerances involved at the sprayed concrete lining interface, the anchors needed to be through-fixings, allowing the PES brackets to be offered up, lined and levelled, with holes drilled using the brackets as templates. The anchors were threaded studs, with two nuts clamping to allow the PES bracket to be held firmly in position while grout was poured.

Anchor choice was also influenced by the Boston Interstate 90 tunnel ceiling collapse of 2006 (consequent on creep in chemical fixings). Crossrail's technical standards prohibit such anchors working in direct tension in overhead fixings. The adopted anchors were therefore self-tapping anchors, used extensively for secondary fixings on the Channel Tunnel Rail Link. These offered the advantage of achieving full shear and tension capacity immediately when screwed into

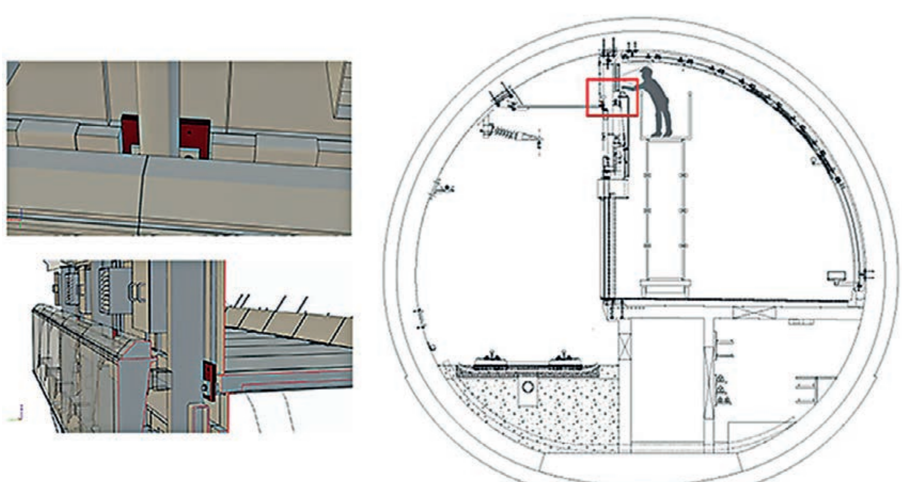


Figure 9
BIM model used to develop access and maintenance strategy

position, without the need for any subsequent operation or curing time. To ensure that delamination in the tunnel lining would not occur, and that the threaded anchors would not loosen or fatigue, Crossrail commissioned testing at Imperial College London. A testing rig was designed to replicate the connection between the PES-frame and sprayed concrete lining and the anchors were subjected to a cyclic load, representative of 10 years in service.

Durability, design life, and maintenance

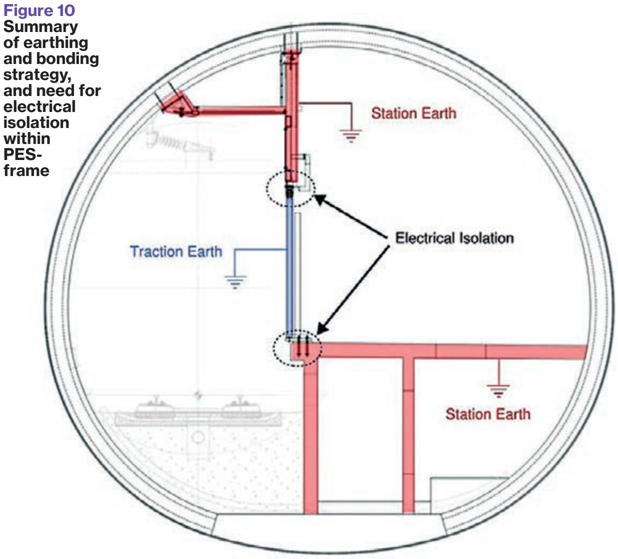
The permanent and variable loads applied to the PES are relatively small, compared to loads on other elements of station structure. On the platform elevation, the PES is required to support lighting, signage and other items of equipment, totalling 1kN/m^2 (i.e. a typical cladding load). The smoke plenum soffit, formed in precast concrete, weighs 2.2kN/m^2 on plan.

As far as variable actions are concerned, crowd load is represented by a 3.0kN/m line load. In addition, piston loads from train movement create pressure changes, with a typical value of 0.8kN/m^2, and an extreme case of 1.2kN/m^2. As such, the static load capacity of the PES-frame does not present an engineering challenge. The key factor, however, is fatigue: each train movement creates a complete reversal of piston pressure. Given the operational timetable of the network, with trains every two minutes at peak times, there are over 24M fatigue cycles during the 120-year design life. Consequently, fatigue governed the design and detailing of welded connections within the PES-frame.

Another design challenge created by the Elizabeth line's running schedule is the demand for ongoing inspection and maintenance. This task is to check the steel and corrosion protection condition and assure nuts remain tight – albeit all nuts are secured with locking washers. To aid inspection, all connections were designed with the bolts visible, a condition verified by using a 3D BIM model (Figure 9).

The corrosion protection system needed to be a minimum-maintenance solution. Stainless steel structure was considered, but adequate life was achieved from a cheaper galvanised finish (typically 140μm).

Figure 10 Summary of earthing and bonding strategy, and need for electrical isolation within PES-frame



> "EARTH VOLTAGE ON A TRAIN IS 'FLOATING' RELATIVE TO ITS SURROUNDINGS"

Electrical isolation

Electric train traction relies on the return current from the overhead lines passing through the rails. Over the distances involved, rails have significant electrical resistance, with the net result that earth voltage on a train is 'floating' relative to its surroundings and can be in the order of 50V.

This is of no concern when the train is moving and the passengers are separated from the surroundings. However, when the train stops, it is imperative that passengers cannot touch the train or any surrounding metallic infrastructure located on a separate electrical earth. The platform edge structure therefore must be electrically bonded to the adjacent rails and isolated from the surrounding station earthing system.

This is achieved by electrically isolating the screen doors and the adjacent PES-posts from the remainder of the PES-frame above and the platform edge rebar below (Figure 10). The latter isolation was achieved by locally reinforcing the platform edge with non-conducting glass fibre-reinforced plastic rebar.

The former isolation, however, presented a challenge. The screen doors and the PES-frame form a wall bounding the platform space. As such, they are subject to the Sub-Surface Railway Stations Regulations[4], which originated in response to the Kings Cross underground fire of 1987. The regulations stipulate that any material used in the construction of a wall in a public place must be of limited combustibility – and this cannot be achieved with a polymer. Consequently,

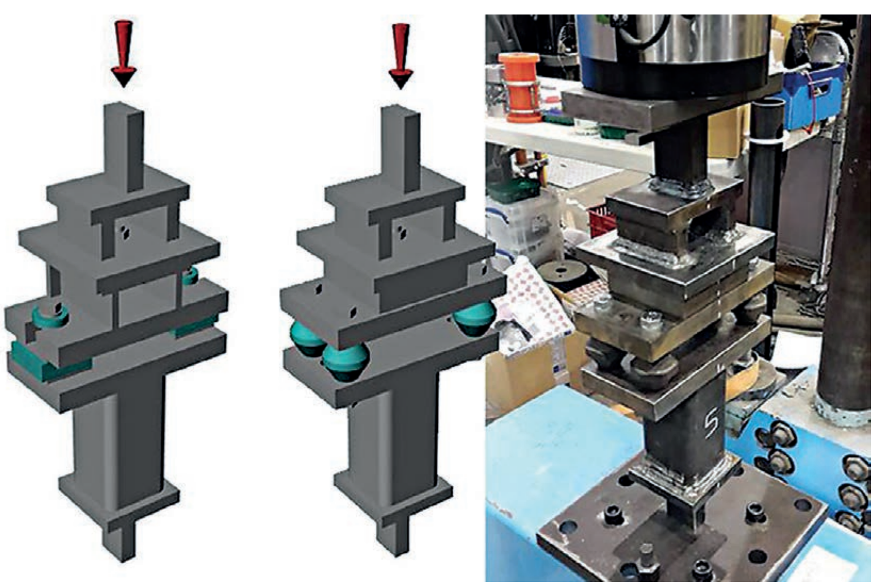


Figure 11 Isolator testing rigs

Figure 12
Installation of PES

three different isolator materials were shortlisted and their performance under cyclic load tested at Imperial College London. Mica-glass and Macor® – both machined glass products – and ceramit-14 – a ceramic – underwent cyclic load testing in rigs representative of their positioning in the PES-frame (Figure 11). The mica-glass performed best and was the material selected for the isolating components. It is anticipated these will require periodic replacement and frame detailing was developed with this in mind.

Construction and operations

The C100 design team prepared the PES-frame reference design for a generic, straight run of tunnel platform to RIBA Stage F1. As such, the station contractors (Figure 12) were presented with an assured steelwork design, including full connection design, as well as precast planks for the plenum soffit with full reinforcement design. In this way, C100 created a major saving in time and cost through common design and coordination effort, compared to each contractor working up a PES themselves, and even greater value in a common safety regime and maintenance processes.

Conclusions and lessons learned

When the Elizabeth line opens to the public in December 2018, there will be a completely new subsurface environment on the London transport network. The 250m long platforms with 5m unobstructed headroom will change passengers' expectations of subsurface rail, made possible by the gathering of lighting, signage, communications and services distribution onto the vertical plane of the PES, with the smoke-extraction plenum concealed behind.

"THE STATION CONTRACTORS WERE PRESENTED WITH AN ASSURED STEELWORK DESIGN"

The delivery of the PES design by the C100 Architectural Component Design package brought undoubted design and maintenance efficiencies. Given the complexities of the interfaces with the tunnel lining, electrical isolation, coordination with the door locations, and offsite testing, the level of design supervision required would have been significantly greater had these issues been tackled independently by the station teams.

From the structural engineer's perspective, the PES design is intriguing. A cursory glance at the structural spans and the applied loads suggests that the PES is a simple element of secondary steel. Challenges have arisen, however, from the interfaces with other systems and are inherent in a heavily serviced, spatially constrained railway. The key to unlocking these challenges has been a structure with built-in flexibility. Adaptable geometry allows the location of support posts to be varied, and adaptable load paths allow the structure to be hung as well as propped. In this respect, the development of such 'smart' structural components, with parameters that can be 'flexed' to suit local, temporary or future conditions, may become increasingly common for large infrastructure projects.

Acknowledgements

The authors are grateful to Dr Sunday Popo-Ola, who oversaw the material testing of the isolator components and threaded anchors at Imperial College London, and led the subsequent analysis of the results.

Atkins is a part of SNC-Lavalin's Engineering, Design and Project Management business.

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Rigorous assessment of existing overhead line gantries for the Elizabeth line

Simon Fryer
MEng, CEng, MIStructE, MICE
Technical Director, BuroHappold Ltd, UK

Matthew Crouchman
BEng (Hons), CEng, MIStructE, MICE
Associate, BuroHappold Ltd, UK

Thomas Eckhart
MEng
Engineer, BuroHappold Ltd, UK

Synopsis
The Elizabeth line will use above-ground sections of existing Great Eastern and Great Western tracks between Stratford and Maidenhead where new overhead line equipment (OLE) and traction power supply will be installed.

The OLE is supported by gantries of various types and configurations. In the case of the Great Eastern, the gantries date from the electrification of UK railways in the 1940s.

Initial structural assessment carried out had shown that existing gantries on the route were inadequate to carry increased loading from the upgraded OLE. However, a rigorous procedure incorporating detailed non-linear structural analysis was developed that eliminated some of the inherent conservatism in traditional codified approaches. Particular benefit was found in the case of the many types of slender structure where buckling was a governing factor. Using non-linear techniques, it was possible to demonstrate that families of structures were suitable for incorporation in the Crossrail (Elizabeth line) scheme.

This paper describes the approach that was used. The project is remarkable for significant programme and cost savings that were accomplished using sophisticated engineering analysis. It is also noteworthy from a sustainability point of view, as it allowed the existing infrastructure to be reused.

NOTATION

ATF	autotransformer feeder wire
IWC	idealised worst case
OLE	overhead line equipment
SWC	specific worst case
TPS	traction power supply

Introduction
The above-ground sections of the Elizabeth line route that extend from Maidenhead to Shenfield incorporate upgraded overhead line equipment (OLE) and traction power supply (TPS) cables that will be supported on existing OLE gantries as far as possible. Most of the gantries on the Great Eastern railway date from the electrification programme of the 1940s and may therefore be among the oldest surviving overhead electrification gantries in the country. Those on the Great Western railway are more recent, dating from the Heathrow Express electrification scheme of the 1990s.

The TPS project will install two or four new autotransformer feeder wires (ATFs) along with associated earth wires to the route. In parallel with this, the OLE on the Great Eastern is being renewed with a more modern system.

The gantries may be categorised into families of similar structural types. Common families are single masts, cantilevers, head span and portal structures, some varieties of which are illustrated in Figure 1. Most of the members making up the gantries are relatively slender rolled steel sections, either single or compound. The masts of the gantries are generally either embedded in, or bolted to, mass concrete foundations.

Loading on these gantries includes contributions from:

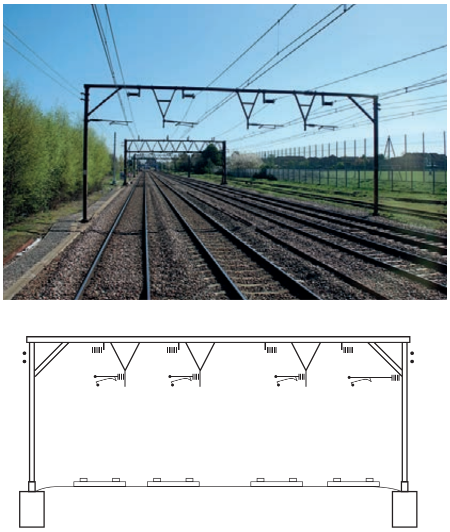
- the self-weight of the structure and wires
- wire tension, taking account of temperature, deviation angles and eccentricities
- wind on the structure and wires
- ice on the structures and wires.

Since the loading on the existing gantries would change as a result of the proposed works, a structural assessment was required. Initially, assessments carried out by others in accordance with BS 5950-1[1] indicated that the structures would not be suitable for re-use due to high calculated utilisations. However, it was considered by Network Rail that these initial assessments were in some cases unduly conservative. Therefore, a review based on rigorous assessment was commissioned. The aim was to ascertain, as realistically as possible, the structural utilisations so that the number of structures requiring replacement could be accurately determined.

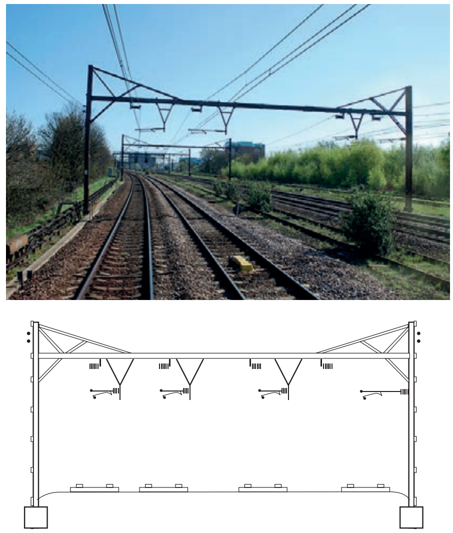
Methodology
Strategy
The project required over 700 gantries to be assessed to a demanding timescale. Therefore, it was important to develop an efficient strategy for the work. It was considered important to eliminate unknowns and therefore remove conservatism as far as possible. In particular, the following aspects were considered:

Wind loading
Wind actions on structures located in cuttings can be significantly less than those in open country on embankments; therefore, site-specific wind loading was considered. Both along-track and across-track wind loading needed to be considered. Wind loads are applied to the gantry structure and the wires.

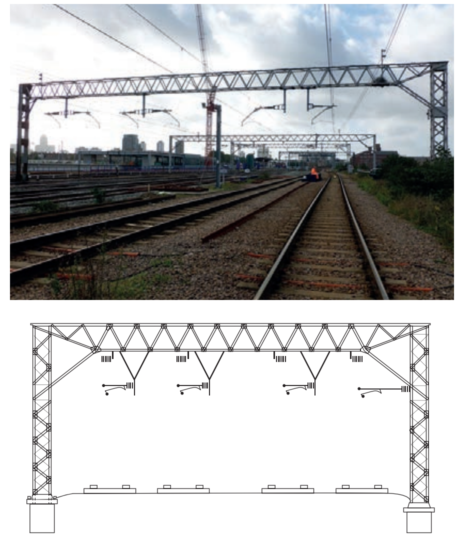
Figure 1
Types of OLE gantry



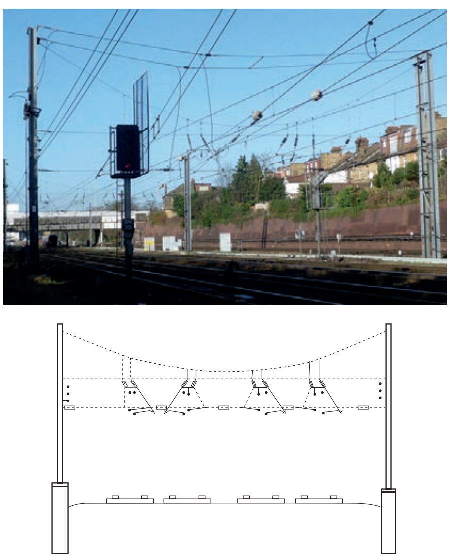
a) Single-span knee-braced portal (Great Eastern)



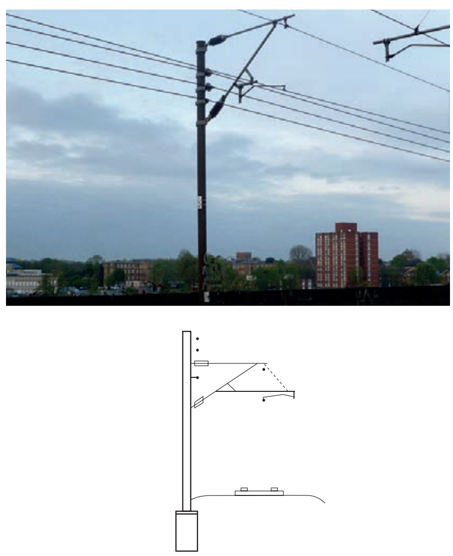
b) Single-span top-tie portal (Great Eastern)



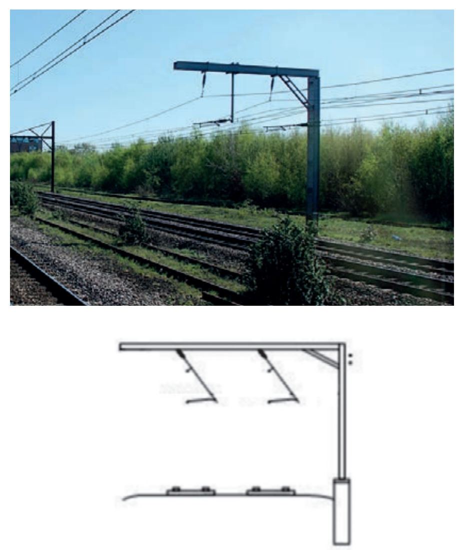
c) Single-span lattice portal (Great Eastern)



d) Head span (Great Western)



e) Single-track cantilever (Great Western)



f) Two-track cantilever (Great Western)

Wire loading
As well as the new ATFs and earth wires, loads are applied by the contact and catenary wires. These wires can either be auto-tensioned or fixed termination. Auto-tensioned wires maintain the same tension regardless of temperature and this is therefore relatively well defined. Fixed-termination wires have tension that varies with temperature, typically increasing significantly at low temperature. (The assessment considered temperatures down to –18°C, which is considered conservative for the region under consideration).
In a number of cases, gantries were assessed with both fixed-termination and auto-tensioned wires due to the phased replacement of older equipment.

Wire tensions are considered as external loads in the structural analysis, with any 'guying effect' conservatively ignored. Deviation of wires occurs at gantries due to track curvature or wire stagger on a straight track, and this is significant as it results in lateral loads. The registration arm of auto-tension equipment will move with temperature – the resulting position of wire loads and any associated eccentricity must be carefully modelled. Dynamic loads resulting from cable breakages were not considered.

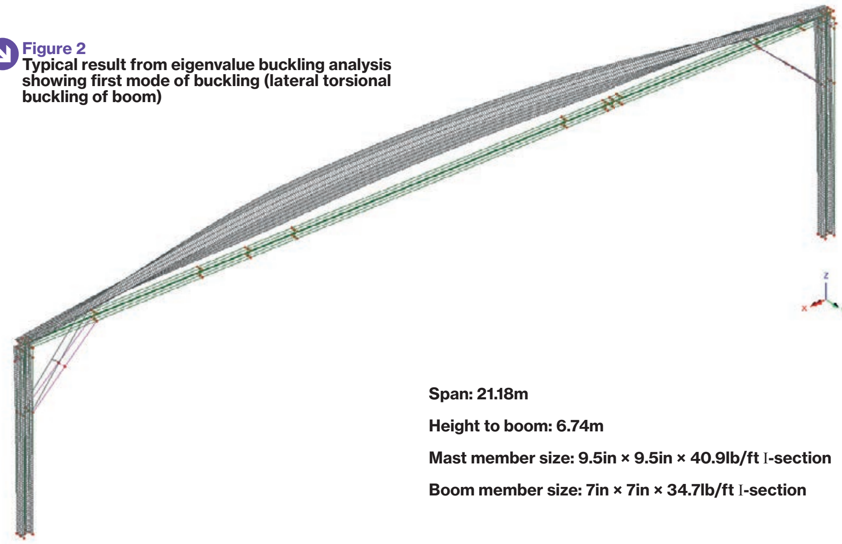
"THE PROJECT REQUIRED OVER 700 GANTRIES TO BE ASSESSED TO A DEMANDING TIMESCALE"

Ice loading
Ice loads are applied to both the structure itself and the wires and are assessed based on a 9.5mm radial thickness. The contact wire is assumed to be kept clear of ice by the passage of pantographs.

Condition
All of the structures were inspected by means of walk-through and high-level surveys to check their condition. Bearing in mind the age of the structures, the assessment assumed a default condition factor of 0.95 to cater for a moderate level of corrosion, etc. (The original treatment of the gantries was paint on the Great Eastern railway and galvanising on the Great Western.) Any observed defects adjudged to be more severe than this were explicitly considered in the assessment. It was assumed that adequate future maintenance would be undertaken to prevent any further deterioration.

Geometry
Record drawings existed for the majority of the gantry structures. Several structures have undergone modifications during their lifetime and this was checked and recorded as part of the site inspection. The number of items of OLE registered at the structures was also recorded and key dimensions and section sizes were checked.

Figure 2
Typical result from eigenvalue buckling analysis showing first mode of buckling (lateral torsional buckling of boom)



Span: 21.18m
Height to boom: 6.74m
Mast member size: 9.5in × 9.5in × 40.9lb/ft I-section
Boom member size: 7in × 7in × 34.7lb/ft I-section

Material testing
A limited number of material samples were taken on site from non-critical parts of the older Great Eastern gantries so that laboratory tests could be performed to verify the assumed steel strengths of the structures. For historic steelwork on the Great Eastern route, a yield strength of 230N/mm^2 was considered. For more recent steelwork on the Great Western route, yield strengths were based on the use of grade 50B (345N/mm^2) or grade 43A (275N/mm^2) steel, as noted on record drawings.

Datasheets and categorisation
Structure datasheets were prepared for each gantry with key dimensions (member sizes, boom height, across-track span, along-track span), wire heights and track alignment information, as well as inspection remarks and site photographs.

Each family of gantries was categorised into sub-families and the key data were

Figure 3
Typical result from non-linear analysis indicating onset of non-linearity at load factor of approx. 2.4, based on displacement (m)

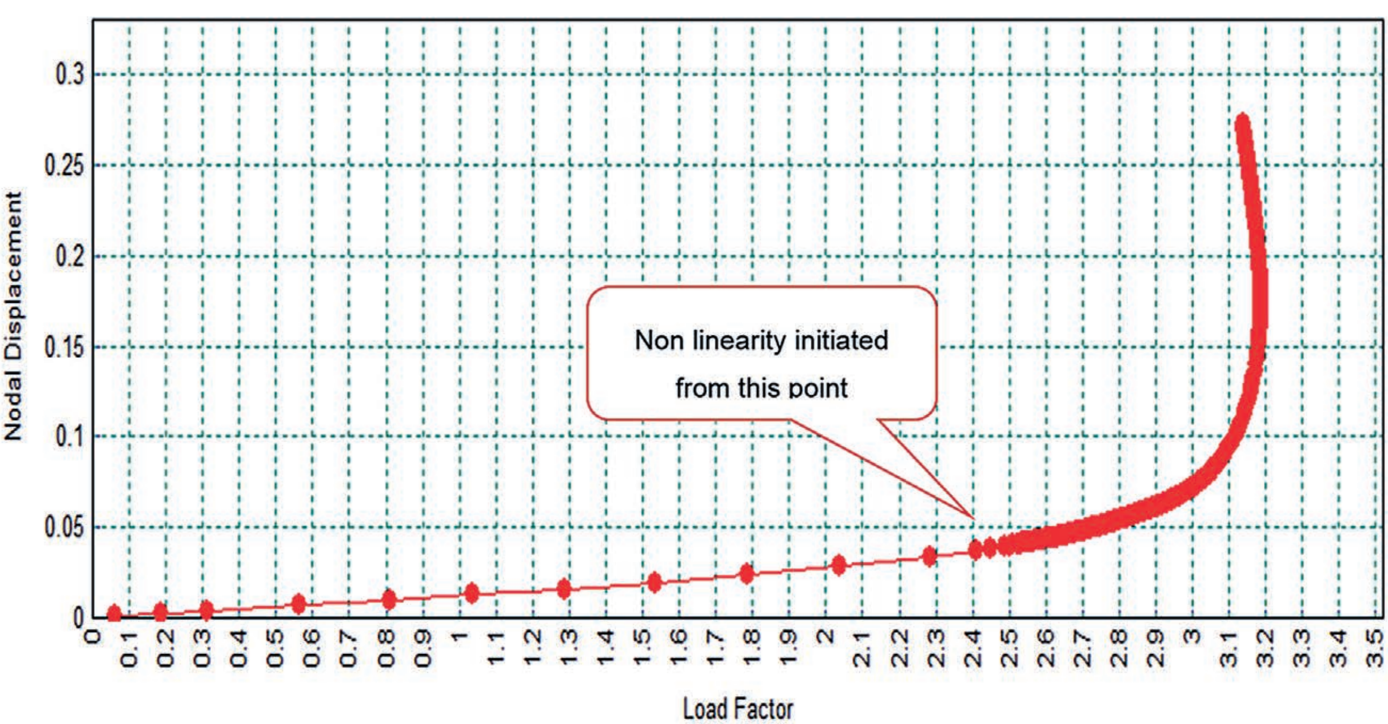
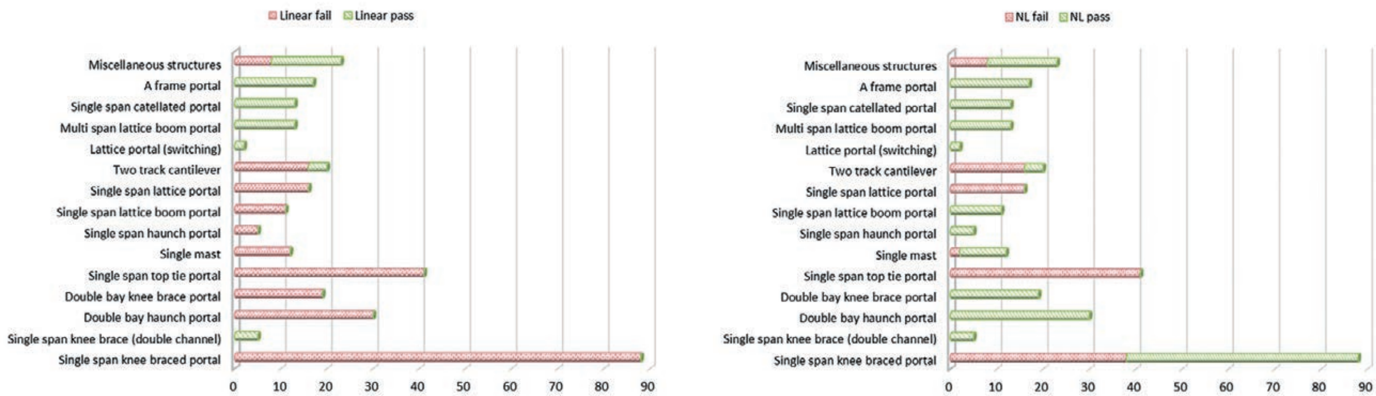


Figure 4
Summary of assessment results for main types of structure (Great Eastern)



a) Linear assessment results

b) Non-linear assessment results

tabulated so that the worst-case structure could be identified ('specific worst case' or SWC). Where it was unclear which structure would have the highest utilisation, an 'idealised worst case' (IWC) was used, derived using an envelope of variables. The assessment was carried out on the SWC or IWC structure and, if passed, the remainder of the sub-family was deemed also to pass.

When failures were found, the assessment progressively 'drilled down' into individual cases to study them in more detail.

Structural analysis

A staged approach to assessment analysis was taken. In the first instance, static analysis (including a check on connections) was considered, followed by non-linear analysis where this would give a better representation of the structure's behaviour. The conventional static analysis was an important first step in order to identify the potential problem areas within the structure. Serviceability deflections were also checked at this stage, although it was generally the ultimate limit state that was found to be critical.

The overall capacity of the slender gantry structures was in many cases found to be limited by the ability of the mast or boom members to resist lateral torsional buckling. This capacity is influenced by the structure's geometry, section properties, the shape of the bending moment diagram and the position of loads (in particular, their location relative to the shear centre of the member concerned, with destabilising loads having a particularly severe effect). In order to eliminate conservatism, rigorous analytical checks were proposed.

The approach adopted for the rigorous analysis was to model the gantry members

"THE RIGOROUS NON-LINEAR ASSESSMENTS SHOWED A SIGNIFICANT IMPROVEMENT"

with shell elements in the LUSAS finite-element program[2] (see Figure 2 for a typical model plot). A full second-order analysis incorporating material and geometric non-linearity was then performed. In the geometrically non-linear analysis, initial imperfections were introduced into the mesh by carrying out an eigenvalue buckling analysis and scaling the deformed shape for several critical buckling modes. The resulting geometry was then used as the starting point for the non-linear analyses, wherein loading was applied incrementally and, by subsequently plotting deformations against load factor, it was possible to determine the point at which divergent, non-linear structural behaviour occurs (Figure 3). A structure was considered to have adequate resistance to buckling if its behaviour was still within the linear zone when ultimate loading had been applied to the structure. Note that using this technique, all loads are subject to the same factor, which must account for uncertainty in the applied actions, material properties, analysis accuracy and structure condition.

A significant advantage of this approach is that the analysis models give direct results of the load factor, which can be related to the structural utilisation (where structural utilisation = 1 / load factor), without the requirement for post-processing of results and assessment of individual section capacities. This approach is in accordance with cl. 5.2.2(7)a) of EN 1993-1-1[3] which states: *'If second order effects in individual members and relevant member imperfections ... are totally accounted for in the global analysis of the structure, no individual stability check for the members ... is necessary'.*

The freedom from subsequent application of codified section checks removes any undue conservatism from the assessment process. This is a satisfactory approach for assessment, but would be cumbersome for design when member sizes need to be individually optimised. Since the initiation of the rigorous assessment work, a new Network Rail standard has been published with improved guidance on the design of OLE gantries[4].

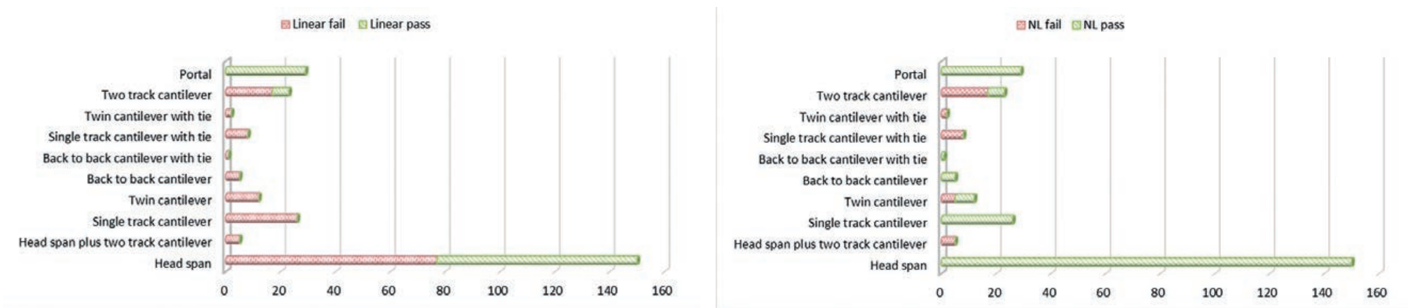
The assessment involved a repetitive process of model creation and analysis. Therefore, a Visual Basic script was developed to speed up the process of generating the analysis models for the gantries by deriving the loading and load combinations to be applied to them.

All assessments involving non-linear analysis were subject to independent checks using a different software package (ROBOT).

Assessment results and strengthening design

The assessment results are summarised graphically for the Great Eastern and Great Western structures in Figures 4 and 5 respectively. It can be seen that the rigorous non-linear assessments showed a significant improvement in terms of the number of gantries shown to be able to sustain the new TPS/OLE loading compared to the linear analysis. Overall, this number increased from 40% of the gantries considered to 75%.

Figure 5
Summary of assessment results for main types of structure (Great Western)



a) Linear assessment results

b) Non-linear assessment results

Of the remaining structures, a number failed assessment due to condition. A common defect was corrosion where water tends to collect at the base of the mast. Single-span top-tie portals on the Great Eastern and two-track cantilevers on the Great Western were found to have a problem with connection capacity, which is why they failed even when assessed using non-linear analysis.

Where this had occurred, or mast base capacity was found to be inadequate, remedial measures were investigated. The solution proposed was a steel strengthening collar to be fixed around the base of the mast and bolted into the foundation. The space between the collar and the mast member was then infilled with concrete (Figure 6).

Figure 6
Collar strengthening at base of mast

With this and other strengthening measures that were identified, it was possible to incorporate the vast majority of the OLE gantries in the Crossrail works, with only a handful requiring complete replacement.

Conclusions

The work delivered significant cost and programme savings to the Crossrail project by demonstrating that the majority of the 700+ existing OLE gantries affected by the TPS upgrade works were suitable

"A COMMON DEFECT WAS CORROSION WHERE WATER TENDS TO COLLECT AT THE BASE OF THE MAST"

for re-use or could be strengthened. This positive outcome was achieved by removing undue conservatism in the gantry assessment, by reducing the number of unknowns (geometry, loading, condition, material strength) while maintaining a suitable level of safety. The use of rigorous non-linear analysis for the slender structures was proven to give particular benefits when considering susceptibility to lateral torsional buckling.

In order to streamline similar processes in the future, a parametric approach to generating analysis models and deriving loading has been developed.

Acknowledgement

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Project team

Client: Network Rail
Structural engineer: BuroHappold
Geotechnical engineer: BuroHappold
Main contractor: Costain
Subcontractor: Keltbray

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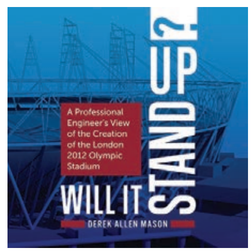
Review

Simon Pitchers enjoys this account of the creation of the London Olympic Stadium, both for its lessons on running successful projects and the fascinating facts that will enrich a dinner-party conversation.

Will it stand up?

Author: Derek Allen Mason
Publisher: Rethink Press
Price: £16.99
ISBN: 978-1-78133-280-1



In the future, few of our profession will use much mathematics in their daily working lives. Already, there are machine-learning systems out there that can solve problems by developing autonomously their own coded algorithms, enabling themselves to solve problems more efficiently and comprehensively than any human.

Those who think that, while machines may be able to crunch numbers, the human engineer's territory of conceptual design is unassailable by computers, are in for a surprise. Pessimists predict the death of our profession and the dawn of the 'engitect' or 'architeer'. Consensus is that the profession needs to respond to the change that is rapidly approaching, by broadening the knowledge of its membership. We need to understand more about non-structural factors that come into play in building projects, whether it's law, politics, M&E systems, architecture, finance or the drafting of a lease. We must step outside of the island that is our comfort zone, learn more about the wider world of building and claim new territory, otherwise machines will erode and reduce the sphere of our expertise. We need to become top of everyone's list of interesting people to invite to their dinner party.

That is why all grades of membership of our institution should read this book.

The book doesn't talk much about the future. It doesn't go into much geeky detail about the Olympic Stadium design. It is written in a clear, simple hand that is understandable to both a lay reader and a professional engineer. At 161 pages it is a small but worthwhile commitment of reading time. We all felt pride in 2012 during the London Olympics, but this back-story of success should make us, as a profession, even prouder.

Derek Mason, the author, was the third-party checker of the stadium design. He is also an athlete and a chartered structural engineer. He describes the fascinating story that starts from the very conception of Britain's bid for the 2012 Olympics right up to the legacy left by the Olympic Village.

Derek structures his book around seven principles of running a successful project:

- have a common goal
- develop a good detailed plan before you start
- develop a good communication strategy
- develop the ethos of good teamwork
- decide on the legacy from the outset
- develop robust checking procedures before you start
- make sure your project is fit for purpose.

These are put forward as a blueprint for success in most projects. They are very good principles too, though if you put two engineers in a room they would have a good argument as to whether these were the KEY principles. However, the overriding message that emerges is the strength of vision, leadership, purpose and collaboration that prevailed in this most brilliant of exemplar projects.

There are many horizon-broadening snippets of information contained in this book:

- An 80 000-seater stadium was built for the Olympics, but 55 000 of those seats were temporary.
- The weight of steel in the stadium is less than half of that used in comparable stadiums and one quarter of the steel used in the (nonetheless incredibly impressive) Beijing Olympic Stadium.
- The roof design was adapted to use steel tubes left over from a North Sea gas pipeline project in Russia and 52t of scrap metal, including confiscated knives and guns, were also used in the construction.
- A partial roof covering was found by wind-tunnel design to be optimal to ensure that wind speeds inside the stadium did not exceed 2m/s – otherwise athletic records could be affected.
- The building isn't clad – it's 'wrapped' in a lightweight fabric – a brilliant way of not over-egging the finish of a predominantly temporary structure.
- Because the opening ceremony had not been commissioned at the start of the design, the designers didn't know what loads would be imposed and had to guess.
- The stadium is built on an island site of industrial wasteland surrounded by rivers, canals and railway lines, completely occupying it and forcing facilities such as food outlets and shops to be grouped in 'pods' outside the stadium. This brings spectators closer to the action.

It's a fine testament to the project and its style reminded me slightly of that life-saving work *Engineering Mathematics* by Stroud and those reality property TV programmes where there is a re-cap after each commercial break. The repetition might irritate slightly, but it definitely helps the clarity of the message. It also means that you'll remember more for that all-important dinner party.

Simon Pitchers
BSc (Hons), CEng, MIStructE
Simon is a member of Council, a Trustee of the Institution, a media commentator and director of Craddys, a 50-person civil and structural consultancy.

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Topics of importance openly discussed

In defence of brainteasers as teaching aids

Emeritus Professor Jim Croll has joined the debate about the value of the 'And finally...' teasers in *The Structural Engineer*.

It is difficult to understand why Bill Harvey (Verulam, January 2018) should be quite so 'incandescent' at recent, very laudable efforts in the 'And finally...' pages aimed at encouraging improved understanding of structural behaviour. His focus on the solutions for the simple rigid jointed portal frame, covered in the October 2017 issue, seems particularly baffling.

For the past few decades, I have been using similar examples in an attempt to improve students' understanding of structural behaviour. This is as an antidote to our increased reliance in university courses on methods that form the basis of most structural analysis software, and the use of such software to perform analyses in practice. Encouraging students to draw and relate bending moment and deformation diagrams one to the other allows the mutual importance and interrelationships between the principles of force equilibrium and deformation compatibility to be thoroughly understood.

So, for the 'And finally...' problem of October 2017, reproduced here as Figure 1a, the deformation would most certainly take the form shown in Fig. 1a, as was given by the suggested solution B. This deformation line would relate directly to the bending moment diagram of Fig. 1b, so that moments are drawn on the convex face of the deformation line and, of course, are zero where the curvature changes sign at the points of contra-flexure.

Hogging moments at each end of the beam, resulting from the constraint provided by the columns to the end rotations of the beam, have the effect of lifting the free body bending moment to ensure the moments are zero at the points of contra-flexure. All of which helps the student to understand the relationships between force equilibrium and deformation compatibility. As long as the student is made aware that the deformations are drawn to highly exaggerated scales simply to make them visible, then surely no one can take issue with this?

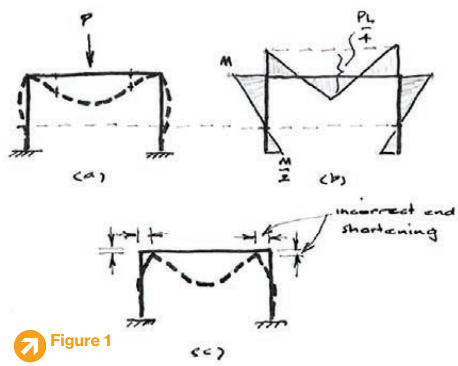


Figure 1

It is an understandable and common error that, when drawing deformation lines to these exaggerated scales, an almost unconscious allowance is made for the fact that were the deformations to be actually that large, then to preserve the original length of the beam and columns, the joints should be drawn, pulled inward and downwards, as shown in, say, Fig. 1c. So, the alternative solutions A and D are examples of this very common form of error.

But, of course, in designing our real structures, we should be limiting the level of deformation to around 1/250 of the span. Drawn at real scale, such deformations would be less than the thickness of the pencil lines and consequently invisible to the naked eye. So, at real scale, there would be no tendency to show the joints as shifting to compensate any perceived change in length.

Properly used, such examples can provide a powerful vehicle for the understanding of structural behaviour, as suggested by Martin Ashmead, and most certainly not the 'educational disaster' claimed by Bill Harvey.

I am currently putting the finishing touches to a book, based upon a course provided at University College London over the past 30 years, that makes extensive use of similar forms of qualitative analysis. This book, and the courses upon which it is based, go on to demonstrate how these physically based approximate analyses in the context of the static (lower bound) theorem of plasticity and ultimate state design provide powerful bases for the design of structures, reducing reliance on computer software and, by encouraging better structural understanding, aiding conceptual design processes. At the very least, they provide a method for interrogating the veracity of output from commercial software.

Solving the cranked beam problem

With his views firmly expressed in his previous contribution, Professor Croll has also commented on the trickier cranked beam problem discussed in the April issue.

Could I also proffer some comments on the resurrected cranked beam problem originally set by Martin Ashmead back in May 1982? Reading Bill Harvey's description of the system behaviour (Verulam, April 2018), reproduced as Figure 2a, I am afraid leaves me little wiser and clearly also left the editor rather perplexed.

Here is an example where the understanding being encouraged in the 'And finally...' pages can be usefully deployed. Replacing the vertical load (taken as $\sqrt{2}\,P$ so that the axial and normal components are approximately P) with its components normal and parallel to the axis of the cranked beam, it is possible to treat this as a pure membrane solution whereby the axial load component P is directly transmitted by column behaviour to be equilibrated by an equal and opposite reactive force P at the pinned lower support A, plus bending due to the normal component P at the top of the cranked section. The bending contribution would have

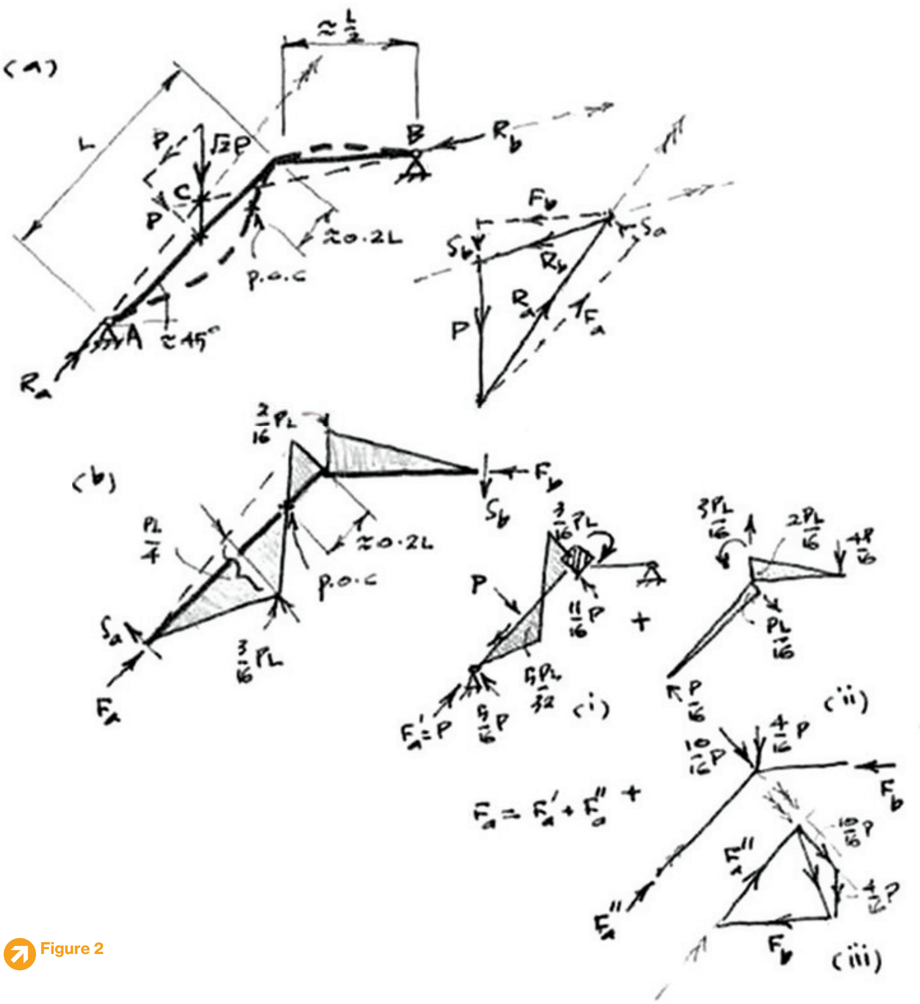


Figure 2

a deformation line as shown in Fig. 2a and an associated moment distribution as Fig. 2b.

Were it required for the behaviour to be converted into an approximate quantitative solution, perhaps suitable for the initial design of the system, it could be observed that the line of action of the reaction R_b must pass through the location on the inclined beam where the point of contra-flexure (poc) occurs. This in turn defines the point C where the reaction R_b intersects the line of action of the applied $\sqrt{2}$ P load. For this three-force system, equilibrium demands the three forces be concurrent, thereby uniquely defining the line of action of the reaction R_a, and the magnitudes of the reactive forces determined from the triangle of forces shown inset within Fig. 2a. Even with the qualitative nature of the sketch shown, it is possible to estimate the force magnitudes. Recognising these lines of thrust provides another way of interpreting the moments shown in Fig. 2b, since the moments simply represent the product of the end reaction times their normal distance from the line of thrust to the point on the frame axis.

How such qualitative methods can, with practice, be converted into quantitative behaviour is explained for this problem in the insets to Fig. 2b. Taking the joint between the inclined and horizontal beam to be conceptually rigid allows the moments to be given exact quantitative values for the propped cantilever, all as shown in inset (i). The out-of-balance clockwise joint moment, $3PL/16$, can then be eliminated by applying an anticlockwise moment to the original frame of the same magnitude, resulting in an anticlockwise rotation of the joint. Taking the horizontal beam to have roughly half the length of the inclined beam, this correction moment will then be distributed in the ratio 2:1 between the horizontal and inclined beams, as shown in (ii), resulting in the quantitative moment distribution of Fig. 2b, which is sufficiently accurate for it to form the basis of at least an initial design.

To complete the quantitative analysis for this frame, it is also necessary to apply the corrective joint forces at the knee, shown in sketch (iii). These are equal and opposite to the reactions developed at the knee as part of the idealisations illustrated in the insets (i) and (ii) of Fig. 2b. This then allows full specification of the axial forces in the members. Those with as much grey hair as myself will recognise the processes briefly described above as effectively the steps involved with the moment distribution method.

Professor Croll has taken this a step further than required in the original puzzle, which only demanded a prediction of the shape of the bending moment diagram. As ever, predictions of magnitudes by approximate methods reveal insights into performance. Professor Croll's methodology is at least partly based on an appreciation of system stiffness, which is the point Bill Harvey wanted to bring out.

Clarifying the cranked beam problem

Reader Melvin Hurst sends us a correction.

I am sure that I'm not the first to point out that, in your Verulam pages of April 2018, when you discuss Martin Ashmead's brainteaser of May 1982, there was a slip of the pen (or should that be slip of the keyboard?) on your part. At the beginning of the second paragraph you state that the poser is statically determinate if the supports at A and B are infinitely stiff in the horizontal direction.

However, the structure is only determinate if, as Bill Harvey notes, there is either a horizontal roller at A or a vertical roller at B. As it is, there are four unknown reactions, although the two horizontal ones must be equal and opposite, leaving only two statical equations to be formed from which the remaining three unknowns can be determined. This is, of course, the definition of an *in*determinate structure.

There is also a phrase missing from the last sentence of Bill Harvey's third paragraph (highlighted in italics): it should read '... and the force at A must be W vertically along with *a horizontal force* H, *with the resultant* inclined to meet force W where...'.

My view on the debate is that both equilibrium and stiffness must always be considered, although, for a determinate structure, equilibrium alone will suffice.

Melvin is, of course, correct. The teaser was indeterminate and his addition to Bill's contribution adds clarity. The essential message remains that actually the solution to the teaser is linked to the assumptions on support condition, which will lie between infinitely stiff and unrestrained, and the 'answer' is only as accurate as those assumptions. As Bill and Melvin state, stiffness must always be considered.

Water trapped in hollowcore floors

It's always nice to find an answer to a reader's queries. Here, based on his experience as Chief Engineer and Quality Manager of three hollowcore floor manufacturers, Cliff Billington replies to Sean Lightowler about water trapped in flooring units (June 2018).

The water in question is a result of rain. During prolonged exposure in the manufacturer's yard and on site, rain will penetrate the relatively porous top surface of the floor unit. However, the bottom zone, being under prestress, has no cracks and is pretty much impervious, and thus the water will accumulate in the hollow cores.

For plain, open-ended units, this is not a problem, as the water will simply run away. However, when used on a steel frame, the units often have a reduced-depth end to fit under the top flange of a beam. To compensate for the loss of cross-section, and to stabilise the remaining concrete, a concrete bung is formed (Figure 3). This bung obviously prevents water from escaping, and it is normal for the manufacturer to form drainage points to deal with this. These are formed by 'drilling' from above in line with the hollow cores, using a blunt drill bit, often a piece of rebar, so as not to damage the smooth casting bed.

The drilling does not go right through the bottom flange, and it is necessary to open the drain by piercing the remaining thin skin from above with a pointed piece of rebar. It is not recommended to drill at mid-span as Sean suggests. If a floor unit has a camber of say 30mm, then drilling at mid-span and releasing water will still leave 30mm of water at the ends, which can still cause staining. It is, in any case, not best practice to drill upwards with electrical tools into cascading water.

The hairline cracks are not caused by the water, although in extreme cases it is possible for a core completely full of water to freeze and split a unit in two. It is more likely that the cracks are already present and are simply made more obvious by water percolating through them and leaving dark streaks.

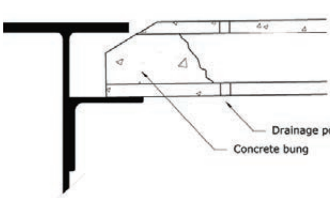


Figure 3

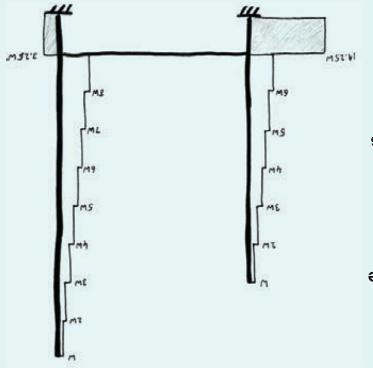
Well, there you have it.

AND FINALLY...

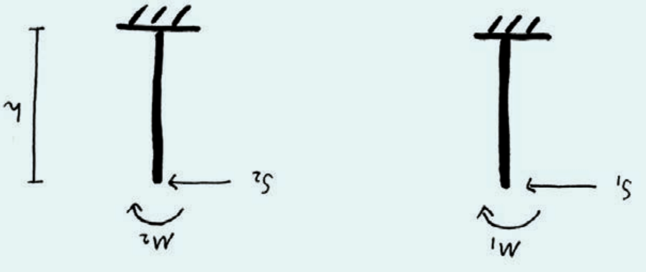
Answer to July's question

At first glance, it might be expected that the taller block would have the larger base shear, or that the shear would be shared equally between the two at ground level. In reality, the base shear in the shorter block is much higher. Note that it also has a larger base moment, so the foundation design could be more onerous for the shorter block.

The key is to remember that, due to the rigid podium, the deflection of each core must match at first-floor level. This deflection is made up of two components: the deflection due to the shear force and the deflection due to the moment. As the moment at first floor is higher in the taller tower, the shear force must be lower to compensate.



Consider a free body diagram cut just below the podium



Moments : $M_1 = (1+2+3+4+5+6)Wh = 21Wh$

$M_2 = (1+2+3+4+5+6+7+8)Wh = 36Wh$

Shears : $S_1 = 7W + P$

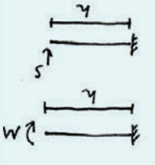
$S_2 = 9W - P$

where P is the force transferred by the podium at first floor

Compatibility : Rigid podium means first floor deflection in each core is equal

Deflection due to an end moment $= \frac{Mh^2}{2EI}$

Deflection due to a point load $= \frac{Sh^3}{3EI}$



$$\frac{21Wh^3}{2EI} + \frac{(7W+P)h^3}{3EI} = \frac{36Wh^3}{2EI} + \frac{(9W-P)h^3}{3EI} \quad (\times 6)$$

$$63W + 2(7W+P) = 108W + 2(9W-P)$$

$$63W + 14W + 2P = 108W + 18W - 2P$$

$$4P = 49W$$

$$P = 12.25W$$

$$S_1 = 7W + 12.25W = 19.25W$$

$$S_2 = 9W - 12.25W = -3.25W$$

Diary dates

Unless otherwise stated, evening technical meetings start at 18:00 (with refreshments from 17:30) and are free of charge to attend. Registration is required via events@istructe.org

History Study Group meetings start at 18:00 with refreshments from 17:30. Registration is not required except for the Annual Business Meeting held in January.

CPD courses are held at HQ unless otherwise stated. Times and costs vary.

Note that more current information may be available from the Institution website: www.istructe.org/events-and-awards

Regional Group Committee members should submit details of forthcoming events to: regionsupport@istructe.org

MEETINGS AT HQ

47–58 Bastwick Street, London EC1V 3PS, UK

CPD COURSES

(Held at Institution HQ unless otherwise stated)

Wednesday 4–Thursday 5 July
Temporary Works Design (2-day course)
Ray Filip
Members £475 + VAT; non-members £615 + VAT
10:00–17:30 (both days)
Booking: www.istructe.org/cpd-courses

Thursday 12 July
Temporary Works Appreciation
Ray Filip
Members £290 + VAT; non-members £370 + VAT
10:00–17:30
Booking: www.istructe.org/cpd-courses

Monday 3 September
Design and Analysis of Tall Buildings
Feng Fu
Members £290 + VAT; non-members £370 + VAT
10:00–17:30
Booking: www.istructe.org/cpd-courses

Tuesday 4 September
Conceptual Design for Structural Engineers – An Introductory Course (Online)
Oliver Broadbent
Members £290 + VAT (EU only); non-members £370 + VAT (EU only)
10:00–12:30 BST on four consecutive weeks
Booking: www.istructe.org/cpd-courses

Thursday 6 September
Eurocode 4: Composite Design
Dennis Lam
Members £290 + VAT; non-members £370 + VAT
10:00–17:30
Booking: www.istructe.org/cpd-courses

Friday 7 September
An Introduction to Structural Design for Fire Safety
Danny Hopkin and Ian Burgess
Members £290 + VAT; non-members £370 + VAT
10:00–17:30
Booking: www.istructe.org/cpd-courses

Monday 10 September
Concrete Basements (Liverpool)
Jenny Burridge
The Quaker Meeting House in Liverpool, 22 School Lane, Liverpool L1 3BT
Members £290 + VAT; non-members £370 + VAT
10:00–17:30
Booking: www.istructe.org/cpd-courses

Thursday 13 September
Dangerous Structures
Tim Parrett
Members £290 + VAT; non-members £370 + VAT
10:00–17:30
Booking: www.istructe.org/cpd-courses

Thursday 20–Friday 21 September
Understanding Structural Design (2-day course)
David Brohn
Members £475 + VAT; non-members £615 + VAT
10:00–17:30 (both days)
Booking: www.istructe.org/cpd-courses

Thursday 20 September
Wind Loading on Structures to EN 1991-1-4
John Owen
Members £290 + VAT; non-members £370 + VAT
10:00–17:30
Booking: www.istructe.org/cpd-courses

Thursday 20 September
Structural Steelwork Design to Eurocode 3 (Sheffield)
Dennis Lam
Engineering Training Centre, 4 Europa View, Sheffield Business Park, Sheffield S9 1XH
Members £290 + VAT; non-members £370 + VAT
10:00–17:30
Booking: www.istructe.org/cpd-courses

Friday 21 September
Dynamic Response of Wind-Excited Flexible Structures
Alessandro Palmeri and Giorgio Barone
Members £290 + VAT; non-members £370 + VAT
10:00–17:30
Booking: www.istructe.org/cpd-courses

Monday 24 September
Post-Tensioned Design and Detailing
Matthew Gilliver
Members £290 + VAT; non-members £370 + VAT
10:00–17:30
Booking: www.istructe.org/cpd-courses

Thursday 27 September
CDM2015 – A Practical Guide for Designers and Practice Managers
Liz Bennet
Members £290 + VAT; non-members £370 + VAT
10:00–17:30

Booking: www.istructe.org/cpd-courses

HISTORY STUDY GROUP

Tuesday 18 September
Jesse Hartley's bridges
Mike Chrimes

Tuesday 16 October
Tensegrity systems
Geoff Morrow

Tuesday 6 November
The Victoria Palace Theatre and recent work to it
Chris Boydell

REGIONAL GROUPS

Beds & Adjoining Counties

Tuesday 24 July
Annual Dinner and President's Visit
Faith Wainwright MBE
Combination Room, Peterhouse College, Trumpington Street, Cambridge CB2 1RD
Price: Members £30 (graduates £20; student members £10)
18:45: Pre-Dinner drinks. Dinner: 19:30
Booking: www.eventbrite.co.uk/e/istructe-beds-and-adjoining-counties-annual-dinner-and-president-visit-tickets-46869853995

Secretary: Shun Yu
(shuntingyu@gmail.com)

Chester and North Wales

Thursday 5 July
Fracking
Speaker: tbc
Holiday Inn Chester – South, Wrexham Road, Chester, Cheshire CH4 9DL
18:00 for 18:30

Thursday 6 September
Claims and Professional Indemnity
Speaker: tbc
Holiday Inn Chester – South, Wrexham Road, Chester, Cheshire CH4 9DL
18:00 for 18:30

Thursday 11 October
What being a Smart Contractor really means
Mo Perkins
Holiday Inn Chester – South, Wrexham Road, Chester, Cheshire CH4 9DL
18:00 for 18:30
Register: www.eventbrite.co.uk/o/institution-of-structural-engineers-chester-amp-north-wales-regional-group-13064193660

Secretary: James Drew
(James.drew@ramboll.co.uk)

East Midlands

Tuesday 11 September
Close Surface Reinforced Masonry Design
Speaker: tbc
Yew Lodge Hotel, Packington Hill, Kegworth, Derby DE74 2DF
18:00 for 18:30

Tuesday 16 October
Breathing Life into 1960s Buildings
Speaker: tbc
George Green Library, University of Nottingham
18:00 for 18:30

Secretary: Shaun Strugnell
(s.strugnell@hbpwconsulting.co.uk)

Hong Kong

Friday 6 July
Investigation of Water Seepage in Buildings
(joint meeting with Centre for Research & Professional Development)
Dr Eddie Lam
HJ301, The Hong Kong Polytechnic University, Hung Hom, Hong Kong
Cost: $1500
Language: Cantonese
Registration: 09:15; lecture 09:30 to 12:30; 14:00 to 17:00

Secretary: Eddie Lam
(email: cesslam@inet.polyu.edu.hk)

Lancashire and Cheshire

Friday 14 September
Annual Dinner and North West Awards
Faith Wainwright MBE
Midland Hotel, Manchester, 16 Peter St, Manchester M60 2DS
Price: Members and guests £75; student, graduate and retired members £45
(10% discount for members entering an award)
18:30 for 19:00

Tuesday 18 September
Why Not?
Neil Thomas
University of Manchester, Renold Building, PO Box 85, Sackville Street, Manchester M1 3BB
17:45 for 18:30

Thursday 4 October
Health Hazard Management within the Design Phase
(joint meeting with ICE Cheshire)
Lloyd Edmonds
The Centre, Birchwood Park, Warrington, Cheshire WA3 6AE
18:00 for 18:30

Secretary: Helen Fairhurst
(Helen.L.Fairhurst@sellafieldsites.com)

Midland Counties

Tuesday 25 September
Restraint of Reinforced Concrete Elements
John Forth
University of Birmingham, Building Y3 – Mechanical and Civil Engineering Building, Room G34, Edgbaston Birmingham B15 2TT
18:00 for 18:30
Contact: Mitchell Gray
(mitchell.gray@entrust-ed.co.uk)

Secretary: Richard Davis
(richardavis@hotmail.co.uk)

North Thames

Tuesday 3–Wednesday 4 July
Seismic analysis and design of structures (2- half day course)
Piroozan Aminossehe
Room 307, Skempton Building Imperial College, Exhibition Rd, London SW7 2AZ
Members £312; non-members £372
13:30–18:00 (both days)

Thursday 12 July
Summer Boat Party
Tower Millennium Pier, Lower Thames St, London EC3N 4DT
(Guests must arrive at the pier at 17:30 for security check)
Dress: Smart casual (red and green colours preferred)
Price: Student & graduate members £10.90; members £15.90; non-members £19.90 (tickets limited–first come first serve basis)
17:45–21:45
Register/tickets: IStructEParty18.eventbrite.co.uk

Sunday 30 September
Heritage Walk–St Pancras Station by 150
(joint event with HKIE UK Chapter)
Lester Hillman
Eastern end of the raised forecourt in front of St Pancras Hotel, Euston Road, London NW1 2AR
11:00–13:00
Register: KingsX150.eventbrite.co.uk

Thursday 11 October
AGM and Technical Talk on Principal Place
Nello Petrioli and Brandon Eastwood
The Institution of Structural Engineers HQ, 47-58 Bastwick Street, London EC1V 3PS
17:30 Reception for 18:00
Register: WSP-PPR.eventbrite.co.uk

Secretary: Simon Leung
(simon.leung@simonleung.co.uk)

Northern Ireland

Tuesday 25 September
Expert Witness Cases–Recurring Themes and Lessons Learned
Don McQuillan
Room PFC/02/026, Peter Froggatt Centre, Queen's University Belfast, Belfast BT7 1NN
18:15

Secretary: Andrea Johnston (andrea.johnston7@btinternet.com)

Scottish

Sunday 9 October
CPD Seminar: Specifying Structural Steelwork (note new date)
David Brown
Stirling Court Hotel, University of Stirling, Airthrey Rd, Stirling, FK9 4LA
Cost: £60.00 Graduate Members; £70.00 Members; £80.00 non-members
14:30 for 15:00. Finishes at 19:00
Register: https://www.eventbrite.co.uk/e/seminar-specifying-structural-steel-tickets-43312556021 or seminars@istructescotland.org

Secretary Danny Wright (danny_cross_wright@live.co.uk)

Singapore

Friday 21 September
Working Together for a Creative and Collaborative Future (IStructE Presidential Address) (IStructE/IES Joint Committee event)
Faith Wainwright MBE
National University of Singapore Engineering Auditorium
Cost: Free of charge to IStructE and IES members
19:00–21:00 (light dinner served at 18:00)
Registration: On a first-come-first-served basis, limited to 150 participants (completed registration form must be submitted)
Contact: Singapore-IStructE@ies.org.sg

Secretary: Wijaya Wong (wijaya.wong@outlook.com)

South East Counties

Tuesday 25 September
2017 IStructE Award Winner and Annual General Meeting
Speaker: tbc
Croydon Park Hotel, 7 Altyre Road, Croydon CR9 5AA
18:00 for 18:30

Secretary: Eric Li (eric.li@walsh.co.uk)

Surrey

Sunday 1 July
Summer visit – Portsmouth Historic Dockyard
Professor Eddie Bromhead
National Museum of the Royal Navy, Visitor Centre, Victory Gate, HM Naval Base, Portsmouth PO1 3LJ
Tickets: Adult £29.00; Senior £25.00; Child 16 and under £18.50
10:00 (meet at Main Gate) –17:30
Contact: Nigel Westwood (nwestwood@blueyonder.co.uk)

Thursday 5 July
Summer Pub Quiz
Victoria Pub, Surbiton, Surrey KT6 4JT
19:00–21:00
Contact: Natalja Petkune (natalja.petkune@gmail.com)
Register: www.eventbrite.co.uk/e/summer-pub-quiz-tickets-47013808567

Monday 10 September
Benefit from Confidential Reporting on Structural Safety
Dr Paul McNulty
The Conference Centre, Atkins, Woodcote Grove, Ashley Road, Epsom, Surrey KT18 5BW
17:30 for 18:00

Secretary: Ruslan Koutlukaev (ruslan.kutlukaev@gmail.com)

Wales

Friday 3 August
IStructE and ICE Young Professionals Summer Ball 2018, Cardiff
The Angel Hotel, Castle Street, Cardiff CF10 1SZ
Price: £45/person incl. VAT
Dress: Black tie
19:00–01:00. (Arrival: 19:00 drinks reception)
Contact: Alice Richards (alice.richards@wsp.com)
Booking: www.eventbrite.co.uk/e/ice-and-istructe-young-professionals-summer-ball-2018-tickets-46075661544?aff=erelexpmlt

Tuesday 9 October
Four Elms on Four Elms Road
Pat Ruddock and Tom Martin
Cardiff University Engineering Department, Faculty Lecture Theatre, Trevithick Building, The Parade, Cardiff CF24 3AA
17:30 for 18:15

Secretary: Pierre Grigorian (pierre@phg-consulting.com)

Western Counties

Thursday 20 September
Flood resilience
Fiona Gleed
Bath University, Claverton Down, Bath BA2 7AY
18:00 for 18:30

Secretary: Jason Walker (jsrwalker@gmail.com)

Yorkshire

Wednesday 4 July
Thermally Active Building Systems (TABS)
James Griffiths
Cedar Court Hotel, Denby Dale Road, Wakefield WF4 3QZ
18:00 for 18:30

Secretary: Farzad Neysari (tel: 0113 2818282; email: farzadneysari@aol.com)

INTERNATIONAL CONFERENCES

Adelaide, Australia

Tuesday 25–Friday 28 September
ASEC 2018: Australasian Structural Engineering Conference
Adelaide Convention Centre, North Terrace, Adelaide SA 5000, Australia
Web: aseconference.org.au

California, USA

Wednesday 24–Saturday 27 October
DFI 43rd Annual Conference on Deep Foundations
Hilton Anaheim, Anaheim, California, USA
Web: www.dfi.org/dfieventlp.asp?13325

Dubai/Abu Dhabi, UAE

Saturday 20–Thursday 25 October
Polycentric Cities: The future of vertical urbanism
JW Marriott Marquis Hotel Dubai
Web: ctbuh2018.org/program/conference-synopsis/

Cairo, Egypt

Saturday 24–Wednesday 28 November
GeoMEast 2018 International Congress and Exhibition: Sustainable Civil Infrastructures: Structural Integrity
Mena House Hotel, Cairo, Egypt
Web: www.geomeast2018.org

Spotlight on Structures

Research Journal of The Institution of Structural Engineers

In this section we shine a spotlight on papers recently published in *Structures* – the Research Journal of The Institution of Structural Engineers.

***Structures* is a collaboration between the Institution and Elsevier, publishing internationally-leading research across the full breadth of structural engineering which will benefit from wide readership by academics and practitioners.**

Access to *Structures* is free to Institution members (excluding Student members) as one of their membership benefits, with access provided via the 'My account' section of the Institution website. The journal is available online at: www.structuresjournal.org

Structures prizes announced

This month we are delighted to announce the winners of the 2018 *Structures* prizes. Congratulations to Andrew Liew, Leroy Gardner and Philippe Block on winning the 'Best Research Paper Prize' for their paper on 'Moment-Curvature-Thrust Relationships for Beam-Columns' published in Volume 11, August 2017.

We also congratulate Chris Burgoyne and Owen Mitchell on winning the 'Best Research into Practice Paper Prize' for their paper on 'Prestressing in Coventry Cathedral' published in Volume 11, August 2017.

Both papers will be free to read until 7 September 2018.

The annual *Structures* prizes are judged by The Institution of Structural Engineers Research Panel and supported by Elsevier. Each prize carries an award of £500.

Best Research Paper

Moment-Curvature-Thrust Relationships for Beam-Columns

Andrew Liew[a], Leroy Gardner[b] and Philippe Block[a]
[a] Institute of Technology in Architecture, ETH Zurich, Switzerland
[b] Department of Civil and Environmental Engineering, Imperial College London, UK

Moment–curvature–thrust relationships (M–κ–N) are a useful resource for the solution of a variety of inelastic and geometrically non-linear structural problems involving elements under combined axial load and bending. A numerical discretised cross-section method is used in this research to generate such relationships for I-sections, rectangular box-sections and circular or elliptical hollow sections. Moment–curvature–thrust curves are derived from axial force and bending moment interaction curves by pairing the curvatures and moments for a given axial load level. These moment–curvature–thrust curves can be transformed into various formats to solve a variety of structural problems. The gradient of the curves is used to find the materially and geometrically non-linear solution of an example beam-column, by solving numerically the moment–curvature ordinary differential equations.

The full paper is available at https://doi.org/10.1016/j.istruc.2017.05.005.

Best Research into Practice Paper

Prestressing in Coventry Cathedral

Chris Burgoyne[a] and Owen Mitchell[b]
[a] Dept of Engineering, University of Cambridge, UK
[b] Mott MacDonald, Cambridge, UK

Coventry Cathedral was completed in the early 1960s and has some prestressed concrete elements to resist lateral thrust from the roof. Other prestressed structures of a similar age have had corrosion problems and this has drawn attention to the fact that there is little publicly available information about the structural system at Coventry. This paper addresses that issue and is in three sections. The first summarises the four different prestressing systems in the Cathedral and estimates the amount of prestress and its purpose in each location. Although there is no evidence of corrosion in the building at the moment, it is impossible to inspect the existing tendons, so the second section considers what might happen to the structure if corrosion of the tendons were to occur. It is concluded that very little warning of failure would be given, which would be especially important for the tendons over the baptistry window and those in the nave ties. The final section considers what could be monitored to give as much warning as possible about future problems. The effects of loss of an individual tendon, which would not by itself be sufficient to cause failure of the structure, would cause only very small strains that would be difficult to distinguish from the background strains caused by temperature change. Many of the principles discussed in the second and third sections would be applicable to many other prestressed concrete structures.

The full paper is available at https://doi.org/10.1016/j.istruc.2017.04.003.

Big Foot supporting the ventilating of the underground construction workers

Big Foot Systems has supplied HD Cubes, HD Beams, Bespoke Frames and Custom Heavy Duty (HD) Frames to support large amounts of ventilation equipment being used during the construction of a new railway line. During fit out and track laying in the tunnels large air handling units (AHUs) and hundreds of metres of ducting all required support to provide fresh air to the construction workers before the permanent infrastructure was installed.

The chosen solution was to fit AHUs at either side of the central section of the site, requiring very specific supports. Custom HD Frames supported two 100m³/s AHUs that were installed in the old rail tunnels with extremely limited clearance. Hundreds of metres of 3m x 3m ductwork was supported by Custom HD Frames to connect the AHUs to the outside.

The biggest challenge was supporting a 60,000kg, 25 x 8 x 5m 200m³/s AHU sited outside, with a 6m x 6m duct rising 10m to the building entry point. Dozens of Big Foot's HD Cubes and Beams comprised the ground level support, with a custom steel structure holding the high-level ductwork, for which a full FEA analysis was carried out to ensure stability.

Further information: Big Foot Systems
(email: enquiry@bigfootsupport.com; tel: +44 (0)1323 844355)

Non-penetrative support system installed for rooftop plant on wind farm's onshore substation

Big Foot Systems has supplied rooftop chiller plant and ductwork support as well as safe access and high-level maintenance platforms to a £25M project to construct a high voltage onshore substation as part of the new 'Hornsea Project One' offshore wind farm, near Immingham, Lincolnshire.

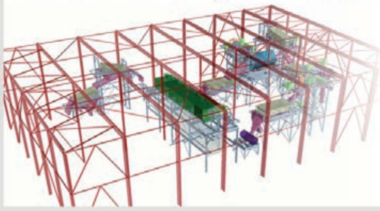
The flat roof of the substation was single ply PVC membrane which posed a challenge. It was feared this installation would damage the roof while penetrations in the single ply membrane would risk both potential leaks and thermal bridging. Normally penetrative supports are used for chiller and safe access installations on flat roofs. Principal contractor Balfour Beatty specified Big Foot's non-penetrative supports and Big Foot Technical worked with Balfour Beatty to design a solution which combined the maintenance platforms with plant and duct support.

Due to the restriction of space on the roof, some of the walkways had to be cantilevered off the edge of the building and so calculations were done to use the correct amount of ballast, ensuring that the walkways were safe for contractors.

Further information: Big Foot Systems
(email: enquiry@bigfootsupport.com; tel: +44 (0)1323 844355)

Telephone: +44 (0)20 7880 6217 Email: callum.nagle@redactive.co.uk

ANALYSIS & DESIGN



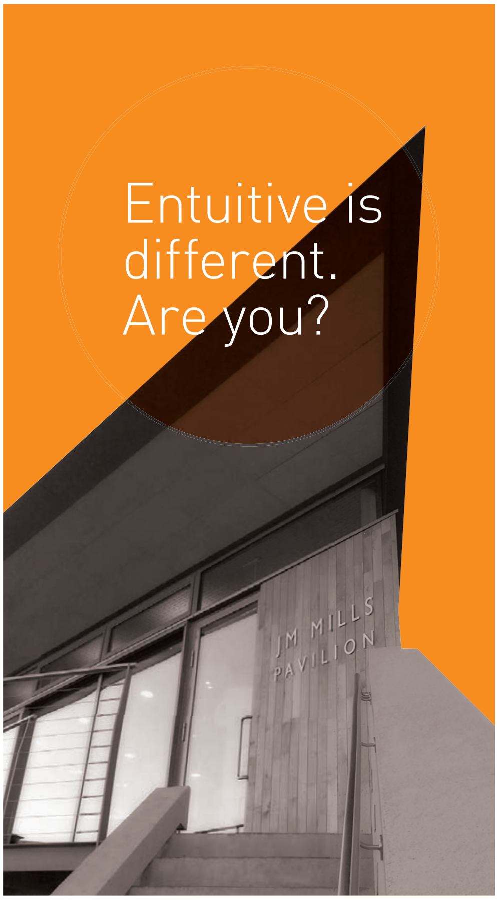
ANALYSIS & DESIGN

SOFTWARE

REINFORCED CONCRETE DETAILING

STRUCTURAL DESIGN & CAD DRAWING

REINFORCED CONCRETE DETAILING



SENIOR STRUCTURAL ENGINEER London, UK

Based in London, this role offers the chance to lead projects on the technical side with a high level of client and architect exposure. The Senior Engineer will work across the diverse projects that our UK team undertakes, ranging from bespoke residential projects to architecturally prominent new buildings and public artworks, as well as our wider international work.

Join us for an opportunity to...

Deliver diverse and challenging projects + Pioneer areas of technical excellence + Forge new client relationships + Grow our practice in the UK

For more information, please visit our careers page:
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Entuitive is an award-winning international structural engineering and building envelope consultancy. Check us out.

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ENTUITIVE

ABACUS DESIGN ASSOCIATES

PRINCIPAL STRUCTURAL ENGINEER / DIRECTOR DESIGNATE

A rare and exciting opportunity exists for a successful, ambitious and energetic Principal Structural Engineer to join the Abacus Design team at Loughborough, with a view to swift promotion to Director.

Abacus Design, an employee-owned practice, undertakes structural and civil engineering design, inspection and assessment services for a wide range of clients within the East Midlands and throughout the UK.

Short-listed candidates will be asked for interview which will also include a short written examination and may be held in two stages.

The successful applicant will be an ambitious, hard-working and dedicated Chartered Structural Engineer with at least 3 years' relevant experience since becoming chartered. Evidence of outstanding communication skills is also required.

An attractive salary (including a non-contractual bonus) will be offered to the successful applicant.

HOW TO APPLY

Candidates should send their CV and covering letter to **recruitment@abacusdesign.co.uk**

BECKETT RANKINE
Marine Consulting Engineers

Graduate Maritime Civil Engineer

Beckett Rankine specialise in bespoke engineering design solutions for the maritime sector. We are a friendly, independent, central London engineering consultancy. Our dynamic and inclusive culture encourages free thinking and we strive to deliver innovative but appropriate solutions to a global portfolio.

We need a motivated, versatile candidate who can adapt to a variety of projects undertaking tasks that include;

- Analysis of structures and the production of calculations,
- Production of sketches and drawings,
- Solving of engineering problems,
- Ensure project work is carried out within agreed deadlines,
- Monitoring and checking of works on site,
- Checking contractor designs and fabrication drawings.

Required attributes;

- At least a 2:1/Merit MEng in Civil/Structural Engineering or equivalent,
- A passion for maritime projects,
- A team player with the ability or potential to manage others,
- Good communication skills, both written and verbal, in English,
- IT skills including MS Office, AutoCad, Tedds and structural analysis software,
- Knowledge of CDM 2015 Regulations, H&S regulations, ISO 9001 & 14001.

Graduate training scheme:

- fully accredited Institution of Civil Engineers training scheme,
- encouragement and support towards becoming a Chartered Engineer.

Our training scheme has an excellent success rate as a result of the type and variety of project experience we are able to provide combined with our mentoring system which is focused on your success.

Please send your CV and covering letter to careers@beckettrankine.com
For more detail about what we do see www.beckettrankine.com

SENIOR STRUCTURAL ENGINEER
Coventry

FRANK W. HAYWOOD AND ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS

Frank W. Haywood & Associates are a busy award winning Midlands based consultancy, who specialise in providing structural services for medium to large prestigious buildings for private clients.

We have an immediate requirement for a Chartered or very near chartered Senior Structural Engineer who is capable of working under their own initiative, running projects from concept to completion. They will be expected to liaise with clients and architects, be self-motivated, comfortable working on their own producing design packages including drawings or working with and mentoring a junior technician / draughtsman.

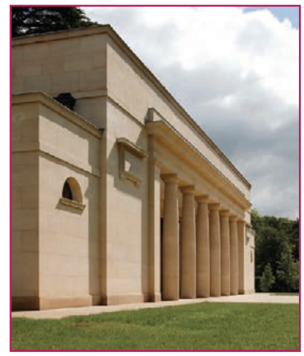
The projects on which we work are generally constructed predominantly from masonry including finely detailed stonework, with secondary elements in steel and concrete where required. The position available is considered to be more fitting to Engineers from a small consultancy background wishing to work on slightly larger projects as opposed to Engineers used to large multi storey framed commercial or residential projects. Thus, technically competent Engineers with a broad background of building structures in all materials and experience of handling projects on site from surveying through to inspections, are preferred.

A good team player with fluent English, a proven track record and a full UK driving licence is required.

The applicant will need to be fully conversant with 2D AutoCAD, Microsoft Office and a key understanding of design analysis programs is essential.

Salaries are negotiable dependant on qualifications and experience.

Please send your CV and covering letter, including salary expectations to:
paul@fwhaywood.co.uk



www.fwhaywood.co.uk

wardell armstrong

Associate Director Structural Engineering Glasgow

This is an exciting opportunity at a senior leadership level to develop Structural Engineering services, working closely with our existing team in Glasgow and the Civil & Structural Engineering, Geotechnical and Landscape Architect teams in Edinburgh serving markets in Scotland. This is a new position with an excellent opportunity for progression. The successful candidate will ideally possess Chartership with the Institute of Civil Engineering or Structural Engineering, and will play a significant role in developing our Glasgow Office. The role will also collaborate with structural engineers based in our 11 other offices across the UK.

Please upload your CV at wardell-armstrong.com
or email hr@wardell-armstrong.com

PART-TIME, HOME BASED
CHARTERED STRUCTURAL ENGINEER

We are a small structural and civil consultancy practice established over 30 years ago. We need one or two chartered structural engineers based in certain areas of London and the South-East to handle jobs local to them on a self-employed, sub-consultant basis but covered by our professional indemnity and public liability insurance.

You would be working from your own home and can choose the hours you work and how many jobs you take on. The work involves mainly inspections and reports on building defects and structural design of alterations and extensions, both domestic and commercial. It would suit a person who's taken early retirement but wishes to keep their hand in. Must be chartered and able to write good English. We pay around £200 for a typical inspection and report, and £35 per hour for design work. No formal contract is involved.

All new enquiries and invoicing are handled at head office, so you would not need to get involved in red tape. We would expect you to have a computer, an A3 printer and basic survey equipment. Autocad is preferable but not essential.

If you are interested, please see our web site: www.abbott-holliday.co.uk and write to Peter Holliday at Wissenden Oast, Bethersden, Kent, TN26 3EL or email peter@abhol.co.uk.

And finally...

The place to test your knowledge and problem-solving ability.
If you would like to submit a quiz or problem, contact tse@istructe.org

This month we present a reader contribution from Ewan Macpherson on shear forces. The answer can be found on page 98.

Question

A building is to be formed of a seven-storey block and a nine-storey block, joined together by a rigid podium at first-floor level (Figure 1). Horizontal wind loads act at every storey and each block is stabilised by its own core. The same core cross-section has been used for both blocks.

Complete the shear force diagram shown in Figure 2 for ground to first-floor level.

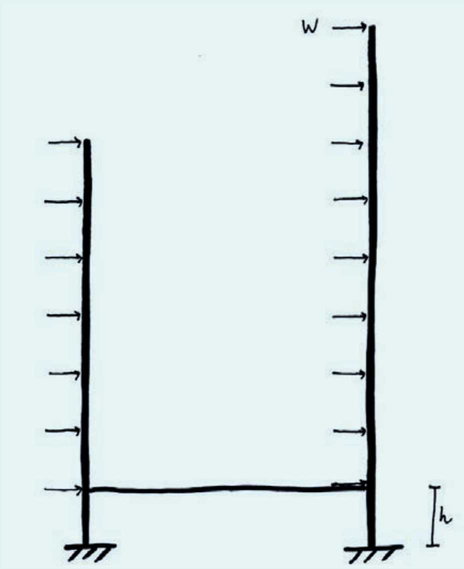


Figure 1

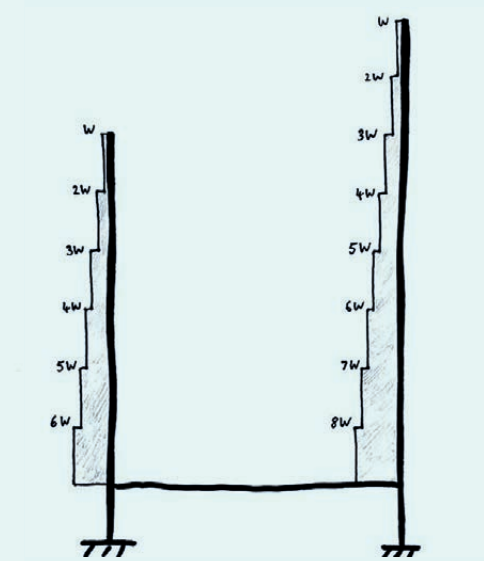


Figure 2

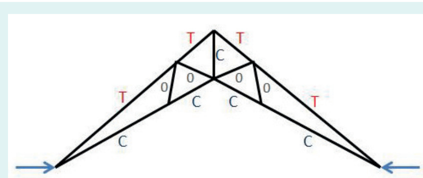
Submit your problem for consideration to tse@istructe.org. The author of each published problem will receive a single e-book of their choice from the Institution's current catalogue.

Submit your own teaser

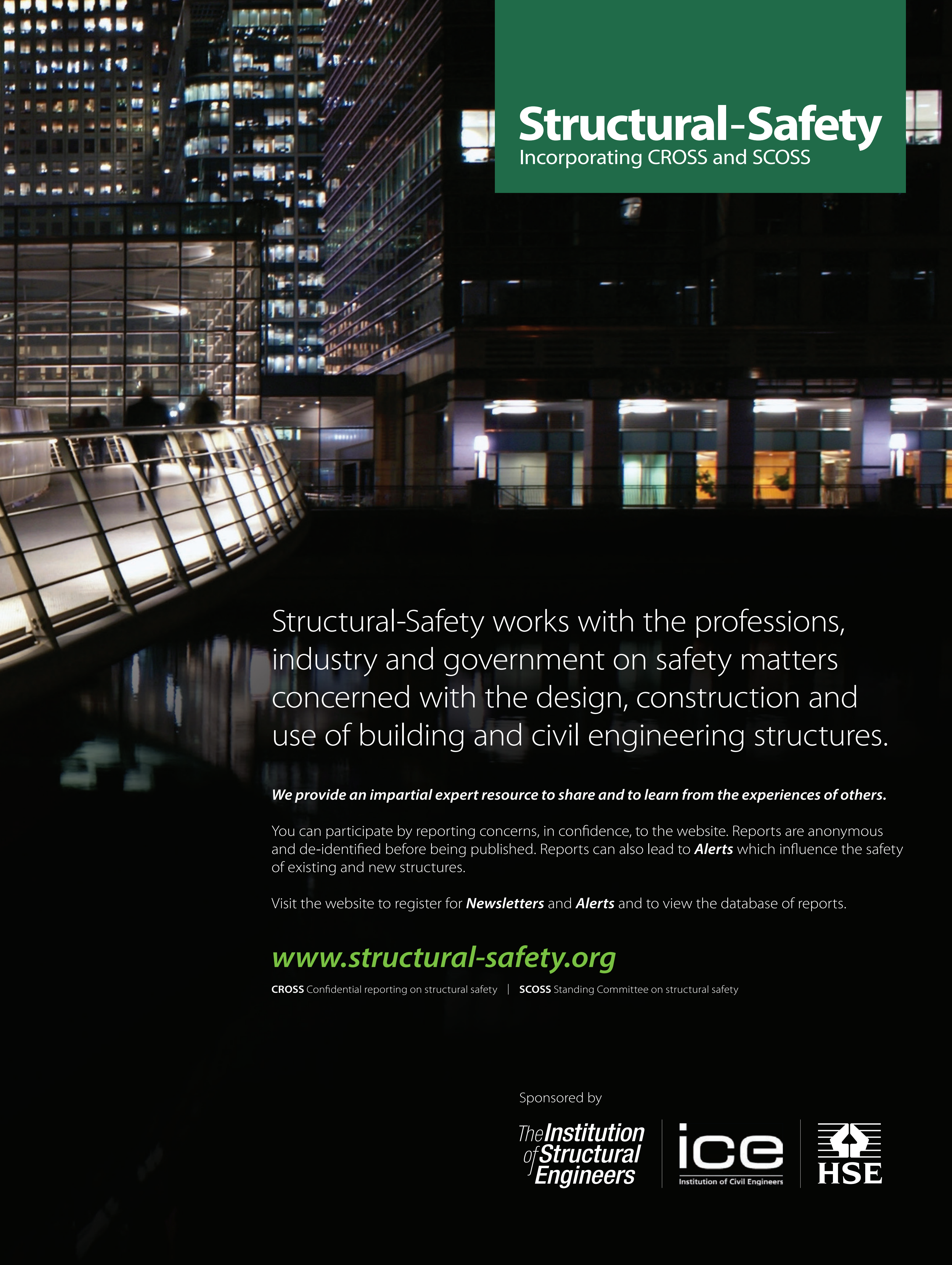
The Structural Engineer invites contributions to the popular 'And finally...' brainteaser section of the publication.

Readers are invited to submit a simple problem addressing an aspect of fundamental structural understanding. Problems will ideally be in the form of a question and multiple-choice answer, with accompanying diagram(s).

The author of each published problem will receive a single e-book of their choice from the Institution's current catalogue.



Submit your brainteaser to tse@istructe.org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