Universidad de Oviedo
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Aspectos geotécnicos de la construcción de una nueva línea de metro subterránea en la cuenca terciaria de Londres

Programa de doctorado: Biogeociencias

Emilio Linde Arias

Universidad de Oviedo
Universidá d'Uviéu
University of Oviedo

RESUMEN DEL CONTENIDO DE TESIS DOCTORAL

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2.- Autor	
Nombre: Linde Arias	DNI/Pasaporte/NIE:
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RESUMEN (en español)

Los incidentes originados por causas geotécnicas pueden llegar a ocasionar retrasos, sobrecostes e incluso la pérdida de vidas humanas en los proyectos de ingeniería civil. La interacción terreno-estructura depende, principalmente, de las propiedades de los materiales geológicos afectados y un diseño satisfactorio requiere de un preciso modelo geológico y geotécnico de partida dado que cualquier error puede cuestionar la estabilidad de la estructura. Esto resulta, si cabe, aún más importante en el caso de las excavaciones subterráneas, como por ejemplo los túneles, que conllevan numerosos riesgos adicionales por la interacción con el espacio subterráneo y la incertidumbre asociada con el mayor o menor grado de conocimiento geológico. En un ámbito urbano, como Londres, los riesgos se incrementan, dada la posible afección a otras estructuras previas del entorno, bien sean subterráneas o superficiales.

La investigación que conforma este trabajo de tesis doctoral ha pretendido aportar novedades al conocimiento de la geología y comportamiento geotécnico de los materiales cenozoicos de la cuenca de Londres, a partir de: (i) la recopilación de la información obtenida a partir de las investigaciones geológicas y geotécnicas desarrolladas durante diversas excavaciones subterráneas y (ii) el análisis del comportamiento de los materiales ante los cambios tensionales debido a la excavación de los túneles, en especial en la formación Arcilla de Londres y en los niveles arenosos del Grupo Lambeth.

En particular, las principales conclusiones son:

- La investigación geotécnica e hidrogeológica en el sector la península de Limmo, al este de Londres, a la que se suman los datos obtenidos durante las excavaciones para diferentes estructuras subterráneas en esta zona, confirman la presencia de un sistema de fallas que afectan a los materiales del Paleógeno. Se interpreta que estas estructuras son resultado de una inversión tectónica de una cuenca pull-apart transformada en una estructura de flor positiva, lo que resulta consecuente con las hipótesis previas que apuntaban a una compartimentación de la cuenca, que se manifiesta en superficie a través de la red de drenaje.
- En la Formación Arcillas de Londres se produce un aumento de su fisuración en el entorno de las fallas reduciéndose, en estas zonas, de forma significativa los parámetros resistentes representativos de estos materiales. Esto conlleva una problemática asociada en las obras subterráneas, constatada en los frentes de excavación y en las destrozas, si bien no condiciona su integridad estructural.
- Afrontar la excavación de capas de arenas saturadas del Grupo Lambeth en frente abierto en cualquier diámetro de excavación (cavernas y galerías) es viable siempre que se adopten medidas de drenaje activo de las mismas. Debido al reducido caudal de bombeo requerido y de la distribución espacial en capas lenticulares, es necesario utilizar eyectores si se desea hacer desde superficie. La capacidad de realizar un bombeo desde el frente de excavación está limitada por la presión sub artesiana, siendo recomendable reducir esta presión a valores menores de 1 bar.
- El drenaje de las capas de arenas del Grupo Lambeth puede ocasionar asientos en superficie, cuya magnitud depende del espesor de la capa drenada, de su módulo de deformación y de su permeabilidad, aspectos a fijar en futuros proyectos. En los casos

analizados se documentaron asientos reducidos, de 1mm/1m de rebaje, que no comprometen la seguridad de las estructuras superficiales.
• La utilización de sistemas de bombeo para la excavación de túneles precisa de un mantenimiento intensivo para evitar ascensos repentinos del nivel freático debido a pérdidas de presión por fugas o problemas de suministro eléctrico, siendo aconsejable disponer de sistemas redundantes y de monitorización.
• Se constata la importancia de actualizar los modelos geotécnicos correspondientes a proyectos de obras subterráneas deben de actualizarse permanentemente con la información obtenida durante el transcurso de las obras. Este aspecto ha quedado ejemplificado la mejora del modelo geológico a partir del uso de datos granulométricos y los proporcionados por los taladros de prospección y los manómetros, sin necesidad de detener el avance de la tuneladora o de acceder al frente de excavación en condiciones hiperbáricas.

RESUMEN (en Inglés)

Incidents caused by geotechnical causes can lead to delays, cost overruns and even loss of life in civil engineering projects. The soil-structure interaction depends, mainly, on the properties of the affected geological materials and a satisfactory design requires a precise geological and geotechnical model since any error can question the stability of the structure. This is, if possible, even more important in the case of underground excavations, such as tunnels, which carry numerous additional risks due to the interaction with the underground space and the uncertainty associated with the degree of geological knowledge. In an urban environment, the risks increase, given the possible affection to other pre-existing.
Although the geological knowledge of the London Basin has been developed since the 19th century due to the extraordinary urban development of numerous infrastructure projects, there are still some uncertainties.

The investigation that makes up this doctoral thesis has sought to improve the knowledge of geology and geotechnical behaviour of Cenozoic materials by: (i) compiling the information obtained from the geological and geotechnical investigations carried out during various underground excavations and (ii) analysing the behaviour of the materials in the face of tension changes due to the excavation of the tunnels, especially, London Clay Formation and the sand lenses in the Lambeth Group.

In particular, the main conclusions are:

- The research in the Limmo peninsula sector, to which are added the data obtained during excavations for different underground structures in this area, confirm the presence of a fault system that affects Paleogene materials. These structures are interpreted to be the result of a tectonic inversion of a pull-apart basin transformed into a positive flower structure, which is consistent with previous hypotheses that pointed to a compartmentalization of the basin, which manifests itself on the surface through the drainage network.
- In the London Clay Formation an increase of fissuring has been detected, related to the presence of faults and therefore, reducing, the representative resistant parameters of these materials significantly. This entails instability of the face during the excavation.
- The excavation of saturated sand layers of the Lambeth Group in open front in any excavation diameter (caverns and galleries) is viable as long as active drainage measures are adopted. Due to the reduced pumping flow required and the spatial distribution in lenticular layers, it is necessary to use ejectors if you want to do it from the surface. The ability to pump from the excavation front is limited by sub-artesian pressure, and it is recommended to reduce this pressure to values less than 1 bar.

SR. PRESIDENTE DE LA COMISIÓN ACADÉMICA DEL PROGRAMA DE DOCTORADO EN BIOGEOCIENCIAS

INDICE

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1 INTRODUCCIÓN

1.1 Justificación de la investigación

Los incidentes originados por causas geotécnicas pueden llegar a ocasionar retrasos, sobrecostes e incluso la pérdida de vidas humanas en los proyectos de ingeniería civil (Hoek y Palmeiri, 1998). La interacción terreno-estructura depende, principalmente, de las propiedades de los materiales geológicos afectados y un diseño satisfactorio requiere de un preciso modelo geológico y geotécnico de partida (Konstantis et al., 2016), dado que cualquier error puede cuestionar la estabilidad de la estructura. Esto resulta, si cabe, aún más importante en el caso de las excavaciones subterráneas, como por ejemplo los túneles, que conllevan numerosos riesgos adicionales por la interacción con el espacio subterráneo y la incertidumbre asociada con el mayor o menor grado de conocimiento geológico. El principal problema suele residir en la variabilidad intrínseca de las condiciones geológicas (van Staveren, 2006). En un ámbito urbano los riesgos se incrementan, dada la posible afección a otras estructuras previas del entorno, bien sean subterráneas o superficiales.

Según un informe de la aseguradora Munich-Re (Wannick, 2007) las obras de túneles han sufrido más perdidas que cualquier otro tipo de proyecto de construcción; así, desde la década de los 90, la industria de la perforación de túneles ha sufrido pérdidas de más 600 millones de dólares (Tabla 5) como consecuencia, principalmente, de problemas geotécnicos en las excavaciones.

Tabla 1.- Principales siniestros en proyectos de túneles en el mundo en el periodo 1994-2007.

Año	Proyecto	Causa	Valor ($ USA)
1994	Great Belt Tunnel, Fünen-Seeland, Dinamarca	Fuego	33
1994	Heathrow Express Link, Londres, Reino Unido	Colapso	141
1994	Ferrocarril subterráneo, Munich-Trudering, Alemania	Colapso	4
1994	Ferrocarril subterráneo, Taipei, Taiwan	Colapso	12
1995	Ferrocarril subterráneo, Los Ángeles, Estados Unidos	Colapso	9
1995	Ferrocarril subterráneo, Taipei, Taiwan	Colapso	29
1999	Túnel de Saneamiento de Hull, Reino Unido	Colapso	55
1999	TAV, Bolonia-Florencia, Italia	Colapso	9
1999	Túnel Bolu, Gümüsova–Gerede, Turquía	Terremoto	115
2000	Ferrocarril subterráneo, Taegu, Corea del Sur	Colapso	24
2000	TAV, Bolonia–Florence, Italia	Colapso	12
2001	Ferrocarril subterráneo, Tseung Kwan , Hong Kong	Colapso	N/A
2002	TAV, Taiwan	Colapso	30
2002	Túnel Socatop, París, Francia	Fuego	8
2003	Ferrocarril subterráneo, Pearl Line, Shanghai, China	Colapso	80
2004	Ferrocarril subterráneo, Pearl Line, Shanghai, China	Colapso	N/A

Año	Proyecto	Causa	Valor ($ USA)
2005	Ferrocarril subterráneo, Línea Circular, Singapur	Colapso	N/A
2005	Ferrocarril subterráneo, Orange Line, Kaohsiung, Taiwan	Colapso	N/A
2005	Ferrocarril subterráneo, Barcelona, España	Colapso	N/A
2005	Ferrocarril subterráneo, Lausana, Suiza	Colapso	N/A
2005	Túnel de Autopista, Lane Cove, Sydney, Australia	Colapso	N/A
2007	Ferrocarril subterráneo, San Paulo, Brasil	Colapso	N/A
			Total: > 600

Asimismo, en 1985 se publicó un informe resumiendo las causas de sobrecostes en 41 proyectos hidráulicos financiados por el Banco Mundial, constatándose que 13 tuvieron retrasos debido a problemas geotécnicos (Tabla 2), oscilando los sobrecostes entre el 9 y el 130 %.

Tabla 2.- Retrasos en proyectos de túneles como consecuencia de problemas geotécnicos (World Bank, 1985).

Año	Proyecto	País	Sobrecoste (%)
1969	Jaguara, Complejo Hidroeléctrico	Brasil	28
1966	Quinto Generador, Complejo Hidroeléctrico	Chile	31
1968	Tercer Generador	Honduras	12
1969	Segundo Generador ENDE	Bolivia	13
1969	Hidroeléctrica Volta Grande	Brasil	130
1969	Tercer Generador, Complejo Hidroeléctrico	Costa Rica	84
1969	Finchaa Hidroeléctrica	Etiopia	23
1969	Reserva Eléctrica con Bombeo	Irlanda	79
1970	Generador Marimbondo	Brasil	53,6
1970	Hidroeléctrica de Kidatu I	Tanzania	32
1971	Hidroeléctrica de Kamburi	Kenia	9
1973	Sexto Generador, Complejo Hidroeléctrico	El Salvador	70
1973	Sigalda, Complejo Hidroeléctrico	Islandia	37

La principal causa de incertidumbre geológica es la carencia de un modelo geológico y geotécnico fiable, hecho generalmente relacionado tanto con aspectos económicos como con la premura para la ejecución a las investigaciones geológicas. A modo de ejemplo, en la Figura.- 1 se muestra la relación entre el número de sondeos y las modificaciones del proyecto en fase constructiva para obras abordadas por el Banco Mundial. Esta entidad estima que, en promedio, menos del 1 % del coste total de un proyecto se dedica a investigación geológica previa. En 1984, el National Research Council de Estados Unidos, después de revisar 87 proyectos, recomendó que al menos un 3 % del coste total de los proyectos fuera dedicado a desarrollar la investigación geotécnica (NRS, 1984). En proyectos de túneles para infraestructuras urbanas, a los problemas presupuestarios y de programa de trabajos, se añaden las numerosas limitaciones para realizar investigaciones en ámbitos edificados. Por tanto, la incertidumbre en el modelo geológico-geotécnico previo resulta generalmente superior en espacios urbanizados donde las labores de investigación ofrecen mayores dificultades.

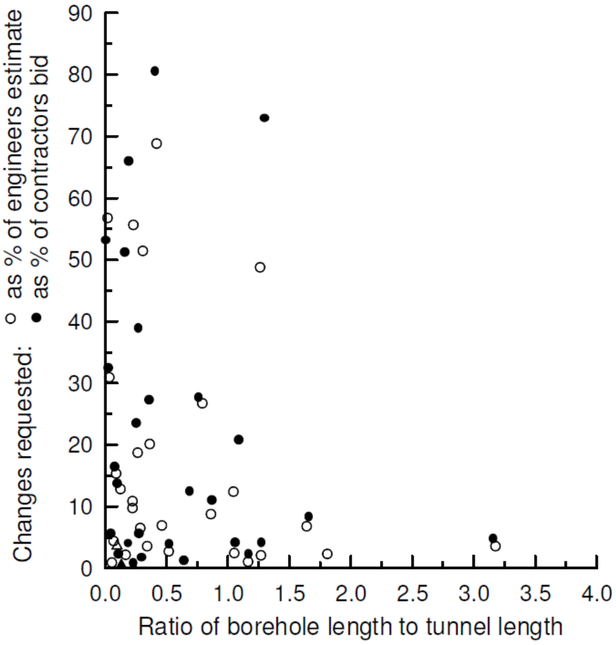

Figura.- 1. Relación entre longitud total de sondeos y modificaciones de proyecto. Datos tomados de World Bank (1985).

En Londres investigación geológica-geotécnica del subsuelo urbano comenzó a principios del siglo XIX. Así, en la actualidad se dispone de más de 8.000 descripciones de sondeos mecánicos en el área urbana de la capital (Mathers *et al.*, 2014).

Sin embargo, aún existen numerosas incertidumbres intrínsecas a su contexto geológico, a las que esta investigación trata de dar respuesta, al menos en parte. La excavación de túneles en Londres ha estado especialmente condicionada por factores geológicos y técnicos. Ya a principios del siglo XIX Whitaker (1872a, 1889a, 1889b) identificó zonas de elevada complejidad estratigráfica. Un ejemplo es el desarrollo limitado en los sectores meridional y oriental de la ciudad, relacionado con la variabilidad de los sedimentos cenozoicos de algunas formaciones, que incluyen paleocanales rellenos con arena, intercalados en arcillas y saturados en agua, a los que se asocian fenómenos de colapso del frente de las excavaciones (Copperthwaite, 1905; Follenfant *et al.*, 1969; Megaw, 1970; Skempton y Chrimes, 1994; Biggart, 2010). Debido al ambiente deposicional de estos sedimentos, los cambios de facies son frecuentes, lo que dificulta la previsión en las obras de la presencia de los niveles arenosos (Page y Skipper, 2000).

Aunque el conocimiento geológico de la Cuenca de Londres se ha visto impulsado desde el siglo XIX debido al extraordinario desarrollo urbano de numerosos proyectos de infraestructuras (Paul, 2016), es conocido que la presencia de fallas ha sido históricamente infravalorada (de Freitas, 2009; Aldiss, 2013). Este hecho explicaría algunos errores cometidos en la delimitación de los depósitos cenozoicos en el subsuelo londinense y que fueron causa de problemas geotécnicos durante la excavación de diversos túneles (Chandler *et al.*, 1998; Lenham *et al.*, 2006; Mortimore *et al.*, 2011; Newman, 2009). De igual forma, la presencia de estas fallas puede influir negativamente en las propiedades mecánicas de las los materiales afectados (Fookes y Parrish, 1969; Skempton, Schuster y Petley, 1969). Cosgrove y Ghail (2010) ya señalaron la necesidad de mejorar el conocimiento sobre la presencia y distribución de fallas en el basamento cenozoico de Londres para disminuir la incertidumbre en el desarrollo de las infraestructuras subterráneas de la capital.

1.2 Estructura de la tesis

La investigación que conforma este trabajo de tesis doctoral ha pretendido aportar novedades al conocimiento de la geología y comportamiento geotécnico de los materiales cenozoicos, a partir de: (i) la recopilación de la información obtenida a partir de las investigaciones geológicas y geotécnicas desarrolladas durante diversas excavaciones subterráneas y (ii) el análisis del comportamiento de los materiales ante los cambios tensionales debido a la excavación de los túneles.

En esta memoria se presenta en primer lugar, a modo de introducción, un resumen de la investigación bibliográfica previa, que se centró principalmente en una revisión de los problemas acaecidos históricamente durante la excavación de túneles en Londres, así como de la estructura geológica de la cuenca sedimentaria de Londres y de las principales características de los materiales geológicos que la componen. A continuación, en la siguiente sección se presentan los objetivos generales del trabajo y la junto con la metodología de investigación abordada.

En el siguiente apartado se resumen los resultados obtenidos en esta investigación, contenidos en cinco publicaciones científicas, tres de ellas en revistas del *Science Citation Index*. Por último, en el capítulo 5 resume las conclusiones de la tesis.

2 REVISIÓN BIBLIOGRÁFICA

2.1 Túneles en Londres

Desde la década de los años 20 del siglo XIX se vienen construyendo infraestructuras subterráneas en la ciudad de Londres. La Revolución Industrial produjo un aumento repentino de la población que ocasionó problemas de tráfico y la congestión de la urbe (Mair, 1998), lo que propició el desarrollo temprano de un sistema de transporte subterráneo. Los primeros túneles se realizaron con la técnica denominada "*cut and cover*" durante los años 60 del siglo XIX (Figura 2). Estas estructuras solían tener unos 10 m de profundidad y se ejecutaban mediante la hinca de un pre-sostenimiento de tablones de madera, que permitía asegurar los niveles de materiales aluviales arenosos, abordándose a continuación la excavación; se requería, además, el drenaje del agua subterránea mediante bombeo (Baker, 1885).

Figura 2.- Ilustración de la excavación de una trinchera cerca de Kings Cross para la construcción mediante el método de "cut and cover" de la Línea Metropolitana, la primera línea de metro subterránea del mundo. Tomado de Building the London Underground (2016).

El primer intento de atravesar el principal cauce fluvial del rio Támesis a través de un túnel lo llevó a cabo Sir Marc Brunel en 1825 (Smith, 2001), en lo que sería el primer túnel subacuático de la ciudad y que conectaría sus márgenes Sur y Norte (Muir Wood, 1995). Esta obra supuso, asimismo, el primer ejemplo de túnel excavado empleando un escudo (Figura 3), diseñado y patentado por el propio Brunel. Este consistía en tres cámaras con 12 marcos cada una, lo que permitía trabajar a 36 operarios simultáneamente. Cada minero avanzaba, aproximadamente, 15 cm diarios y al completarse cada avance se empujaba el escudo apoyándose en el sostenimiento ya colocado entibándose, a continuación, el frente con madera (Maidl *et al.*, 2012).

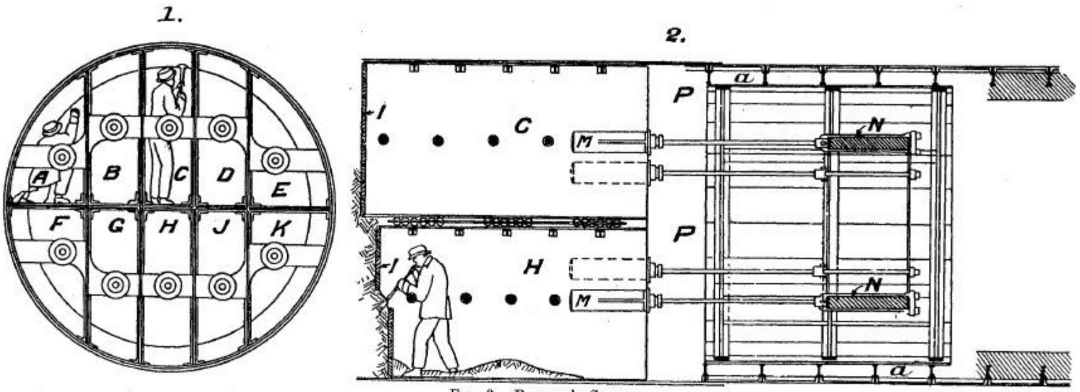

Fig. 3. Brunel's Shield.
From drawing attached to Specification of Patent No. 4204 of 1818.

Figura 3.- Esquema del escudo diseñado por Brunel para su patente de 1818. Tomado de Copperthwaite (1905).

En este caso, los sondeos de la investigación geológica preveían una excavación en una capa de arcillas duras (Formación Woolwich) de hasta 10 m de espesor, que actuaría como barrera impermeable y resistente entre el túnel y los depósitos cuaternarios del río Támesis. Sin embargo, el espesor de arcillas por encima de la clave del túnel resultó menor del esperado en algunas secciones, achacándose a esto el origen de varios colapsos y de la conexión de la excavación con el río (Smith, 2001). Skempton y Chrimes (1994) elaboraron, con los datos tomados durante esta perforación, un modelo geológico del subsuelo que encaja con los niveles actualmente reconocidos como Grupo Lambeth, depósitos sedimentarios caracterizados por sus frecuentes cambios de facies. Por tanto, este túnel representa el primer ejemplo de los problemas que esta secuencia sedimentaria puede llegar a ocasionar durante una excavación subterránea.

Los primeros túneles de la red de metro londinense, excavados en los años 80 del XIX, requerían atravesar zonas urbanas densamente pobladas. En el centro urbano, estos túneles se perforaban en la denominada Arcillas de Londres; unidad que reúne unas excelentes condiciones para la excavación, dadas su rigidez e impermeabilidad, y que garantizan un frente estable durante la excavación (Mair, 1998). Sin embargo, en las zonas Este y Sur del Támesis las condiciones geológicas son más complicadas por la presencia del citado Grupo Lambeth, lo que algunos autores sugieren como una de las razones del menor desarrollo de la red de metro en todo el sector meridional de la ciudad (Paul, 2009).

El siglo XIX supuso una época de grandes avances técnicos, ejecutándose nuevos túneles que cruzarían el trazado del Támesis, como por ejemplo el de Blackwall, con un 1,9 km de longitud y una profundidad máxima de 24,5 m, y el de Rotherhite, de 1,1 km y profundidad máxima de 23 m (Tabor, 1908; Smith, 2001). En cuanto al sostenimiento del frente con materiales arenosos y agua, el empleo de escudos de aire comprimido permitió mejorar notablemente los rendimientos obtenidos en las excavaciones (Hay y Fitzmaurice, 1897; Tabor, 1908).

En 1869, James Henry Greathead empleó un escudo circular para perforar el túnel Tower Subway, que discurría bajo el Támesis y atravesaba niveles de arenas; fue el primero en utilizar anillos de hierro fundido y la estabilización de frentes arenosos con agua, empleando la patente de Lord Cochrane basada en la aplicación de aire comprimido (Maidl *et al.*, 2012). En 1886, Greathead aplicó por primera vez en un mismo túnel todos los avances tecnológicos: escudo circular, aire comprimido para sostener la excavación y sostenimiento basado en segmentos de hierro fundido (Figura 4). Esta sistemática se convirtió, a partir de ese momento, en la empleada en la excavación de la mayoría de los túneles en Londres hasta mediados del siglo XX (Standing y Potts, 2008). El método consistía en la excavación manual de un tramo con la ayuda de un escudo, incluyendo la colocación de anillos de hierro fundido (Tabor, 1908) y la inyección de lechada en su trasdós para minimizar los asientos en superficie (Baker, 1885).

Figura 4.- Aspecto de la excavación del túnel de Rotherhite usando un escudo tipo Greathead y segmentos de hierro fundido en la estación de Bank. Tomado de Diamond y Kassel (2018).

En la Tabla 3, a modo de síntesis, se enumeran las principales patentes relativas a la excavación de túneles desarrolladas a lo largo del siglo XIX en Inglaterra.

Tabla 3.- Principales patentes relacionadas con la excavación de túneles en Gran Bretaña en el siglo XIX (Copperthwaite, 1905).

Año	Inventor	Patente
1818	M.J. Brunel	Escudo rectangular para la excavación de túneles de gran diámetro con mayor eficiencia
1830	T. Cochrane	Excavación con la ayuda de aire comprimido para sostener terrenos arenosos con agua
1874	Gratehead	Escudo circular cerrado con sostenimiento basado en inyección de *slurry* al frente
1876	T. Clapham	Erector mecánico de dovelas
1886	Greathead	Inyección de lechada de hormigón en el trasdós de los anillos para minimizar los asientos en superficie
1896	J. Price	Cabeza cortadora giratoria
1897	T. Thomson	Excavadora mecánica para la ejecución de la destroza

Durante el resto del siglo XIX y principios del XX se abordaron varios proyectos de túneles en subsuelos arenosos, algunos de ellos fallidos como consecuencia del efecto del nivel freático y las deficientes condiciones del terreno (Hay y Fitzmaurice, 1897). Por tanto, a pesar de diferentes intentos, desde la patente de Lord Cochrane el principal método para la estabilización del frente de excavación en arenas saturadas en agua se basaba en el empleo de aire comprimido (Figura 5) para la excavación de arenas o gravas bajo nivel freático (Biggart, 2010).

Sin embargo, esta técnica no está exenta de riesgos. Desde un punto de vista ingenieril, los riesgos están relacionados con episodios de pérdida de presión, con la consiguiente desestabilización del frente, y excesos de presión, que ocasionaban la aparición de chimeneas al superar el peso de la cobertera (Hay y Fitzmaurice, 1897). Además, la exposición de los trabajadores a condiciones hiperbáricas ocasionaba numerosos y graves problemas de salud, constatándose que a partir de 2,5 bares las condiciones de trabajo eran inasumibles (Glossop, 1976). Este hecho condicionó el trazado de algunos túneles al verse limitada la profundidad máxima.

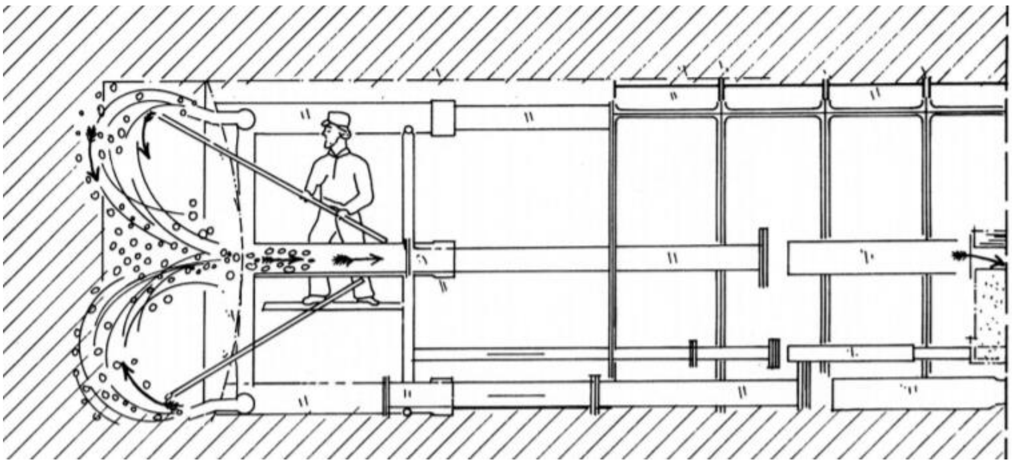

Figura 5.- Escudo de Greathead con lanza para sostener la excavación y deshacer el frente con slurry. Tomado de Maidl et al. (2012).

Las capas de arenas inter-estratificadas con las arcillas del Grupo Lambeth continuaban siendo un problema (Jones y Curry, 1927), por lo que los trazados de los túneles trataban de mantenerse siempre en las Arcillas de Londres (Paul, 2009). Diversas causas, como la presencia de otras estructuras subterráneas, errores en el modelo geológico o la existencia de derechos de propiedad, obligaban a perforar zonas con condiciones muy desfavorables, produciéndose el colapso del frente en numerosas ocasiones, caso de Green Park y Vauxhall durante la construcción de la línea de metro Victoria, en los años 60 (Follenfant *et al.*, 1969). La aplicación de aire en el frente continuó siendo el principalmente método de sostenimiento del mismo, aunque esporádicamente se emplearon otras técnicas tales como la congelación del terreno, el drenaje activo por bombeo o las inyecciones, si bien el aire comprimido siguió siendo necesario en las perforaciones (Megaw, 1970).

En los años 70 del siglo XX se comenzó a desarrollar el concepto de tuneladoras de frente cerrado, basado en la aplicación de una presión en el frente de excavación mediante lodos. El primer prototipo, usado en un túnel de prueba, se debe a John Barlett y partió de un concepto similar al de Greathead de 1874 (Maidl *et al.*, 2012). A partir de esta experiencia, en Japón y Francia se desarrollarían las tuneladoras de presión de lodos similares a las que se usan en la actualidad, que posteriormente han evolucionado a las máquina de balance de tierras, hoy en día aplicables a prácticamente cualquier tipo de condiciones geológicas (King, 2000). Esto implica que, a partir de finales de los años 80, los riesgos derivados de la excavación de túneles urbanos mediante medios mecánicos se redujeron de forma muy significativa al mecanizarse el avance del túnel (Biggart, 2010). En los numerosos proyectos concebidos en los años 80 y construidos en los años 90, como el tren ligero de los Docklands (DLR), la nueva línea Jubilee o el túnel de conexión a la alta velocidad al continente, la excavación principal se abordó con este tipo de tuneladoras (Davies, 1999; Woods, 2002; Alder *et al.*, 2010).

Sin embargo, continúa habiendo contextos en las que los medios mecánicos no son factibles por problemas de accesibilidad o longitud, como por ejemplo las galerías de conexión o los pre-túneles para tuneladoras. Estas excavaciones siguen realizándose con frente abierto y mediante métodos tradicionales, requiriendo en ocasiones algún tipo de tratamiento del terreno para garantizar la estabilidad de la excavación. En Londres, los principales incidentes en la perforación de túneles ocurridos en este período se han ocasionado, precisamente, en excavaciones a frente abierto. En 1994 se produjo el colapso de tres túneles (Figura 6) que estaban siendo excavados en las Arcillas de Londres con frente abierto y sostenimiento de hormigón proyectado, hecho que motivó la ralentización de la implantación de esta técnica en Londres (Thomas, Legge y Powell, 2004).

Figura 6.- Efectos en superficie del colapso de los túneles de Heathrow. Tomado de Jones (2000).

Otro ejemplo de problemas durante excavación con frente abierto ocurrió en 1997, durante las labores de construcción de la galería de conexión entre los túneles principales de la extensión del DLR en las que se empleaba aire a presión como método del sostenimiento del frente (Figura 7). Un exceso de presión provoco la ruptura del material de cobertera, y que ocasionó un cráter de 22 m de diámetro en la superficie.

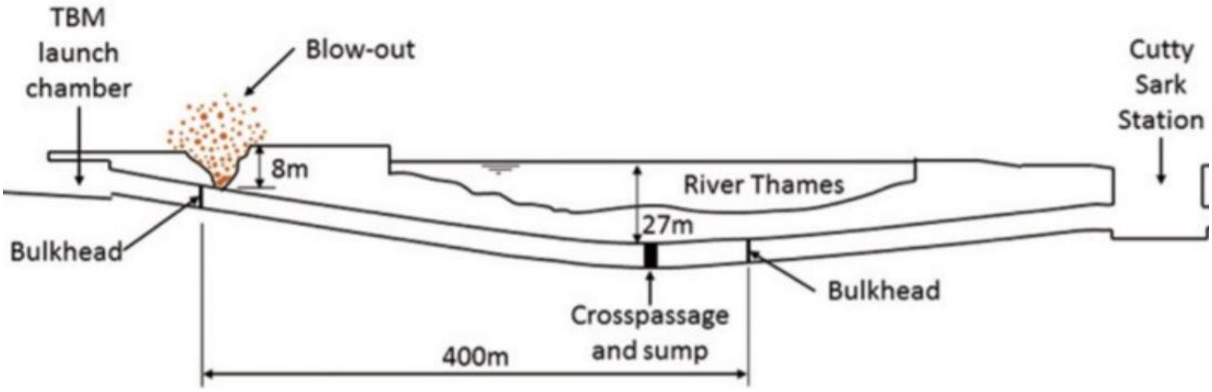

Figura 7.- Esquema de la explosión del túnel de DLR. Tomado de NCE (1998).

La excavación de los túneles para el proyecto de Thames Water Ring Main (sur de Londres), perforados en Chalk, Arenas Thanet y Grupo Lambeth, sufrió numerosos contratiempos relacionados con la presencia, no prevista, de niveles de arena saturados en agua en el Grupo Lambeth (Figura 8) y con fallos en los tratamientos del terreno aplicado, en especial fallos en el drenaje activo mediante pozos de bombeo desde la superficie.

Figura 8.- Inundación del túnel de Tooting Bec, al sur de Londres y excavado en la Formación Upnor. Fue necesario instalar un sistema de congelación del terreno para completar el túnel de lanzamiento de la tuneladora. Tomado de Newman (2009).

En síntesis, se puede argumentar que hasta entrado del siglo XXI la excavación de frentes abiertos en la Cuenca de Londres ha implicado un elevado riesgo, tanto por condiciones del terreno no previstas como por la presencia de niveles arenosos saturados en agua. El empleo de aire comprimido como principal medio de estabilizar la excavación se ha tenido que replantear como consecuencia de los mencionados accidentes y tras la aprobación en 1996 de la normativa de seguridad que regula el trabajo en condiciones hiperbáricas (HSE, 1996), en la que se fija un número de restricciones tan elevado que hacen esta técnica resulte poco eficiente.

En 2009, se comenzaron las obras de la nueva línea de metro Crossrail (actualmente denominada Línea Elizabeth), cuyo trazado sigue una trayectoria E-O, llegando y atravesando el centro urbano de Londres. Este el primer gran proyecto de infraestructura subterránea en la ciudad desde la línea Jubilee y desde la conexión con la línea de alta velocidad al continente. La construcción de los túneles de lanzamiento para las tuneladoras, así como las galerías de conexión entre los túneles principales se proyectó con frente en abierto. El tramo Este del proyecto ha sido proyectado en zonas en las que es muy probable tener que atravesar capas de arenas y zonas de fallas, por lo que constituye una buena oportunidad para probar nuevas estrategias de excavación, diferentes a las tradicionalmente utilizadas en Londres.

2.2 Marco Geológico

2.2.1 Rasgos generales

La Cuenca de Londres se sitúa en el sureste de Gran Bretaña, ocupando un área de morfología triangular de aproximadamente de 250 km de longitud y 50 km de anchura (Figura 9). Alberga una sucesión de edad Mesozoico y Cenozoico, cuyo espesor oscila entre 100 m en el borde septentrional y 1.500 m en el sector meridional, rondando los 300 m bajo el núcleo urbano de Londres (Sumbler, 1996).

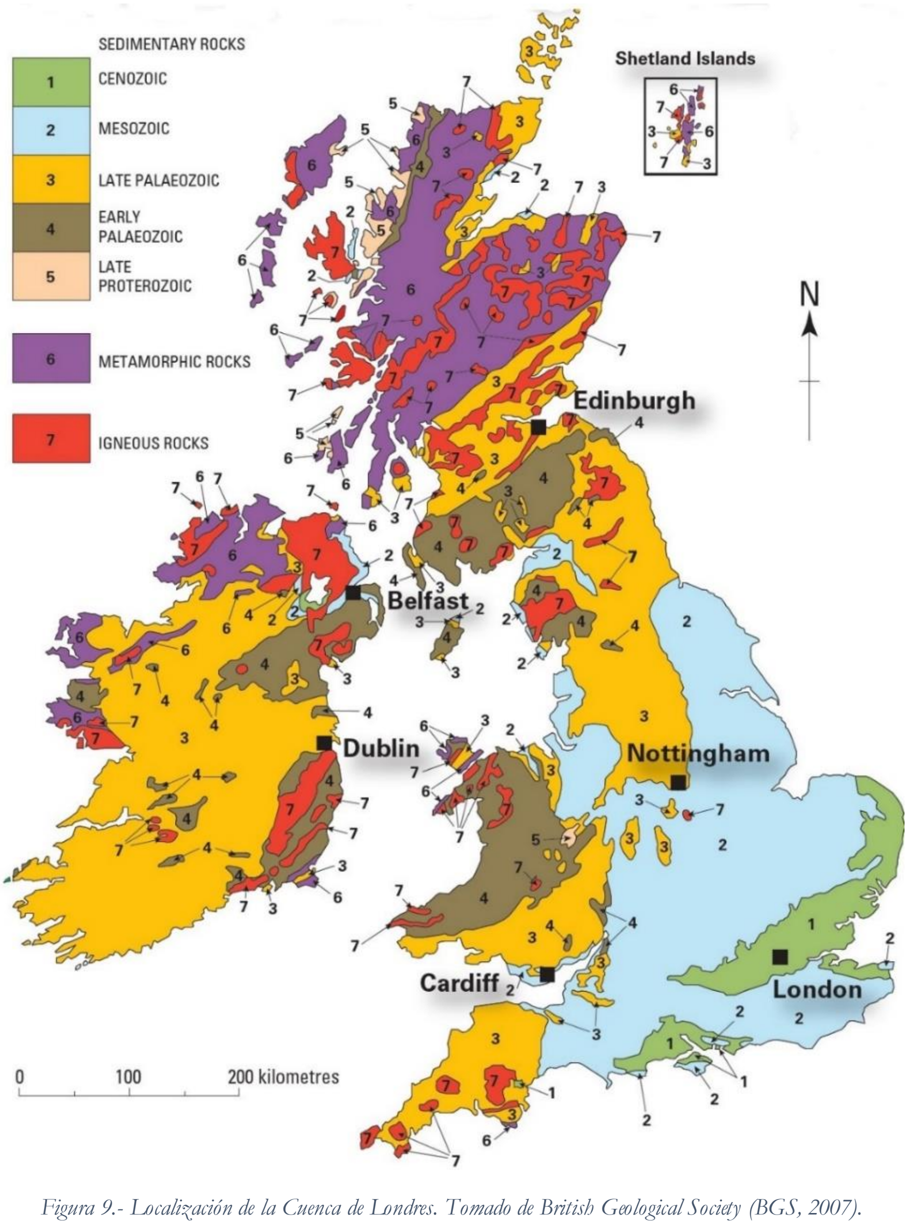

Figura 9.- Localización de la Cuenca de Londres. Tomado de British Geological Society (BGS, 2007).

Esta cuenca ha sido una de las originadas como consecuencia de la reactivación e inversión de fracturas prevariscas (Busby y Smith, 2001) durante el Paleógeno, que afectaría a materiales del Cretácico Superior, principalmente del Grupo Chalk (Ghail, Mason y Skipper, 2015).

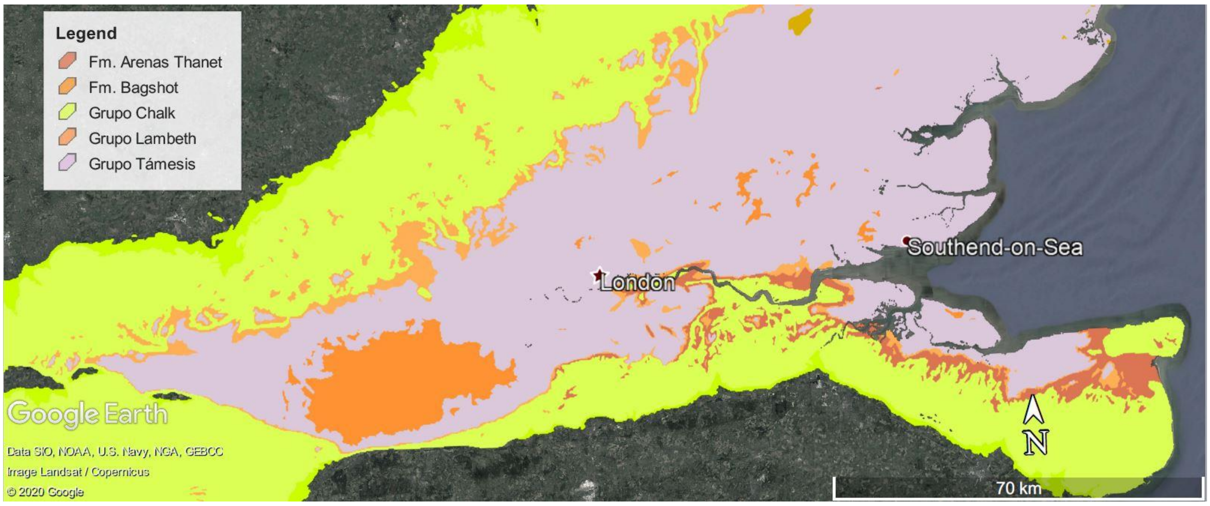

Figura 10.- Cartografía geológica de la Cuenca de Londres. Según BGS (2007).

En la Figura 10 se puede observar como el Chalk aflora al norte y al sur de Londres, constituyendo los bordes septentrional y meridional de la cuenca. Su sector central está recubierto por depósitos del Paleoceno, Eoceno y Cuaternarios, sobre todo el Grupo Támesis, e incluye las Arcillas de Londres, de edad Eocena. Otros materiales identificados en este sector son el Grupo Lambeth y la Formación Bagshot, ambos de edad eocena.

La cuenca está drenada en su mayor parte por el río Támesis y sus tributarios. Asociada al cauce principal existe una zona de depósitos aluviales cuaternarios que van desde la cota 10 m sobre el nivel del mar en el oeste de la región, hasta el nivel del mar en la zona de marismas de la desembocadura en el Este (Sumbler, 1996). Asimismo, se localizan diversas terrazas fluviales que se encuentran hasta a 30 m de cota sobre el nivel del mar (Ellison *et al.*, 2004).A modo de resumen, en la Tabla 4, se muestran las principales unidades litoestratigráficas de la cuenca de Londres durante el Mesozoico y el Cenozoico .

Tabla 4.- Resumen de las principales unidades geocronológicas y litoestratigráficas de la Cuenca de Londres.

Era	Período	Época	Unidades Locales	Litologías predominantes
Cenozoico	Cuaternario	Holoceno	Depósitos Artificiales	N/A
			Depósitos Aluviales	Arcillas y arenas finas
		Pleistoceno	Limos de Langley	Limos
			Depósitos de Terraza	Gravas
	Paleógeno	Eoceno	Formación Bagshot	Gravas y arenas
			Arcillas de Londres	Arcillas sobreconsolidadas
			Formación Harwich	Arcillas y arenas
		Paleoceno	Grupo Lambeth	Arcillas y arenas
			Fm. Arenas de Thanet	Arenas
Mesozoico	Cretácico	Cretácico Superior	Grupo Chalk	Calizas

Sobre la mayor parte de la cuenca se sitúan diversas urbes, entre las que destaca el área metropolitana de Londres, concretamente en la zona central y en ambos márgenes del Támesis, cuya población actual es de casi 9 millones de habitantes (ONS, 2019).

2.2.2 Estratigrafía

En esta sección se describen las principales características estratigráficas y petrográficas de las formaciones mesozoicas y cenozoicas que conforman la Cuenca de Londres (Figura 11).

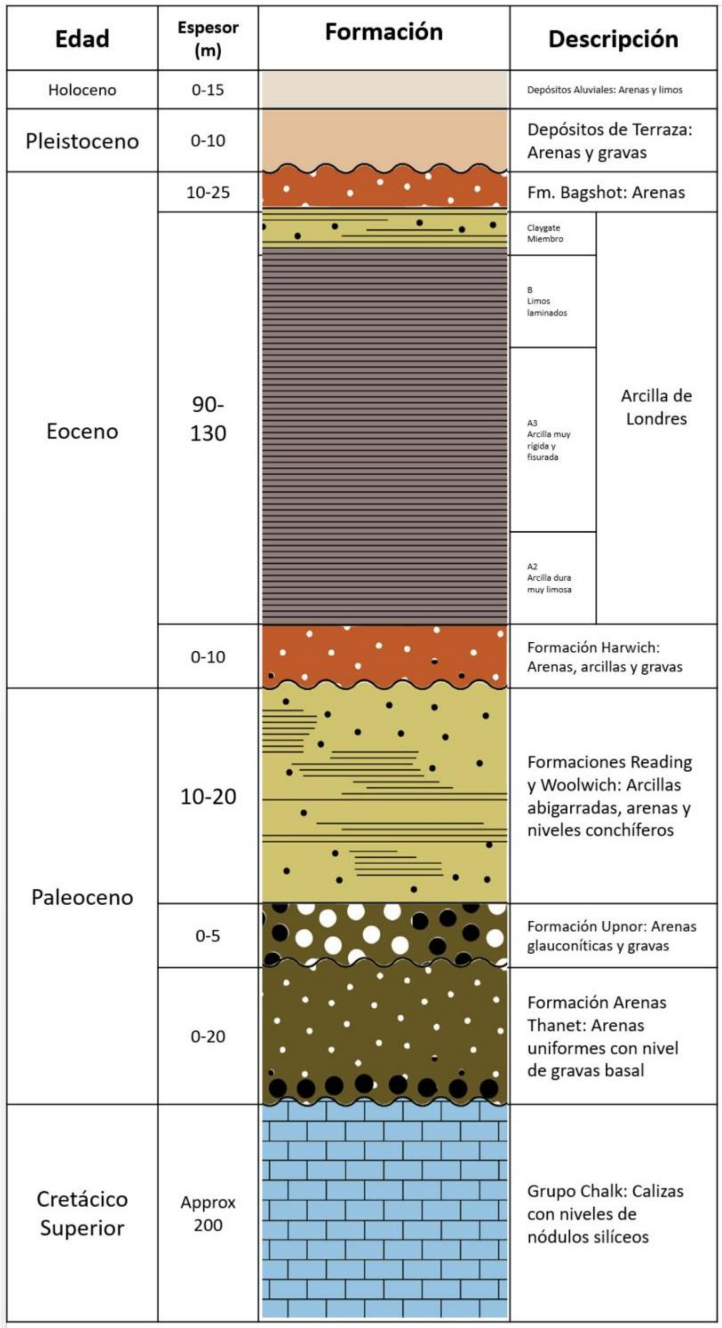

Figura 11. Resumen de las principales formaciones que integran la Cuenca de Londres. Basado en Paul (2016).

Las rocas más antiguas de la Cuenca de Londres son las pertenecientes al denominado Grupo Chalk (Figura 12). Se trata de calizas blanquecinas (> 90% carbonato cálcico), originadas por la acumulación de restos microscópicos de algas plantónicas (cocolitos), depositadas en disconformidad sobre los materiales del Cretácico Inferior (Mortimore *et al.*, 2011). En los niveles inferiores de este conjunto de materiales aparecen horizontes más ricos en arcillas (margas) y calizas con esponjas. Las capas de nódulos silíceos (*flints*), originados en relación a la disolución de esponjas silíceas, diatomeas y radiolarios (Lindgreen *et al.*, 2011), son frecuentes en la parte superior de la unidad.

El Grupo Chalk se depositó durante el Cretácico Superior (100 Ma-65 Ma), en condiciones normales de salinidad, en un mar formado durante una transgresión marina durante el Albense. Ésta supuso un ascenso del nivel del mar unos 300 m (Brenchley y Rawson, 2006), que ocupó todo el noroeste de Europa, en lo hoy es el este de Gran Bretaña, el sur del Mar del Norte y zonas del este de Francia, Alemania, Bélgica y los Países Bajos (Mortimore, 2012) y que alcanzó profundidades que oscilaban entre 100 y 600 m (Toghill, 2006).

Figura 12.- Aspecto de un afloramiento de Chalk en la costa del Sureste de Inglaterra con los niveles de nódulos silíceos (flint) paralelos a estratificación subhorizontal.

En Inglaterra han sido diferenciados los depósitos del Grupo Chalk en tres provincias, situándose la Cuenca de Londres en la Provincia Sur. Ésta se subdivide en dos subgrupos y nueve formaciones que, a su vez, se dividen en otras unidades estratigráficas (Lord *et al.*, 2002). El espesor de este Grupo en la Cuenca de Londres oscila entre 175 y 200 m aproximadamente, dependiendo esta variación de la actividad erosiva local y del desarrollo de fallas sinsedimentarias (Aldiss *et al.*, 2012).

El ascenso tectónico de Gran Bretaña y la consiguiente regresión, ocasionados al final de Cretácico Superior, propiciaron el desarrollo de pliegues y fallas en el Grupo Chalk causando un hiato deposicional de 20 Ma de duración, aproximadamente, hasta el Paleoceno (Toghill, 2006). Las capas más jóvenes del Chalk estuvieron expuestas, en consecuencia, a condiciones atmosféricas y meteorización química y mecánica, generándose una fracturación en los niveles más superficiales (Mortimore *et al.*, 2011), a menudo rellenas de arcillas (Allen *et al.*, 1997).

En la Cuenca de Londres y en el sureste de Inglaterra se habían definido, tradicionalmente, dos secuencias deposicionales mayores de los depósitos paleógenos. La primera correspondía con la Formación Thanet y el Grupo Lambeth, mientras que la segunda se corresponde con el Grupo Támesis, el cual incluye las formaciones Harwich y Arcillas de Londres del Eoceno (Stamp, 1921). Sin embargo, a partir de los años 80, la aplicación de técnicas magneto-estratigráficas (Knox, 1996b) y los datos proporcionados por numerosos sondeos (Ellison *et al.*, 1994) permitieron identificar nuevas hiatos deposicionales y definir una nueva secuencia coincidente con la sedimentación del Grupo Lambeth (Pomerol, 1989). Cada una de estas unidades muestra capas basales de arenas y gravas y abundante glauconita. Estas secuencias se han definido como de tercer orden y se encuentran intercaladas por otras microsecuencias de cuarto orden según Mitch y Van Wagoner (1991).

La Formación Arenas Thanet es la primera secuencia y, por tanto, constituye la primera formación del Paleógeno en Londres que se depositó sobre el Grupo Chalk (Ellison *et al.*, 1994). En términos generales, esta formación está integrada por tres microsecuencias granocrecientes, limitadas por niveles de arenas gruesas y/o gravas con glauconita (Knox, 1996a), y se depositó en un ambiente de aguas someras de plataforma continental (Entwisle *et al.*, 2013). La mayor parte de la formación está constituida por arenas grises (Figura 13), algo arcillosas, muy densas y bien gradadas (Aldiss, 2014). La capa basal está formada por arenas y gravas silíceas oscuras denominadas "*Bullhead Beds*" (Menkiti *et al.*, 2015). Composicionalmente se trata, casi exclusivamente, de granos angulares de cuarzo. La fracción arcillosa está compuesta por montmorillonita, posiblemente procedente de la actividad volcánica contemporánea (Knox, 1992), y es más abundante hacia la base (Linney y Withers, 1998; Aldiss, 2014). Esta formación se encuentra intensamente bioturbada debido a las distintas pausas sedimentarias, por lo que estructuras primaras -como laminaciones o *ripples*- son poco frecuentes (Entwisle *et al.*, 2013). Aunque los afloramientos son escasos y se encuentran hacia el sureste del distrito, esta formación se distribuye a lo largo de la práctica totalidad de la cuenca y área metropolitana de Londres, alcanzando espesores de hasta 30 m, que se reducen hacia el Norte hasta los 2 m (Ellison *et al.*, 1994).

Figura 13.- Aspecto de los niveles superiores de las Arenas Thanet en una excavación que previamente ha sido drenada. En la parte superior se observa el contacto con la Formación Upnor del Grupo Lambeth. Fuente: Autor.

La siguiente secuencia, actualmente denominado Grupo Lambeth (Ellison *et al.*, 1994), está integrada por las formaciones Upnor, Reading y Woolwich (Tabla 5), que afloran en los márgenes del Paleógeno hacia el sureste del distrito. El Grupo Lambeth es un conjunto de 20 a 30 m de niveles arenas, gravas, capas de calizas y arcillas, consecuencia de los cambios en los ambientes deposicionales fluviales y costeros originados por los frecuentes episodios de transgresión-regresión (Paul, 2016). Algunos autores proponen, asimismo, que los cambios de facies están también controlados por la actividad de fallas sinsedimentarias (Knox, 1996b; Royse *et al.*, 2012).

Tabla 5.- Nomenclatura de las unidades litológicas del Grupo Lambeth.

Formación	Nomenclatura anterior	Denominaciones informales	Litología
Formación Reading	Capas Reading (*Reading Beds*)	Arcillas Moteadas Superiores (*Upper Mottled Clay*)	Arcillas moteadas y capas de arena
		Arcillas Moteadas Inferiores (*Lower Mottled Clay*)	Arcillas moteadas y capas de arena
Formación Woolwich	Capas Woolwich (*Woolwich Beds*)	Arcillas Conchíferas Superiores (*Upper Shelly Clay*)	Arcillas conchíferas
		Capas Laminadas (*Laminated Beds*)	Arenas/arcillas
		Loam bandeadas (*striped loams*)	Arenas/arcillas/limos
		Arcillas Conchíferas Inferiores (*Lower Shelly beds*)	Arcillas conchíferas con arenas
Formación Upnor	Capas Inferiores (*Bottom Bed*)	Capas de cantos rodados (*Pebble bed*)	Arenas glauconíticas con gravas en la base de la formación

Prestwich (1853) fue el primero en utilizar los términos Reading y Woolwich para referirse a los materiales de este conjunto sedimentario, que posteriormente permitirían diferenciar dos de las formaciones del Grupo Lambeth. En los años 70 del pasado siglo se definió un nuevo modelo estratigráfico, que incluye 6 litofacies principales (Ellison, 1983) cuya distribución geográfica quedó delimitada (Page y Skipper, 2000). En la Figura 14 se muestra el esquema de distribución de las distintas facies y litologías de este Grupo Lambeth.

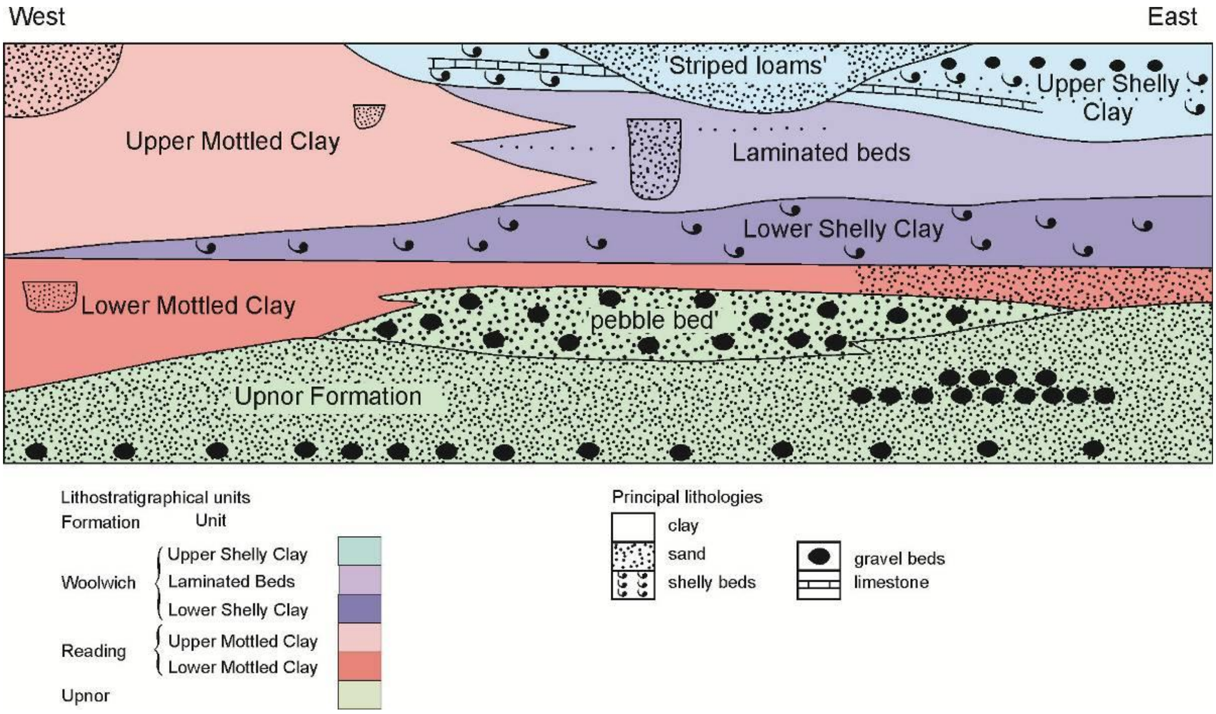

Figura 14.- Diagrama esquemático de las unidades litológicas en el Grupo Lambeth. Tomado de Entwisle et al. (2013).

La Formación Upnor, la más antigua del grupo Lambeth, es predominantemente arenosa y con abundante glauconita, lo que apunta a un ambiente sedimentario marino característico tras una transgresión y que permite explicar el contenido en carbonato (Ellison, 1983). La proporción de gravas aumenta en los niveles superiores de la unidad y, localmente, como ocurre en el centro de Londres, la arena puede estar cementada por carbonato cálcico (Skipper, 1999) e incluso aparecer niveles conglomeráticos (Figura 15).

Figura 15.- Fragmento de paraconglomerado basal de la Formación Upnor con un cemento de arenas glauconíticas y carbonato cálcico. Fuente: Autor.

El tránsito hacia las condiciones de sedimentación de la Formación Reading es gradual, y coincide con un período de bajada del nivel de mar y la formación de paleosuelos y concreciones carbonatadas ligada a procesos pedogénicos (Ellison *et al.*, 1994). Las arcillas tienen un aspecto abigarrado, con tonos azules y pardos, también probablemente debidos a procesos pedogénicos ocurridos en un ambiente de gran humedad (Buurman, 1980). Estos niveles son indicativos de una pausa en la sedimentación, que se conoce bajo la denominación "*Mid Lambeth Hiatus*" (Page y Skipper, 2000).

La Formación Reading está constituida predominantemente por arcillas rígidas y se compone de dos miembros de características similares, las Arcillas Superiores y las Arcillas Inferiores, que se distinguen por su posición en la columna estratigráfica con respecto a la Formación Woolwich (Skipper, 1999). Ambos niveles son abigarrados, con tonos marrones, azules y verdes (Figura 16), dependiendo del estado catiónico del hierro y del grado de oxidación en un clima subtropical. Esta alteración también es la causa del grado de fisuración de las arcillas, ligados a procesos de expansión-retracción (Entwisle *et al.*, 2013). Esta formación se extiende por el norte y el oeste de la cuenca, donde alcanza un espesor de 20 m, y pasa lateral y gradualmente hacia la Formación Woolwich (Ellison *et al.*, 2004). Intercalados en las arcillas aparecen, rellenando paleocanales, paquetes de arenas que llegan a alcanzar varios metros de potencia (Skipper, 1999).

Figura 16.- Aspecto de la Formación Reading en un frente de excavación. Se observa el contacto sobre una capa de arena drenada en la parte inferior. Fuente: Autor.

La Formación Woolwich es contemporánea a la formación Reading, estableciendo Hester (1965) una zona de transición hacia el centro y sur de Londres, donde las dos formaciones interdigitan (Figura 14) y a partir de la cual la Formación Woolwich aumenta su espesor hasta aproximadamente 6 m hacia el sector sureste del distrito. Esta formación, más variable que Reading, está constituida principalmente por arenas, limos y arcillas de tonos grises y negros (Ellison *et al.*, 1994). Ellison (2004) definió tres niveles dentro de ella (Tabla 5): (i) las Arcillas Conchíferas Inferiores, con facies arcillosas oscuras con abundantes conchas apoyadas disconformemente sobre las Arcillas Inferiores de la Formación Reading, lo que indicaría la acumulación de conchas un aporte de sedimentos bajo y un máximo en la extensión de la superficie de inundación; (ii) las Arcillas Conchíferas Superiores, que son muy similares (Figura 17) pero que se distinguen por su posición en el registro estratigráfico, por el mayor número de paquetes de arena rellenando paleocanales y por la mayor variedad del registro fósil, lo que viene representa una superior variedad de ambientes deposicionales; (iii) Capas Laminadas, consistentes en intercalaciones de arenas y limos laminados intercalados con arcillas y presentes en el sureste de Londres, con espesores de hasta 5 m.

Es importante señalar la presencia de una capa de caliza, denominada "Paludina", característica de la parte superior del nivel Arcillas Conchíferas Superiores y que constituye un horizonte de referencia en el contexto estratigráfico de la zona (Berry y Cooper, 1977). En la formación se han encontrado diversos niveles de arenas que rellenan paleocanales que alcanzan espesores de hasta 2 m.

Figura 17.- Aspecto de la Formación Woolwich en un frente de excavación manual. En la parte superior de observa el contacto de las Arcillas Conchíferas Superiores sobre las Capas Laminadas. Fuente: Autor.

La tercera secuencia deposicional del Paleógeno se agrupa bajo la denominación Grupo Támesis, que incluye las formaciones Harwich y Arcillas de Londres, y cuya base es una disconformidad que se puede reconocer en toda la Cuenca de Londres. También se reconoce en Hampshire, representando una transgresión sobre la que se depositaron principalmente sedimentos marinos, posiblemente ligados a un ascenso global del nivel del mar que habría estado precedido por una regresión que condujo a la erosión de parte del Grupo Lambeth (Royse *et al.*, 2012).

La Formación Harwich (Ellison *et al.*, 1994) representa el nivel basal de la secuencia después de un proceso erosivo muy intenso del Grupo Lambeth (Sumbler, 1996). La primera referencia a estos materiales fue realizada por Prestwich (1854), dividiendo estos depósitos Whitaker (1866) en Capas de Base, las Capas Oldhaven y Blackheath. Se trata principalmente de gravas silíceas, arenas finas, arcillas limosas y tobas volcánicas. Esta unidad se extiende en la mayoría de la región, alcanzando un espesor de hasta 12 m, aunque se han detectado bruscas variaciones de espesor, lo que indicaría un base irregular (Ellison *et al.*, 2004). Después de la sedimentación de la Formación Harwich, la subsidencia dio lugar a un ambiente de mar tranquilo que ocupó la mayor parte del sureste de Inglaterra y el Canal de la Mancha, observándose depósitos similares en Holanda y Francia (Burnett, 1972).

Arcillas de Londres es la formación del Paleógeno más extensa y que ha dado lugar a un relieve con elevaciones de bajo ángulo y laderas convexas (Ellison *et al.*, 2004). También ha jugado un papel importante en el desarrollo urbanístico de Londres dado su carácter homogéneo y sus excelentes características geotécnicas (Royse *et al.*, 2012; Paul, 2016). Esta unidad es predominantemente arcillosa, con un 60 % de arcillas limosas o muy limosas bioturbadas, y su potencia llega a alcanzar 150 m hacia el Este y 100 m en el centro de Londres (Sumbler, 1996), acuñándose lateralmente hacia el Oeste. Si bien es una formación bastante homogénea, históricamente se reconocieron términos más arenosos hacia su base (Whitaker, 1866). King (1981) definió cinco divisiones (A-E) basadas en variaciones bioestratigráficas y litológicas, que coinciden con episodios de transgresión y regresión marina, dando lugar a secuencias granocrecientes. En el centro de Londres solo se encuentran las divisiones A y parte de la B (Burland y Standing, 2006). Hacia el este de Londres, en la desembocadura del rio Lee, las Arcillas de Londres están ausentes, hecho que relaciona con la acción de una posible estructura tectónica (Ghail *et al.*, 2015).

Un aspecto importante de las Arcillas de Londres es la presencia de discontinuidades a distintas escalas (milimétricas hasta métricas) y disposiciones que van desde subhorizontales hasta subverticales (Fookes y Parrish, 1969). Su génesis se asocia a diferentes procesos: meteorización, relajación por desmantelamiento de coberteras y disminución de presión vertical, e incluso esfuerzos tectónicos (Chandler *et al.*, 1998). La distribución de las orientaciones de estas discontinuidades, según (Fookes y Parrish, 1969), podría explicarse en relación a esfuerzos tectónicos. El aumento de la fisuración puede ocasionar la variación del comportamiento geomecánico de la arcillas, así como de sus propiedades geotécnicas, incrementando su susceptibles a fenómenos de inestabilidad como el que se muestra en la Figura 18 (Vitone y Cotecchia, 2011).

Figura 18.- Aspecto del frente de excavación de un túnel en las Arcillas de Londres (división B). En el frente se puede observar una discontinuidad subvertical buzando hacia el observador. Fuente: Autor.

Por encima de las Acillas de Londres aparecen las formaciones superficiales cuaternarias, principalmente de origen aluvial, sedimentados en distintos niveles y ligados a épocas glaciares. Se encuentran bien preservados a ambos lados del río y conforman una capa semicontinua de gravas dispuestas sobre los depósitos de Paleógeno, mostrando espesores que llegan a alcanzar los 7 m (Ellison *et al.*, 2004). Estos materiales constituyen un excelente nivel acuífero, muy permeable, lo que les convierte en problemáticos desde un punto de vista geotécnico en el desarrollo de las obras subterráneas (Follenfant *et al.*, 1969).

Es necesario destacar, por otra parte, la presencia de una serie de depresiones que se encuentran rellenas con sedimentos recientes, posteriores al último período interglaciar, situado en el Eemiense, que se localizan normalmente próximas a la desembocadura de los afluentes del Támesis (Berry, 1979). Estas depresiones tienen morfología de embudo y pueden alcanzar profundidades de entre 15 y 25 m, aunque excepcionalmente se superan estos valores, como es el caso de la denominada depresión de Blackwall (Figura 19), que alcanza los 60 m de profundidad (Banks *et al.*, 2015). Se estima que su origen puede estar relacionado con una combinación de la presencia de estructuras tectónicas y procesos periglaciales (de Freitas, 2009). Para la construcción de obras subterráneas es fundamental detectar en fase de proyecto la presencia de este tipo de configuraciones geológicas, ya que de no hacerlo pueden suponer la no previsión de la presencia de niveles de arenas saturadas en agua (Wakeling y Jennings, 1976) que, además de problemas hidrogeológicos, pueden constituir caminos para la circulación de elementos contaminantes (Bricker et al., 2013).

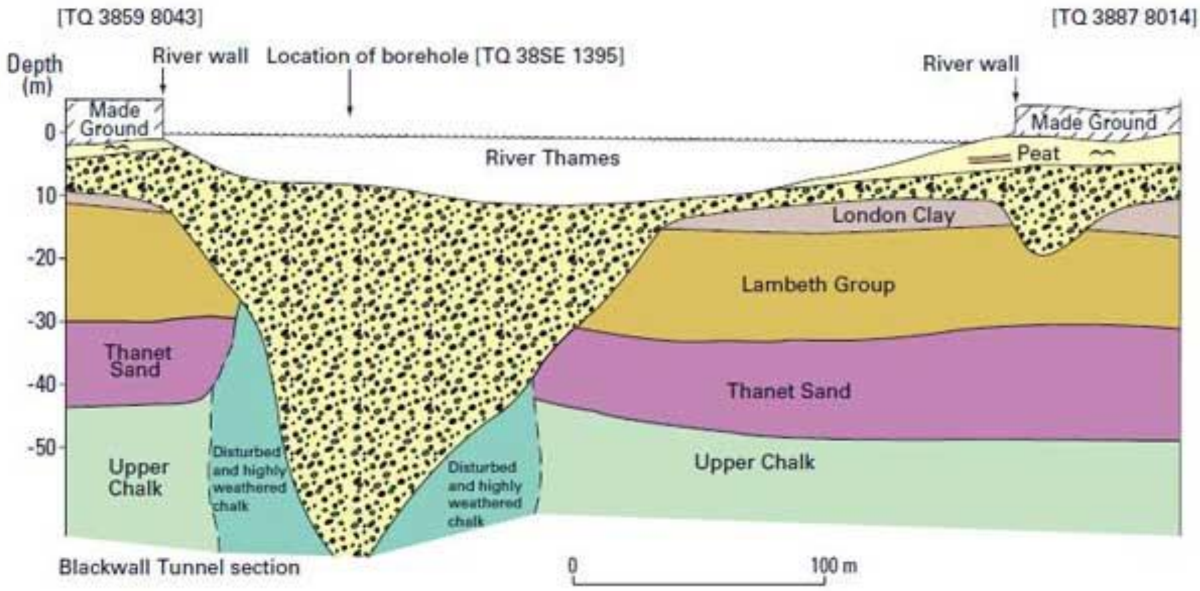

Figura 19.- Esquema de la depresión de Blackwall situada en el río Támesis. Tomado de Ellison et al. (2004).

2.2.3 Estructura

Los depósitos mesozoicos que forman la Cuenca de Londres se sitúan, como se indicó anteriormente, discordantes sobre el basamento paleozoico y precámbrico. Las rocas cratónicas más antiguas corresponden a depósitos de naturaleza clástica y edad Cámbrico. Sobre ellas, se depositaron en el Silúrico sedimentos en ambiente de aguas someras en plataforma continental, siendo de carácter fluvial y procediendo de los macizos originados durante la orogenia Caledoniana en Gales. Todos estos materiales se deformaron en el Devónico Inferior, durante la fase Acadiana de la orogenia Caledoniana (Soper *et al.*, 1987). Posteriormente, tras un período de relativa estabilidad tectónica (Rijkers *et al.*, 1993), sobre estos materiales plegados se depositaron, a lo largo del Devónico Superior, sedimentos clásticos continentales, intercalados con depósitos marinos. En el Carbonífero, se produjo un ascenso de estos niveles Devónicos, elevándose el Macizo Anglo-Brabante, que se extiende desde Gales hasta Bélgica (Pharaoh *et al.*, 1993) y del que forma parte la Plataforma de Londres (Figura 20); en concreto el núcleo urbano de Londres se localizaría hacia el sur de esta Plataforma (Brenchley y Rawson, 2006). El efecto de la orogenia Varisca es limitado en esta zona, dado que el frente de orientación Este-Oeste se sitúa hacia el sur de la misma(Kearey y Rabae, 1996). La principal consecuencia de la deformación Alpina fue la formación de cuencas extensionales, como por ejemplo la cuenca de Weald al sur de Londres (Besly y Kelling, 1988), y el desarrollo de estructuras con lineaciones SSE, SE y ESE (Lee *et al.*, 1991; Lee *et al.*, 1993; Kearey y Rabae, 1996). En el área de Londres se han inferido un número limitado de estructuras, como anticlinales y cabalgamientos que sugieren la relativa estabilidad tectónica de la Plataforma de Londres (Chadwick *et al.*, 1983).

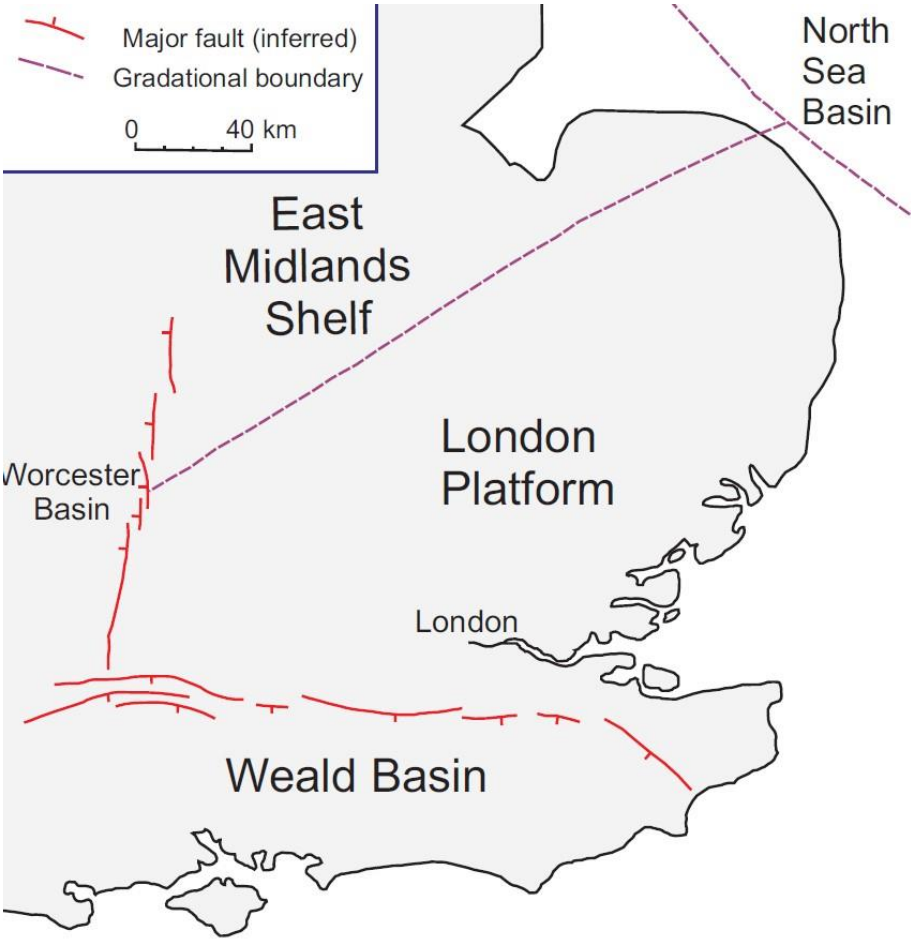

Figura 20.- Estructuras del basamento paleozoico en el Reino Unido y el Sur del Mar del Norte. En la parte inferior se observa la localización aproximada de las fallas que limitan la Cuenca de Weald con la Plataforma de Londres, y que suponen el límite del frente Varisco. Según Lee et al. (1991) y Royse et al. (2012)

A partir del Mesozoico y tras el primer evento tectónico en el distrito, la orogenia Varisca, se produjo una subsidencia generalizada ligada a la formación de fallas sinsedimentarias (Hibsch *et al.*, 1995) consecuencia del inicio de la apertura del Mar del Norte. El segundo gran evento tectónico que afectó al distrito de Londres hasta el Cretácico (Cosgrove y Ghail, 2010), pudo haber generado esfuerzos de tipo extensional oblicuos en la Cuenca de Weald al sur de la ciudad (Chadwick, 1986). Más hacia el Norte, el distrito de Londres se mantuvo estable y cubierto por mares poco profundos. Whittaker infirió la formación de fallas sindeposicionales durante el Jurásico en los depósitos de la Plataforma de Londres (Whittaker, 1985).

La extensión hacia el Norte de la corteza oceánica generada con la apertura del océano Atlántico durante el final del Jurásico y el comienzo del Cretácico, produjo el ascenso del distrito de Londres y la erosión de los sedimentos Jurásicos, haciendo aflorar el basamento Paleozoico (Chadwick, 1993). Esta extensión provocó, asimismo, un inversión tectónica de las estructuras Este-Oeste generadas durante la orogenia Varisca (Royse *et al.*, 2012) y propició que la Plataforma de Londres quedara finalmente sumergida durante la transgresión del Albense (Brenchley y Rawson, 2006).

A partir del Cretácico Superior, la región estuvo afectada por esfuerzos compresivos alpinos (Hibsch *et al.*, 1995), teniendo lugar un descenso global del nivel de mar (Skipper, 1999) que produjo un periodo de erosión del basamento Paleozoico y Mesozoico que se prolongaría entre 10 y 15 Ma (Entwisle *et al.*, 2013). Tras este hiato, comenzaron a depositarse los primeros sedimentos del Paleoceno en un ambiente de transición (estuario, costero, *lagoon*), generalmente en cuencas más pequeñas ocasionadas por la compartimentación tectónica (Chadwick, 1986). En el Oligoceno se ocasionó el mayor episodio compresivo en el área de Londres, desarrollándose la estructura de tipo sinclinorio que dio lugar a la Cuenca de Londres (Sumbler, 1996). Al mismo tiempo, la inversión de fallas profundas al Sur favoreció el desarrollo de pliegues , destacando el Anticlinal de Greenwich (Aldiss, 2013). A pesar de estos episodios compresivos, el número de fallas de origen alpino identificadas en la Cuenca de Londres es muy reducido (Figura 21), siendo más frecuentes hacia el Sur, en la Cuenca de Weald.

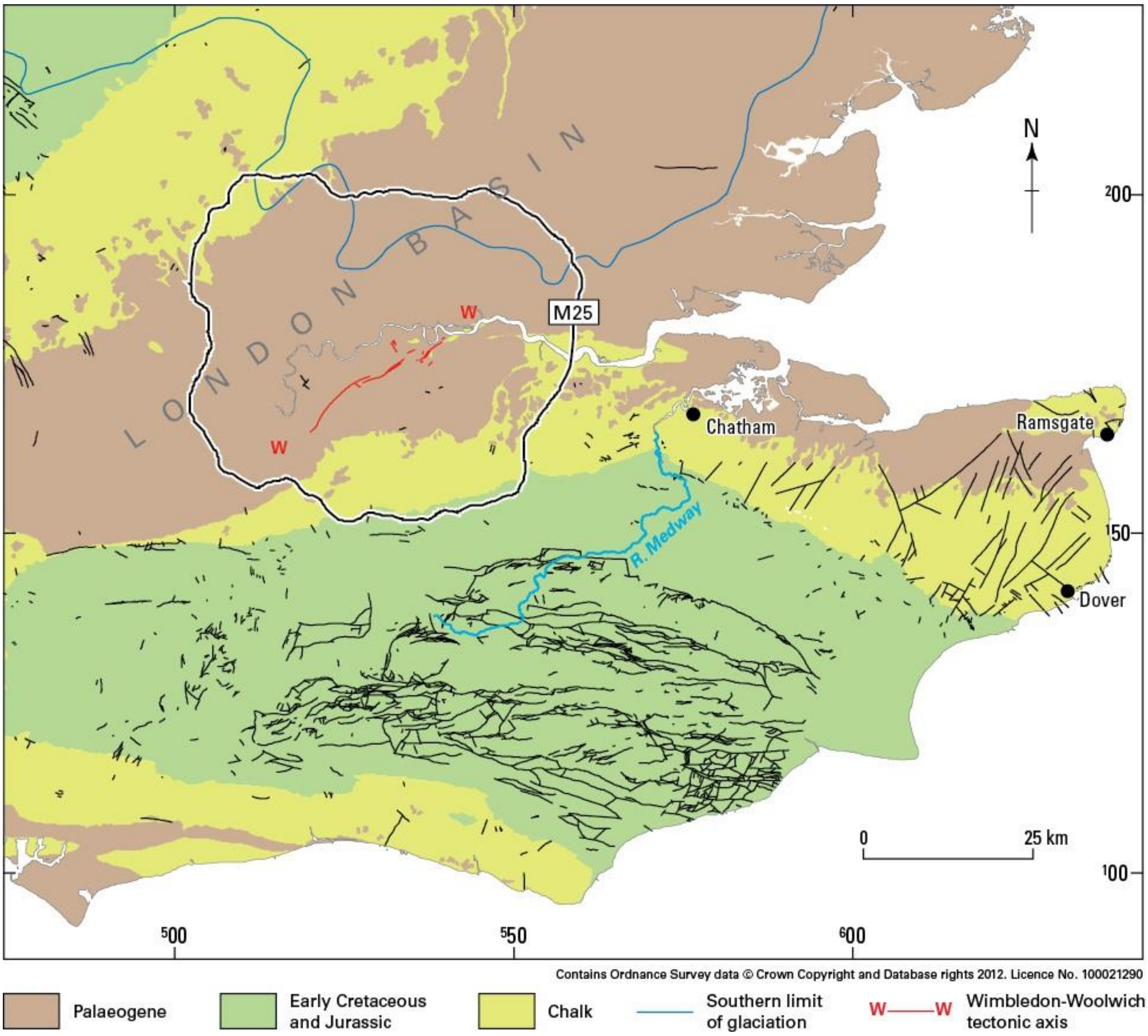

Figura 21.- Principales estructuras tectónicas identificadas en la cartografía geológica de British Geological Survey. Mapa tomado del BGS (2012).

El primer modelo estructural de la Cuenca de Londres fue propuesto por Wooldridge (1923, 1926) mostraba una dirección preferente de las estructuras tectónicas SO-NE (Figura 22). Hasta los años 1990 se mantuvo vigente el modelo de sinclinal para la cuenca propuesto por Sherlock (1947) en la cartografía regional, mostrado en la Figura 23. Los estudios geofísicos realizados para la búsqueda de hidrocarburos en los años 60 del pasado siglo identificaron diversas anomalías que se interpretaron como fallas procedentes de la reactivación de antiguas fallas del basamento paleozoico (Chadwick, 1993), que ya fueron incluidas en la cuarta edición de la cartografía regional de Londres elaborada por Sumbler (1996) que muestra la Figura 24.

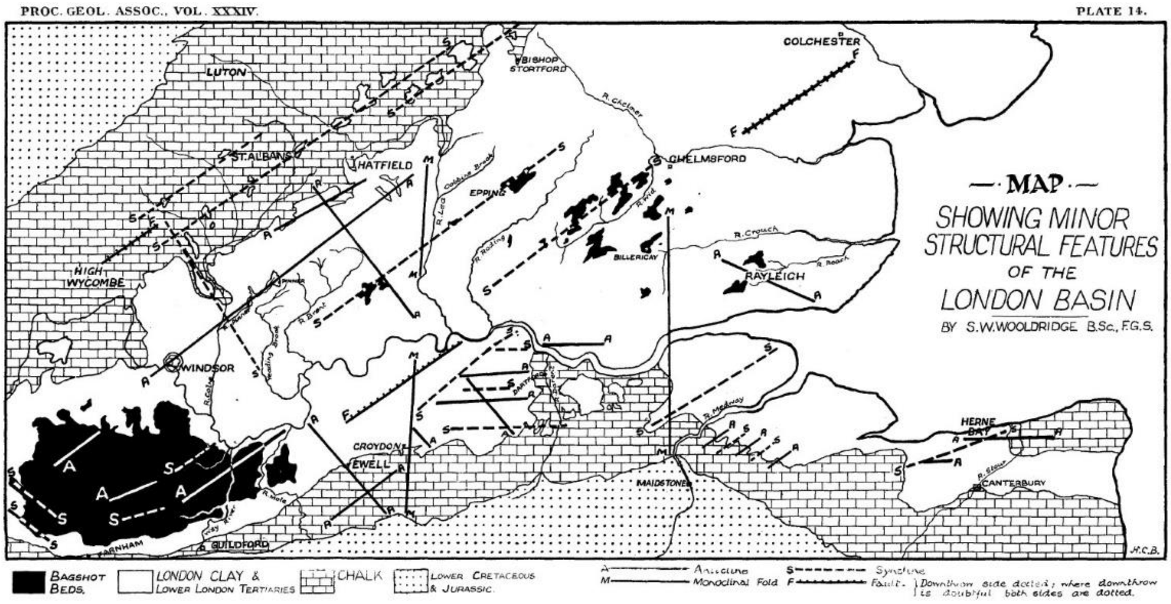

Figura 22.- Estructuras tectónicas de la Cuenca de Londres. Tomado de Wooldridge (1926).

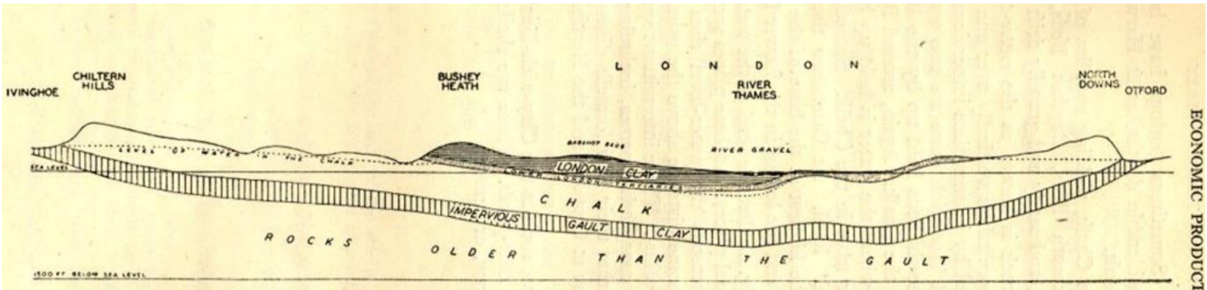

FIG. 24.—*Diagrammatic section across the London Basin to show the Water-table in the Chalk.*
The vertical scale is greatly exaggerated.

Figura 23.- Sección geológica de la Cuenca de Londres de acuerdo a Sherlock (1947).

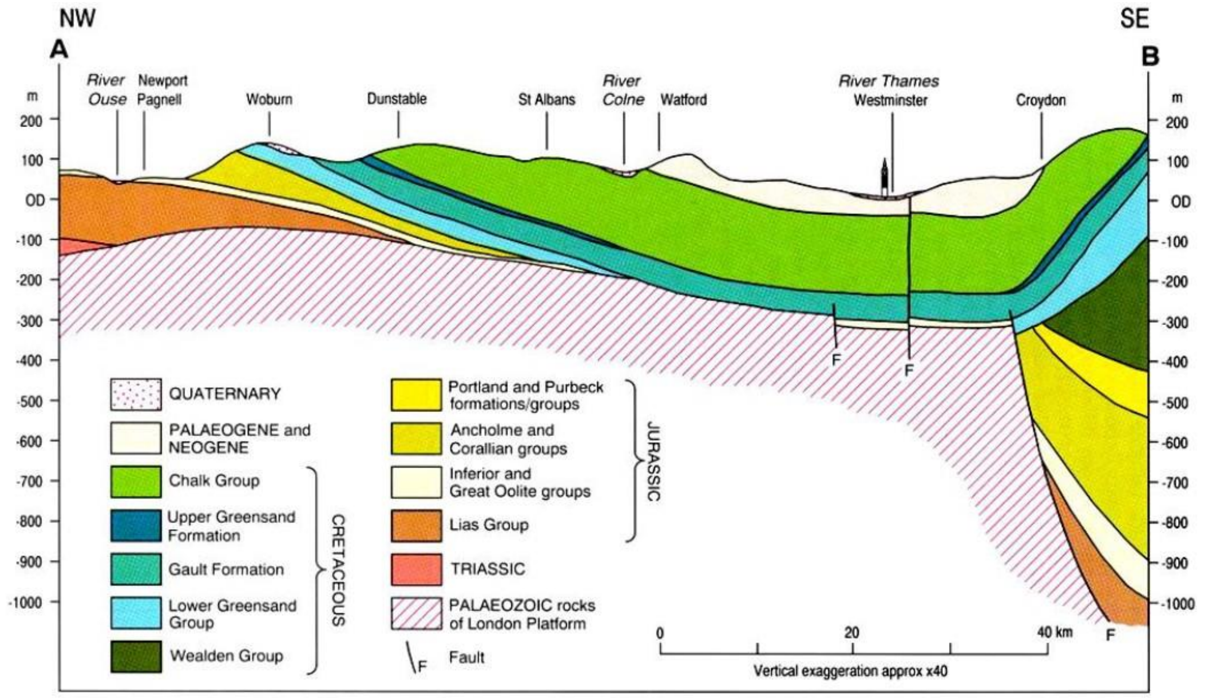

Figura 24.- Sección geológica de la Cuenca de Londres según Sumbler (1996).

En 2009, tras diversos estudios hidrogeológicos, se propuso la compartimentación de la cuenca motivada por la presencia de diversas fallas (de Freitas, 2009), cuya expresión en superficie es la actual red hidrográfica, aunque esté parcialmente oculta por la urbanización de Londres (Figura 25).

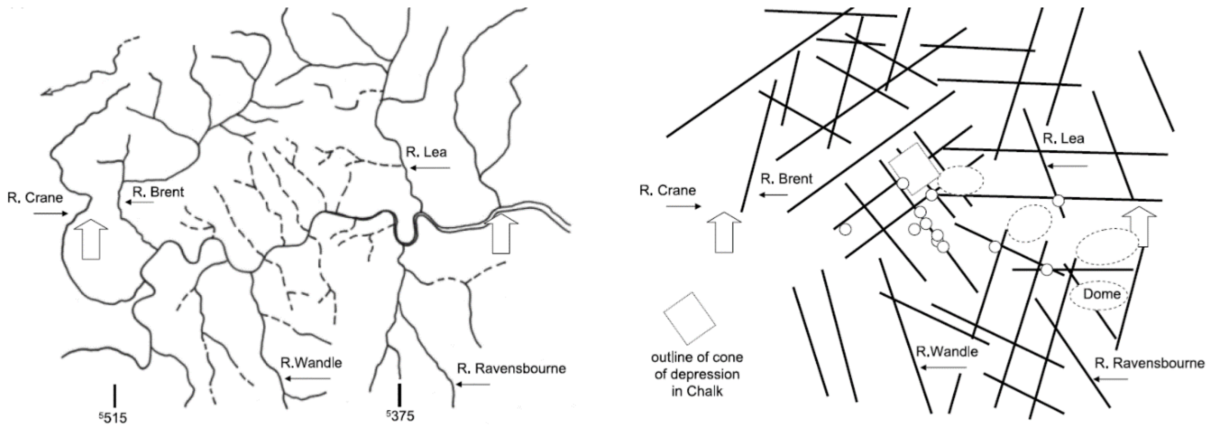

Figura 25.- Mapa de la red de drenaje de la Cuenca de Londres (izquierda) y la propuesta de un sistema de fallas que crea compartimentos. Los círculos, representan posibles fallas según los registros de Metro de Londres y estructuras de disolución. Tomado de de Freitas (2009).

Cosgrove y Ghail (2010) propusieron la existencia de estructuras de tipo extensional como estructuras en flor, negativas y positivas, para explicar algunas de las estructuras geomorfologías de Londres. Aldiss (2013), asimismo, sugirió que los mapas del British Geological Survey habían históricamente infrarrepresentado las fallas en Londres.

2.2.4 Hidrogeología

El Grupo Chalk, junto con las Arenas Thanet y la Formación Upnor, conforma el principal acuífero de la Cuenca de Londres. Se trata de un acuífero confinado por las Arcillas de Londres o por los términos arcillosos del Grupo Lambeth que actúan como acuitardos, de modo que el acuífero se recarga por medio de las precipitaciones en los afloramientos al sur y al norte de la cuenca (Environment Agency, 2018) como muestra la Figura 26.

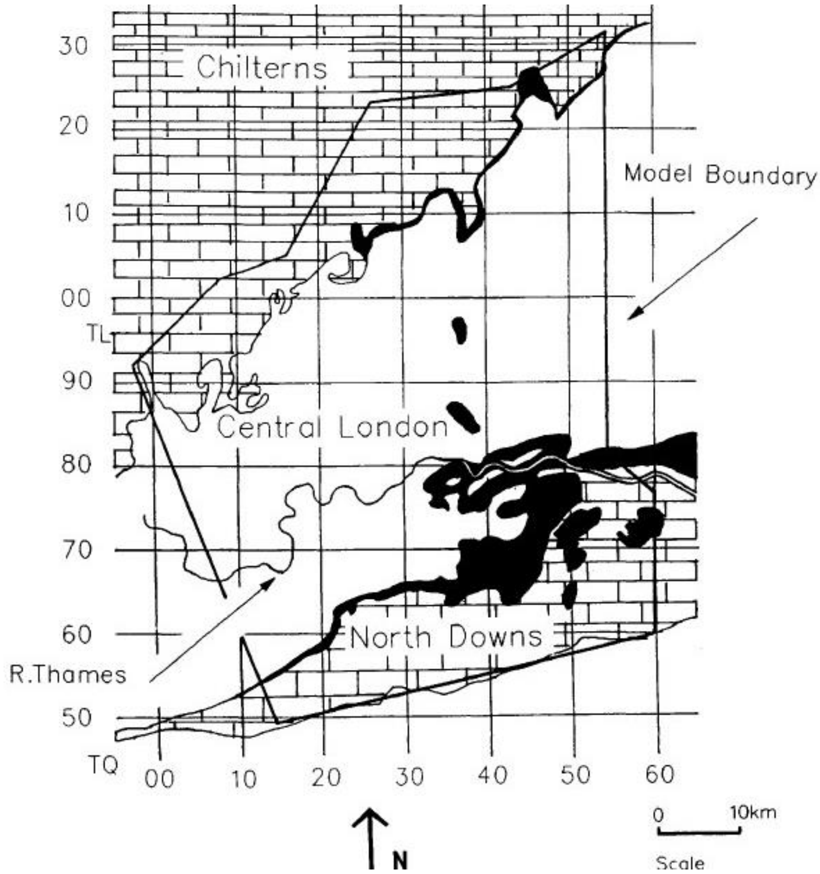

Figura 26.- Zonas de recarga del acuífero de Londres. En cuadrícula el Grupo Chalk y en negro las arenas basales del Paleógeno. Tomado deLucas y Robinson (1995).

Algunas fallas condicionan los niveles freáticos ya que actúan como barreras impermeables, como se puede observar en la Figura 27.

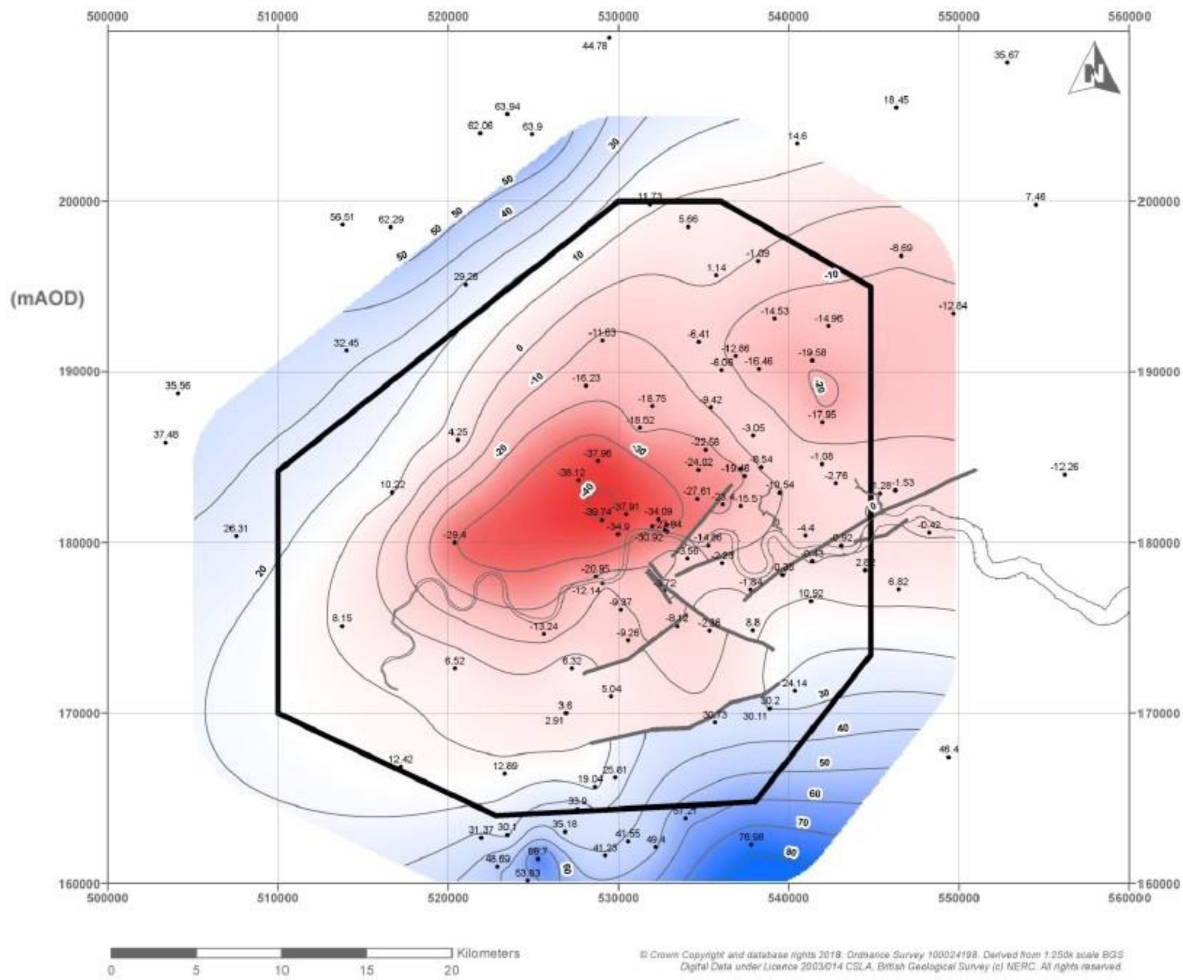

Figura 27.- Niveles piezométrico del acuífero principal en Londres. Tomado de Environment Agency (2018).

El Grupo Chalk constituye la principal unidad acuífera, si bien ofrece poca capacidad de almacenamiento. Muestra una alta transmisividad, con doble porosidad, que permite el flujo de agua en los poros y fisuras (Ellison *et al.*, 1994). Los valores de permeabilidad muestran una elevada variabilidad y parecen depender principalmente del mayor o menor grado de fisuración (Price et al., 1982; Bevan et al., 2010). La transmisividad media es de 160 m^2/día (10^{-4} m^2/s), con valores oscilando entre 1 y 4.300 m^2/día (10^{-2} a 10^{-5} m^2/s) según Allen *et al.* (1997). La permeabilidad parece aumentar hacia los niveles superiores del Chalk, sin embargo, parece estar más relacionada con el espesor de la cobertera (Allen *et al.*, 1997). En la Figura 28 se muestra cómo la transmisividad disminuye con la profundidad necesaria para alcanzar el Chalk.

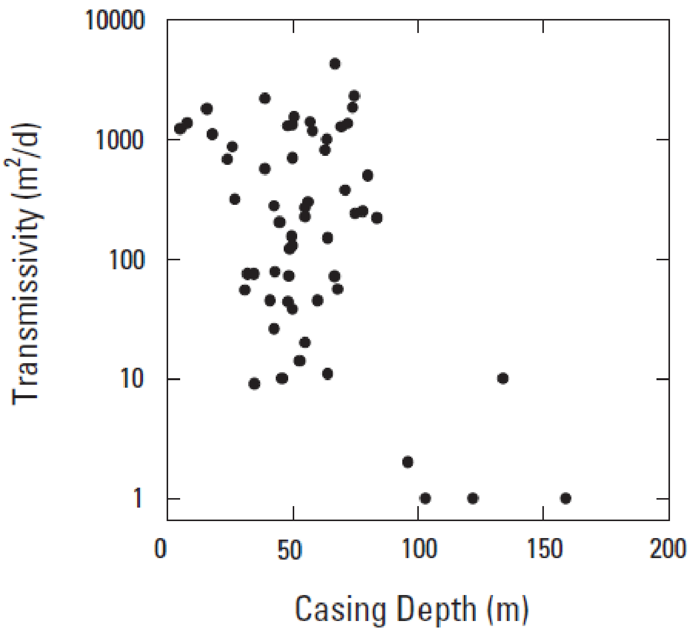

Figura 28.- Relación con la transmisividad (eje vertical) y la profundidad de entubación (eje horizontal) según Allen et al. (1997).

Jones *et al.* (2000) estimaron unos valores promedio de permeabilidad para las Arenas Thanet, a partir de ensayos de laboratorio, de 11,3 m/día (10^{-4} m/s) y un rango entre 0,01 y 167 m/d (10^{-7} y 10^{3} m/s), lo que apunta a la presencia de niveles arcillosos. La presencia de estos niveles tiene una notable trascendencia en el desarrollo de obras subterráneas, dado que para excavar en las Arenas Thanet se debe recurrir al bombeo del Chalk, de mayor permeabilidad, para conseguir el drenaje indirecto de las arenas mediante la continuidad hidráulica. Estas arcillas, por el contrario, pueden actuar como una barrera casi impermeable que limita el flujo de agua desde el Thanet al Chalk (Linney y Withers, 1998).

El nivel piezométrico natural de la cuenca puede establecerse en unos 7,5 m sobre el nivel del mar, ligeramente superior a la cota topográfica de Londres (Lucas y Robinson, 1995); sin embargo, en el siglo XVIII se comenzó a explotar el acuífero principal mediante pozos profundos desde las arenas basales (Jones *et al.*, 2000). A partir del siglo XIX el nivel de extracción creció hasta los años 60, pasando de 9×10^{6} m^{3}/año a 83×10^{6} m^{3}/año (Hurst y Wilkinson, 1986). En ese momento la posición del freático había disminuido hasta los 88 m de profundidad bajo el nivel del mar, lo que motivó un asentamiento general del centro de Londres, ya descrito por Longfield (1932), comparando medidas de cuatro puntos geodésicos. Otro efecto fue la modificación de perfil piezométrico de la cuenca en el centro de Londres, con la formación de dos tramos hidrostáticos, uno comenzando en superficie y el siguiente en la zona superior del acuífero principal (Figura 29).

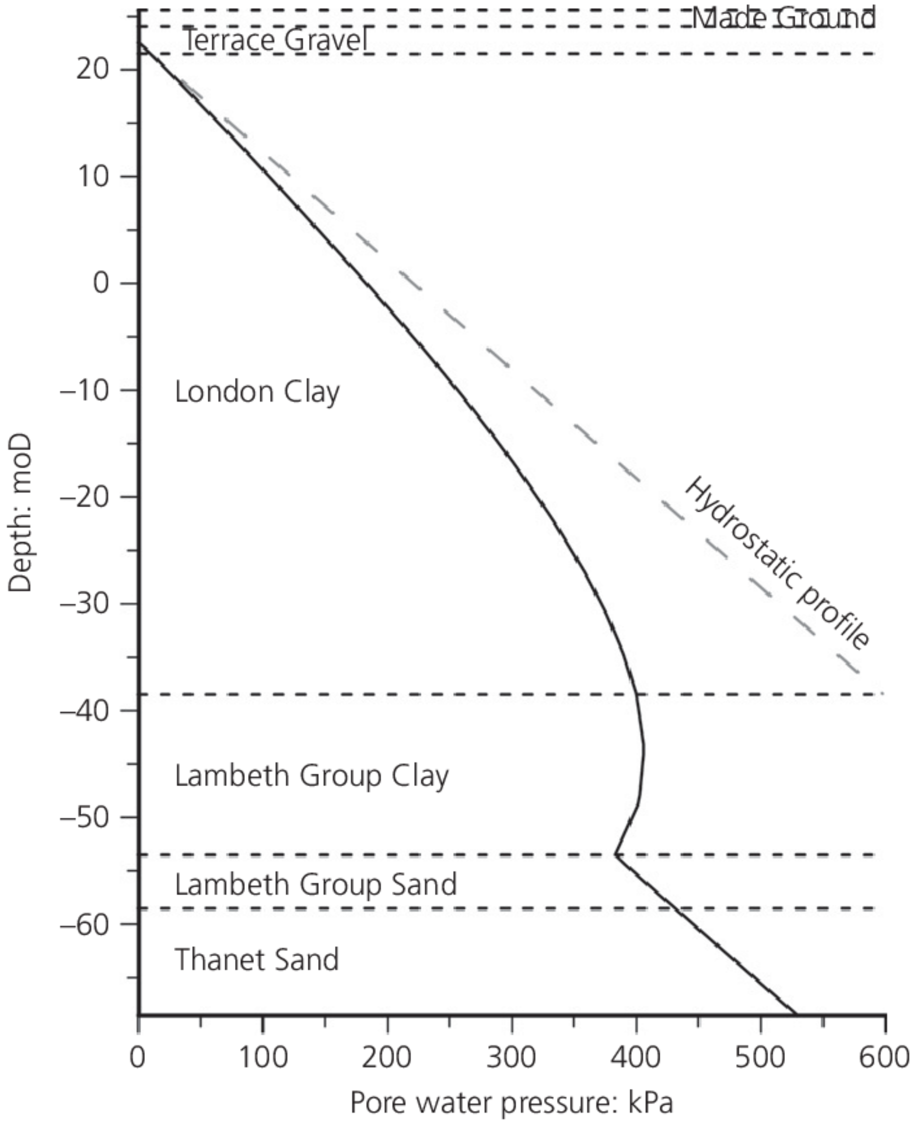

Figura 29.- Perfil piezométrico en Londres según Gawecka et al. (2016).

A partir de los años 70 la actividad industrial en Londres disminuyó significativamente y, con ello, la explotación del acuífero, lo que propició un ascenso generalizado del nivel freático. Como se observa en la Figura 22, a finales de los años 60 ya había comenzado a subir de cota del nivel freático. En 1995 se había producido una variación del nivel de 42 m (Lucas y Robinson, 1995), llegando incluso a comenzar la saturación de la Formación Arcillas de Londres, donde se encuentran la mayor parte de las estructuras subterráneas de la ciudad (Paul, 2009). Este ascenso podría provocar la deformación de túneles o reducción de la capacidad portante de cimentaciones profundas (Simpson *et al.*, 1989).

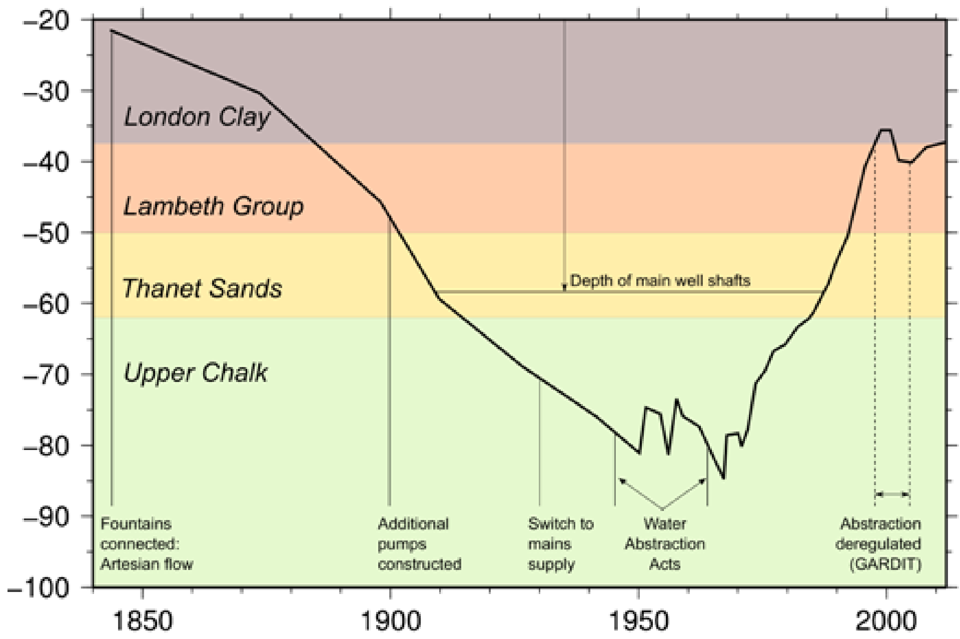

Figura 30.- Nivel piezométrico del acuífero principal en el pozo localizado en Waterloo, en el centro de Londres. Tomado de Jones et al. (2012). Se observa un brusco aumento del nivel más de 40 m en 25 años, desde finales de 1970 hasta mediados de los 90.

Las Arcillas de Londres, la Formación Harwich y los niveles arcillosos del Grupo Lambeth, situados por encima del acuífero principal, se comportan principalmente como acuitardos, mientras que las formaciones superficiales del Cuaternario y los niveles granulares del Grupo Lambeth lo hacen como acuíferos secundarios (Jones *et al.*, 2000).

3 OBJETIVOS

El objetivo principal de este trabajo de Tesis Doctoral consiste en avanzar en el conocimiento sobre las dos principales causas de problemática geológico-geotécnica vinculadas a la excavación de túneles urbanos de Londres: (i) la existencia de fallas, algunas de ellas no cartografiadas; y (ii) la presencia de niveles de arenas en el grupo Lambeth. Para ello, esta investigación se centrará en tres aspectos esenciales:

Reducir y gestionar de forma efectiva de la incertidumbre geológico-geotécnica en fase de proyecto y obra, a través de una recopilación e interpretación adecuada de toda la información previa disponible.

Analizar la problemática causada dentro del ámbito urbano por la aparición de niveles de arenas saturadas en agua en el Grupo Lambeth y valorar sus posibles tratamientos.

Anticipar la presencia de fallas a través de los modelos estructurales y establecer las implicaciones geotécnicas que plantea su presencia en la perforación de túneles urbanos.

Como principales hipótesis de este trabajo de Tesis Doctoral se plantearon las siguientes:

(i) La presencia de fallas dentro de la Cuenca de Londres responde al modelo propuesto por de Freitas (2009) y Cosgrove y Ghail (2010), y afecta al comportamiento geotécnico de las Arcillas de Londres.

(ii) La excavación de niveles arenosos con frente abierto en zona urbana es viable mediante la ejecución de bombeos activos, los cuales pueden ser diseñados de una forma empírica en fase de proyecto y a partir de la información obtenida en fase de excavación.

(iii) La excavación de túneles con métodos convencionales en una cuenca sedimentaria de alta variabilidad geológica es posible a partir de un modelo geológico que permita anticipar el comportamiento geotécnico, siempre y cuando se adopten las medidas necesarias para mitigar los riesgos intrínsecos a una excavación urbana.

Metodológicamente, la investigación se dividió en dos etapas. En primer lugar, se abordó una revisión detallada de la literatura científica y técnica publicada en relación con los principales propósitos de la investigación, es decir:

(i) El marco geológico y la evolución del conocimiento geológico y geotécnico de la Cuenca de Londres.

(ii) La evolución de las técnicas empleadas para excavación de túneles en la Cuenca de Londres y la revisión de los casos de estudio de obras subterráneas más importantes.

(iii) Los problemas geotécnicos e hidrogeológicos relacionados con la excavación de túneles urbanos que mayor incidencia han tenido en el desarrollo de las obras subterráneas, y cómo se ha gestionado la incertidumbre geológica en estas obras de ingeniería civil.

En una segunda fase se abordó la investigación concreta de tres casos de estudio, enmarcados en el Proyecto de Crossrail actualmente denominado Línea Elizabeth, lo que ha supuesto el cuerpo principal de trabajo en esta Tesis Doctoral. Se trata de una nueva línea de metro que atraviesa con dirección Este-Oeste la ciudad de Londres y cuyo sector central cuenta con un trazado subterráneo consistente en dos túneles paralelos excavados con tuneladoras de balance de presión de tierras. Los casos de estudio considerados corresponden a tajos que requirieron el uso de técnicas tradicionales de frente abierto (Figura 31), y fueron seleccionados por su relevancia para alcanzar los objetivos propuestos en este trabajo:

1. El primer caso es relativo a los túneles construidos en la península de Limmo a modo de pre-túnel para el lanzamiento de la tuneladora. El principal propósito ha sido determinar la presencia de fallas durante la excavación, afectando a la Formación Arcillas de Londres, proponer un modelo tectónico y analizar el impacto de estas posibles fallas en el comportamiento geotécnico de las arcillas en excavación.

2. El segundo es relativo a la excavación de las cavernas de Stepney Green, las cuales se excavarían parcialmente en el Grupo Lambeth. Esto permitió obtener información sobre la presencia y las características de las capas de arena, así como evaluar una posible estrategia para gestionar el riesgo de arenas saturadas.

3. Finalmente, el tercer caso consistió en la excavación de la galería de conexión CP6, de 14 m de longitud, ejecutada con una limitada investigación geotécnica previa. Esto permitió investigar cómo se puede gestionar la incertidumbre geológica y determinar algunas características geológicas de paleocanal con relleno de arenas.

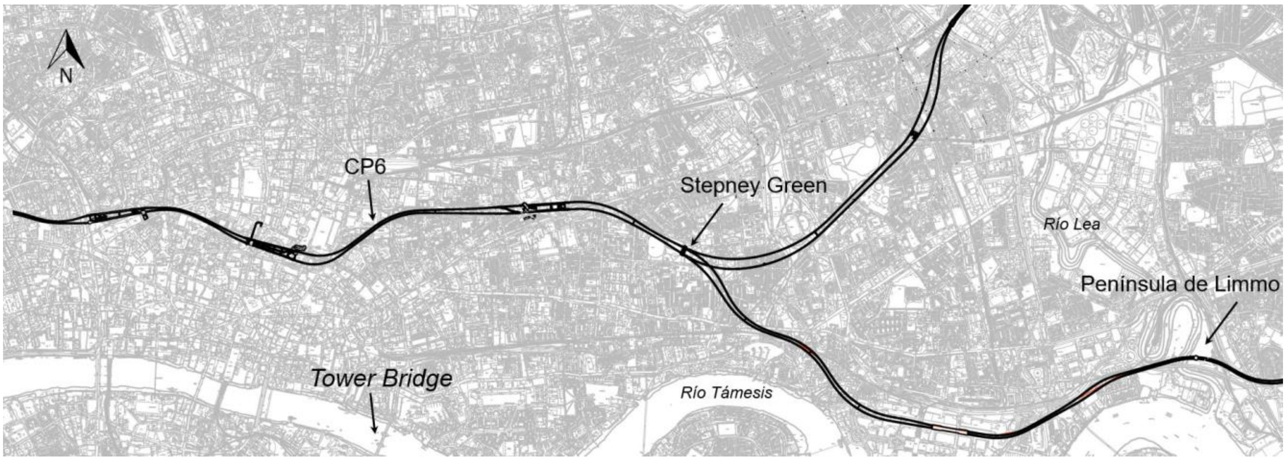

Figura 31.- Ruta de la línea Crossrail (o Elizabeth Line) y localización de los tres casos de estudio.

Los resultados de toda esta investigación se recogieron en cinco artículos científicos, tres de los cuales han sido publicados en revistas incluidas en el *Science Citation Index*.

4 PUBLICACIONES

4.1 Publicaciones SCI

4.1.1 Ingeniería Geológica y túneles en la península de Limmo, Este de Londres

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Engineering geology and tunnelling in the Limmo Peninsula, East London

Emilio Linde-Arias[1]*, David Harris[2] & Richard Ghail[3]

[1] OTB Engineering, London, 46 Loman Street, London SE1 0EH, UK
[2] Mott MacDonald Group Ltd, 10 Fleet Place, London EC4M 7RB, UK
[3] Civil and Environmental Engineering, Imperial College London, London SW7 2AZ, UK
E.L., 0000-0001-6699-8254; R.G., 0000-0002-4918-0685
* Correspondence: elinde@otbeng.com

Abstract: The Limmo Peninsula site has some of the most complex geology of London's Crossrail project and was the launching point for four tunnel boring machines (TBMs) to allow construction of Crossrail's eastern running tunnels. It is located in East London, *c.* 2 km east of the Canary Wharf business district, adjacent to the River Lea. It consists of a ventilation shaft, an auxiliary shaft, two sprayed concrete lining (SCL) tunnels interconnecting the shafts and four SCL adits for assisting in the launching of the TBMs. As part of the design requirements, some geological formations had to be depressurized from surface wells. The site is geologically complex: it is in the vicinity of a drift-filled hollow and it is located within the area of influence of several tectonic features. A geological ground model developed from important new information obtained during the design stage ground investigations and from direct observations conducted during construction stages reveals an inverted transtensional flower structure (i.e. it is now a transpressional restraining bend). Of special interest are the unusually low values of undrained shear strength of the London Clay associated with the tectonic setting.

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Crossrail links east and west of London with a new railway (Black *et al.* 2015), which crosses the Limmo Peninsula, East London. As anticipated, a range of features consistent with faulting were encountered during construction at the Limmo Shafts site on the east bank of the River Lea, north of the Lower Lea Crossing Bridge (Fig. 1). The site is in the floodplains of the River Thames and the River Lea and therefore the topography is flat and low-lying; the ground elevation is *c.* +7 m above Ordnance Datum (AOD) (+107 m above tunnel datum (ATD)).

The principal structures at the site are the main shaft, the auxiliary shaft and the sprayed concrete lining (SCL) tunnels. The main shaft is 44.3 m deep and 30 m in diameter. The stability of the excavation was provided by a diaphragm wall with a toe level 55 m below ground level. The auxiliary shaft is 39 m deep with an internal diameter of 27 m. The initial 17 m was constructed using sheet piles, with sprayed concrete used for the remainder of the shaft. The SCL tunnels provided twin connections for the shafts and also for back and launch adits. They have a teardrop shape that is 8.65 m vertically and 7.98 m horizontally.

The presence of faults and drift-filled hollows (Fig. 1) at the confluence of the River Lea and the River Thames is well documented (Berry 1979; Hutchinson 1980; Banks *et al.* 2015) and led to challenging design and construction conditions. Faults cause a range of geological and hydrogeological hazards, by creating either low-permeability boundaries or high-permeability pathways that affect predicted groundwater behaviour, and by fracturing and damaging the ground, leading to face instability, large volume losses and settlements when encountered during tunnelling and bulk excavations.

Geological context

The site is located in a part of the London Basin long known to be structurally complex (Howland 1991; de Freitas 2009; Royse *et al.* 2012) but only recently recognized as being close to the line of a major basement wrench fault (Ghail *et al.* 2015) that has been reactivated by continuing inversion of the London Basin. Mason *et al.* (2015) measured a sinistral slip rate of *c.* 1.5 mm a^{-1} over recent decades and it is likely that this movement has reactivated other basement normal faults, causing reverse offsets of the London Clay and Lambeth Group of up to *c.* 10 m (Ghail *et al.* 2014).

The Limmo Peninsula also contains several drift-filled hollows, coincident with anomalous rock head on the upper Chalk surface (Fig. 2), through which the Crossrail tunnels were anticipated to pass. Drift-filled hollows are diapiric-shaped depressions, as much as 70 m deep, filled with drift deposits (Berry 1979); the hollow to the south of the Limmo Shafts site is filled with Kempton Park Gravel to a depth of 30 m from the surface. Their origin may be a combination of fluvial scour (Berry 1979) and periglacial pingo formation (Hutchinson 1980) by sub-permafrost groundwater flow along faults (Toms *et al.* 2016). The location of two known drift-filled hollows is shown in Figure 2 as a deep depression in the Chalk surface. A hollow is also evident close to the Limmo Shafts, although this may be an extension of the known hollow to the south.

During the Paleocene, basement fault slip was dextral, and these anomalous rock head hollows may have originated as releasing-bend flower structures forming transtensional basins (Fig. 2). Given the rapid change in water depth from near sea-level during deposition of the Lambeth Group to the relatively deep-water London Clay Formation, this dextral movement most probably occurred during deposition of the upper part of the Lambeth Group, the Harwich Formation and perhaps the A2 part of the London Clay Formation, at *c.* 54.0 – 54.5 Ma. The effect of recent basin inversion (Ghail *et al.* 2015) is apparent in the site data and will be discussed below.

The hydrogeology in the London Basin is characterized by the presence of two main aquifers: the upper aquifer and the lower aquifer. The upper aquifer consists of River Terrace and alluvial deposits and is recharged by the pluvial and superficial waters. The lower aquifer is regionally more important and consists of the Chalk

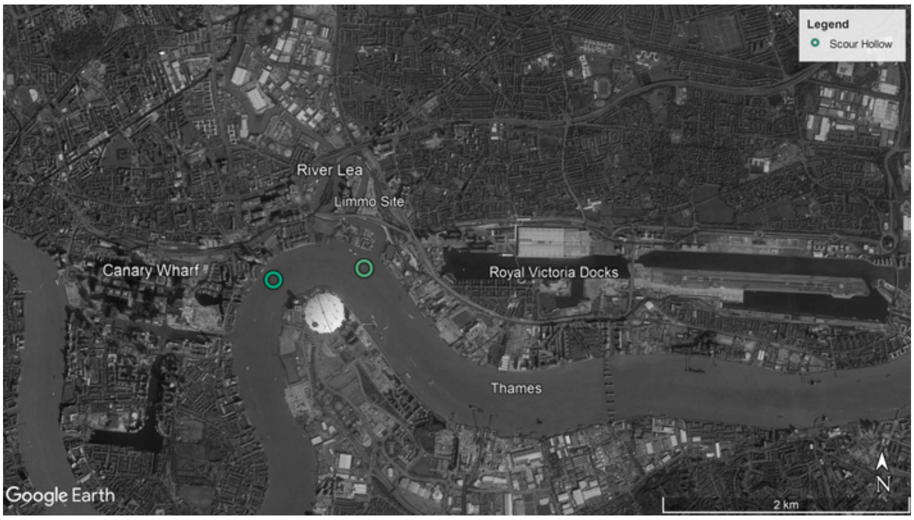

Fig. 1. Location of the Limmo site.

and Thanet Sand together with the basal sands of the Lambeth Group, the Upnor Formation. The two aquifers are separated by aquitards (London Clay and the clayey units of the Lambeth Group). The main source of recharge to the lower aquifer is the water that enters the Chalk in its existing outcrops north and south of London. Also, 2 km south of the site, the River Terrace deposits lie directly above the Thanet Sand, creating a direct connection that allows additional recharge. The granular strata in the Lambeth Group above the Mid Lambeth Hiatus, a regional erosional boundary, and in the Harwich Formation have limited connection to the lower aquifer (Roberts *et al*. 2015).

The main direct effect of this is the possible connection of the lower and upper aquifer through this feature, altering the local hydrogeology. Flow paths through faults and drift-filled hollows provide hydraulic connectivity between the upper and lower aquifers across the site, which is hydrostatic from 101.0 m ATD.

Ground investigations

Boreholes

Given this complex geological background and the scale of the structures a thorough site investigation was undertaken for the Crossrail project. The desk studies included collation of existing ground investigation data and four phases of ground investigation on site; including previous third-party boreholes, 51 boreholes were examined during the investigation (Fig. 3 and Table 1).

Recovery in the boreholes was difficult with frequent core loss reported, even in the London Clay, and the observed variations in strata interface levels are consistent with faulting across the site, suggesting throws of between 4 and 7 m. Figure 4 shows a schematic geological cross-section based on the ground investigation.

Hydrogeology and pumping tests

One of the most significant risks for SCL construction is the presence of high groundwater pressures in high-permeability deposits of the Harwich Formation and Lambeth Group. Although the SCL tunnels were anticipated to be excavated entirely within London Clay, should the amount of cover between tunnel inverts and underlying permeable strata be reduced, base heave could occur. To mitigate this risk, dewatering of the Lambeth Group and Harwich Formation would be required, similar to that employed at other sites of the project (Linde-Arias *et al*. 2015).

This risk was greater for the excavation of the main and the auxiliary shafts, given that their formation levels were lower. The base of the auxiliary shaft was in the London Clay, only 4 m above the top of the Harwich Formation, and the main shaft was in the Harwich Formation. The toe of the diaphragm wall of the main shaft is the clay of the Lower Shelly Beds and the Lower Mottled Beds, impeding the recharge of the sandy horizons above that level.

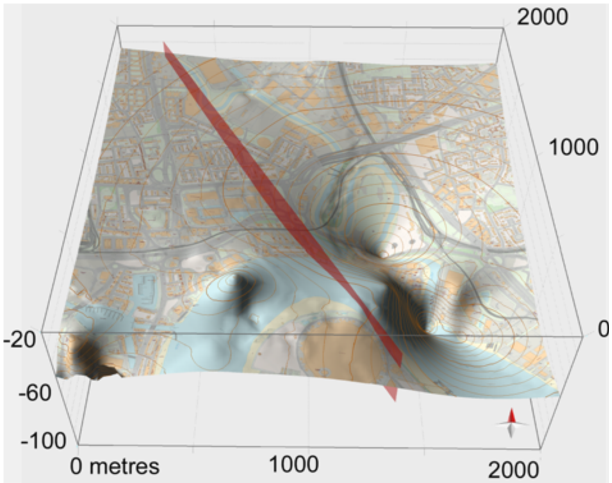

Fig. 2. Limmo Shafts area OS map plotted on uppermost chalk surface, showing approximate orientation of the active sinistral wrench fault identified by Mason *et al.* (2015). Localized hollows in the chalk surface may be indicative of drift-filled hollows, a common geohazard in the Limmo Peninsula. Contour interval is 2 m; box dimensions are 2000 (E) × 2000 (N) × 70 (H) m. Map based on Ordnance Survey Data © Crown copyright/database right 2015; model constructed in Move 2016.

Therefore, in addition to the ground investigations, a comprehensive programme of pumping tests was carried out in the different strata: Harwich Formation, sand units in Lambeth Group (above the Mid Lambeth Hiatus), Upnor Formation, Thanet Sand and Chalk (Roberts *et al*. 2015). These tests were crucial for the design of the dewatering of the lower aquifer. Table 2 summarizes the results.

The pumping tests yielded some interesting peculiarities of the hydrogeology of Limmo site. First, during the tests in the Chalk and Thanet Sand, the response of piezometers above the Mid Lambeth Hiatus indicated connectivity between the Upper Lambeth Group–Harwich Formation and the lower aquifer. Second, the results of the chalk pumping tests were sometimes inconsistent, with drawdown not always decreasing with distance away from the pumped well. Finally, the pumping tests revealed strong regular tidal fluctuation.

Although the above behaviours in some cases could be the result of inaccurate readings or due to natural variations in piezometric

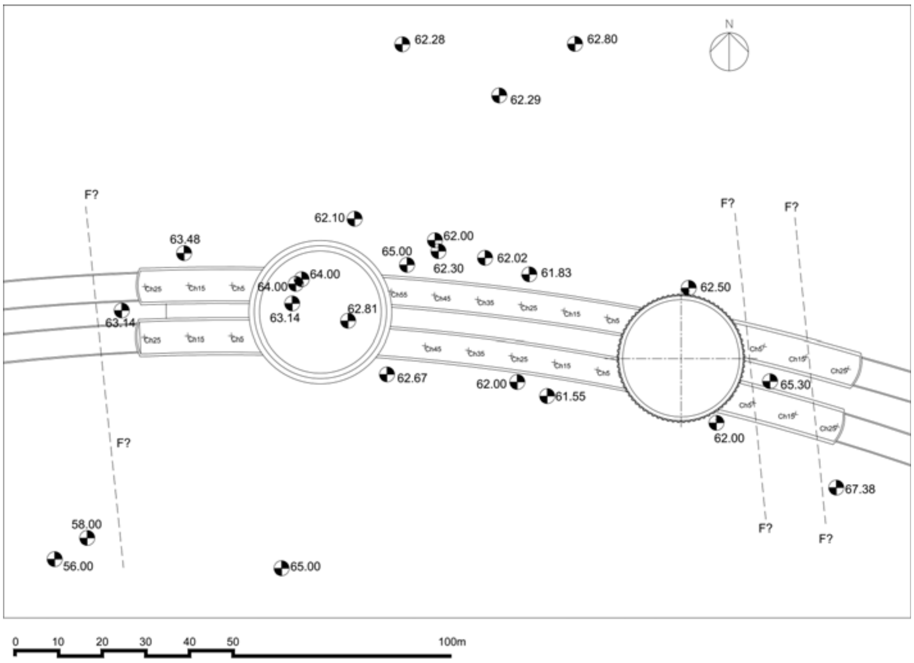

Fig. 3. Boreholes in Limmo site and some of the faults inferred during the pre-construction stage. Numbers show the elevation of the bottom of the London Clay Formation.

levels, together they indicated the presence of numerous faults that create hydrogeological compartments separated by low-permeability barriers and/or create vertical paths through the aquitards.

The analysis of the data allowed an estimate of permeabilities of 1×10^{-5} m s^{-1} for the Thanet Sand and 2×10^{-4} m s^{-1} for the Chalk. Storativity was determined as 0.003 for the Thanet Sand and 0.004 for the Chalk.

Geotechnical tests

Routine laboratory testing was performed on fine- and coarse-grained soil samples obtained from the boreholes, including classification and index tests, and routine strength tests. In all the units the index test results were typical for the London area. A exception to this was that at Limmo, where the undrained shear strength (s_u) measurements were lower than those at other Crossrail sites, especially the minimum values. Figure 5 shows a comparison of the lower bound values for the Limmo site with the rest of the East section of the Crossrail tunnel (Liverpool Street–Pudding Mill Lane) and the West section (Royal Oak–Liverpool Street).

Table 1. *Units encountered in the boreholes and the range of elevations of the top and base*

Stratum		Top (m ATD)	Base (m ATD)
Alluvium/Kempton Park Gravel		100.0 – 96.0	85.5 – 95.5
London Clay Formation		95.5 – 85.5	75.2 – 56.6
Harwich Formation		75.2 – 56.6	74.2 – 61.6
Lambeth Group	Sand Unit	74.2 – 60.4	56.9 – 56.6
	Sand Channels	56.9 – 56.6	61.8 – 55.6
	Laminated Beds	74.2 – 59.8	68.8 – 55.1
	Lower Shelly Beds	68.8 – 55.1	66.6 – 53.2
	Lower Mottled Beds	66.6 – 53.1	59.9 – 48.1
	Upnor Formation	59.9 – 48.1	57.1 – 44.6
Thanet Sand Formation		57.1 – 44.6	30.2 – 27.7
Chalk		30.2 – 27.8	n.a.

n.a., not analysed.

Given that the index properties, including mineralogy, are typical of London Clay, the clay matrix is probably unaltered. Hence, an increase in discontinuity frequency, probably through faulting, is likely to be the main reason for the low strength values. This was also borne out by poor core recovery of several ground investigation boreholes in the London Clay, Harwich Formation and Lambeth Group. This is not believed to be due to poor drilling practices as the wire line system with triple tube protection of the core was employed in the boreholes.

Direct observations during excavation

The excavation of the main and auxiliary shaft and the SCL tunnels allowed direct observations of the Quaternary deposits, London Clay and the Harwich Formation.

Stratigraphy and types of soils encountered

Made ground and Quaternary deposits.

These deposits were encountered between 107 and 97.5 m ATD during the excavation of the shafts. The made ground is *c.* 6 m thick and consists of black or grey silty, gravelly clay, often with organic matter or artificial materials and occasional fragments of bricks and wood. Numerous fragments of wood piles were extracted. At *c.* 104 m ATD during excavation of the auxiliary shaft, the foundations of a previous structure, probably the remains of former ship-building industries, were exposed, causing difficulties for the installation of some of the sheet piles.

The alluvium deposits were encountered between 101 and 97.5 m ATD. These consist of soft grey clay, sometimes sandy and with pockets of organic material, overlying the River Terrace deposits (Kempton Park Gravels). The contact has a slight eastward inclination (*c.* 3–5°) in the auxiliary shaft and is a clayey, sandy to very sandy, medium to coarse flinty gravel.

London Clay Formation.

London Clay was encountered in the shafts and the SCL tunnels, which were excavated wholly within the formation. The upper 2 m (from 93.5 to 95.55 m ATD) is weathered and consists of firm grey clay, characteristically disturbed and with a lack of discontinuities. The unweathered London Clay is an over-consolidated, stiff clay.

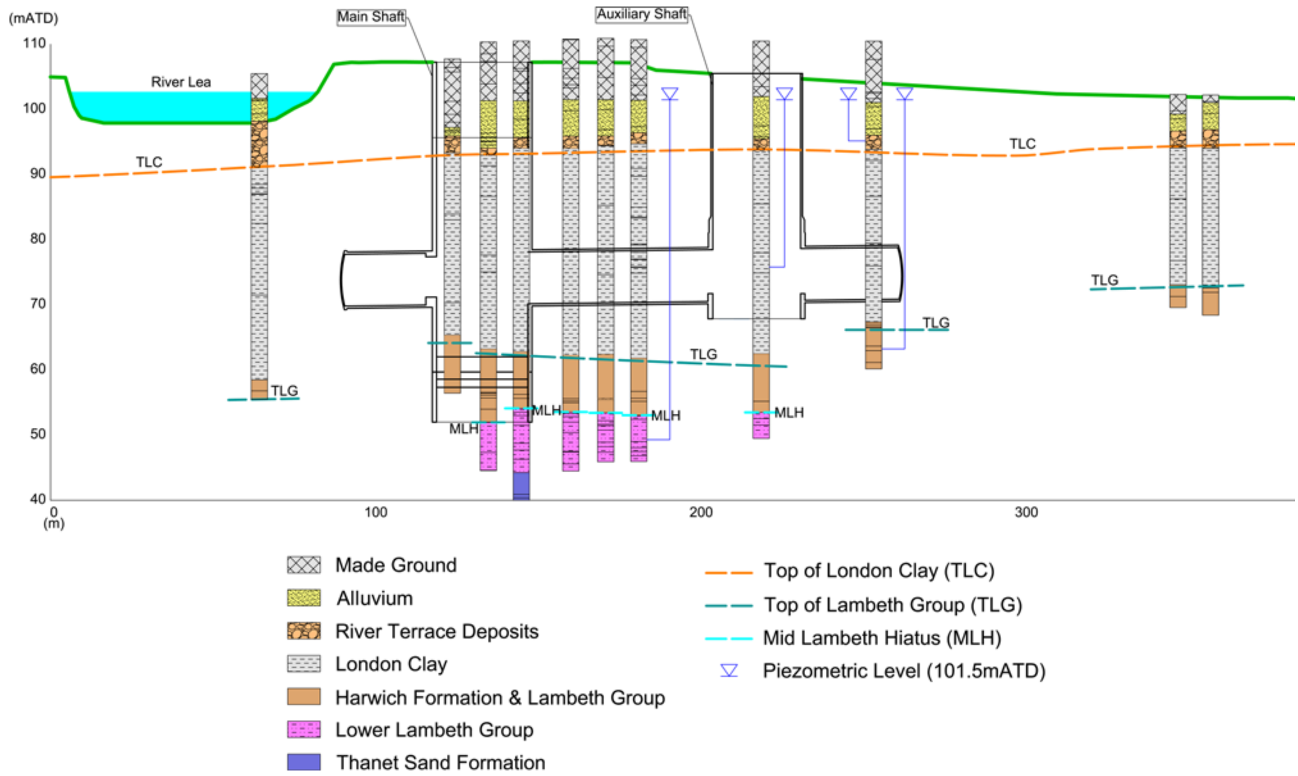

Fig. 4. Geological cross-section at Limmo site.

The formation is divided into a series of units (King 1981), of which the A2 and A3 are encountered at the Limmo site. The boundary between these two units is easily recognized by the increase in silt content and the absence of claystones in the A2. Fissures are a common feature in the London Clay Formation. Usually, they are regarded as of synsedimentary or lithogenic origin. In the Limmo area the spacing is small, ranging from medium to extremely closely spaced. Polished and slickensided surfaces (commonly known as 'greasybacks') were encountered but no trend in the orientation was identified. Joints, defined as discontinuities longer than 1 m and probably tectonic in origin, are also very frequent. They are usually planar and polished, occasionally slickensided. Apertures range between 0.1 and 1 mm, and are sometimes filled with a soft clay. Joints are typically spaced at 100 to 2 m intervals. Discontinuity orientations were recorded during excavation of the main shaft (Fig. 6). The primary set is subvertical, striking WNW–ESE, with a second set striking north–south. However, discontinuities may strike in any direction with dips as low as 40–60°.

Harwich Formation.

The Harwich Formation was encountered only during the excavation of the main shaft, where it is a thin 0.6 – 1.7 m thick stratum consisting of two facies. One of these is a dark grey to very light greenish-grey slightly sandy clay representing the Swanscombe Member, which also contains rare calcrete concretions with a thickness of *c.* 200 mm. They are described as a moderately strong light grey limestone with occasional white fine gravel-sized shell fragments. The other is the Oldhaven Member, represented either by a fine to medium dense brown sand with many white shell fragments or by a moderately strong light grey coarse-grained shelly sandy limestone with rare rounded coarse gravel of black flint.

Table 2. *Pumping test regime*

Stratum	Well depth (m)	Flow rate at steady state (l s^{-1})	Pumping
Harwich	47.5	0.18	69 h
Harwich	49.3	0.14	46 h
Lambeth	55.5	0.4	39 h
Lambeth	57	0.52	42 h
Thanet	79	2.5	40 h
Chalk	114	20	9 days 10 h

Faulting

Observed faults

Desk studies and ground investigation data meant that faulting was anticipated at the Limmo site: the variation in the elevation of the boundaries between different strata is a clear indicator. Figure 7 shows an estimation of the contours of the base of the London Clay Formation using the boreholes, which suggest the presence of faults especially to the south of the shafts.

Despite the above, it proved very difficult to locate the faults in the borehole cores. Even during the SCL works the relative homogeneity of London Clay, with no detectable compositional contrast, made the detection of faults very difficult. Also, faults are complex structures with different elements such as the slip surface or fault core, which accommodates the majority of the strain, and the surrounding damage zone, with low strain and subject to brittle deformation. However, three faults were inferred directly from observations made during the works.

Main Shaft Westbound Launch, Chainage 10.

Between Chainages 9 and 12 a *c.* 1 m band of highly fractured clay was encountered (Fig. 8), with an orientation of *c.* 70/150 (dip/azimuth). It was assumed to be a fault, but its slip direction could not be estimated.

Auxiliary Shaft Eastbound Launch, Chainage 12.

The Eastbound Launch tunnel excavated from the auxiliary shaft started with the top heading completely in the A3. Between

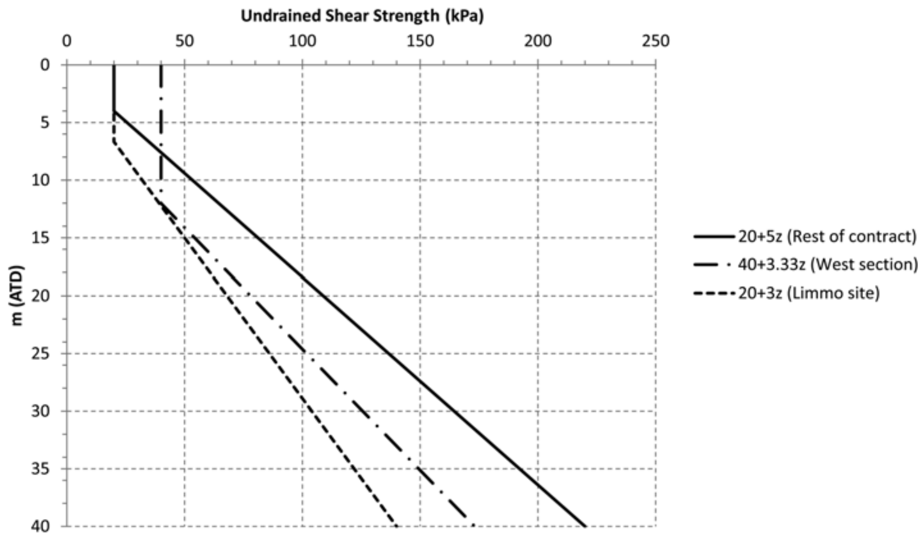

Fig. 5. London Clay lower bound values of the undrained shear strength obtained for Limmo site and for all other Crossrail sites.

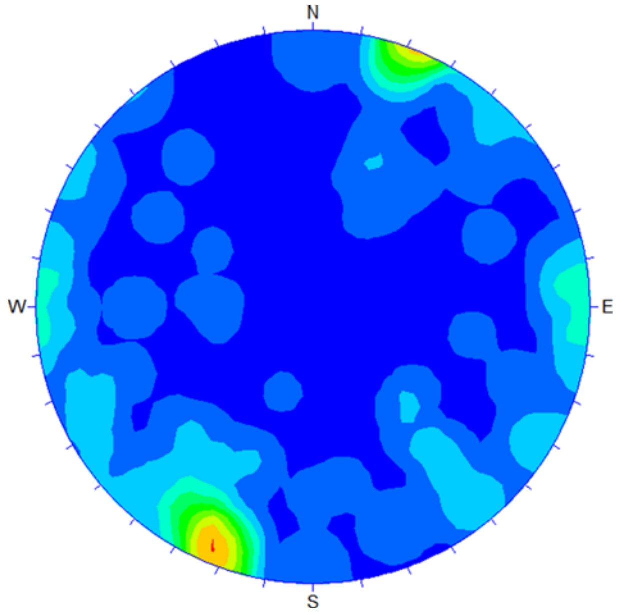

Fig. 6. Stereogram polar density plot of discontinuity orientation.

Chainage 10 and 12 a change was detected, with the A2 unit in the top heading instead of the A3 as previously, implying a fault displacement of several metres. This was detected by the absence of claystones and the increase of silt content.

Main shaft.

In the SCL tunnels west of the main shaft, the A2–A3 boundary was encountered higher than in the SCL tunnels to the east of the shaft by *c.* 1.5 m. This indicates the possible presence of a fault in the main shaft.

Geological interpretation

The faults, fissures and differences in levels across the Limmo area may best be understood in light of the wider geological context detailed above. Inversion of the London Basin reactivated the basement strike-slip faults in a sinistral sense, so that what had been a transtensional releasing bend during the Paleocene is now a transpressional restraining bend. The oblique normal faults of the flower structure below the London Clay Formation reversed and propagated into the previously unfaulted younger sediments, generating new compressional flower structures above each of the reactivated faults below, resulting in considerable structural complexity across the site (Fig. 9)

Repetition of River Terrace deposits in one of the boreholes (B254) indicates that the faults here have been active within the last *c.* 100 kyr. InSAR data (Mason *et al.* 2015) show strike-slip displacements close to the Limmo area of *c.* 1.5 mm a^{-1} over recent decades but it is not clear whether the faults here are active at the present day. If they are, these faults will translate strike-slip displacement into reverse movements, most probably accommodated by creep of the soil around the tunnels rather than displacement of the tunnels themselves. Whether or not these

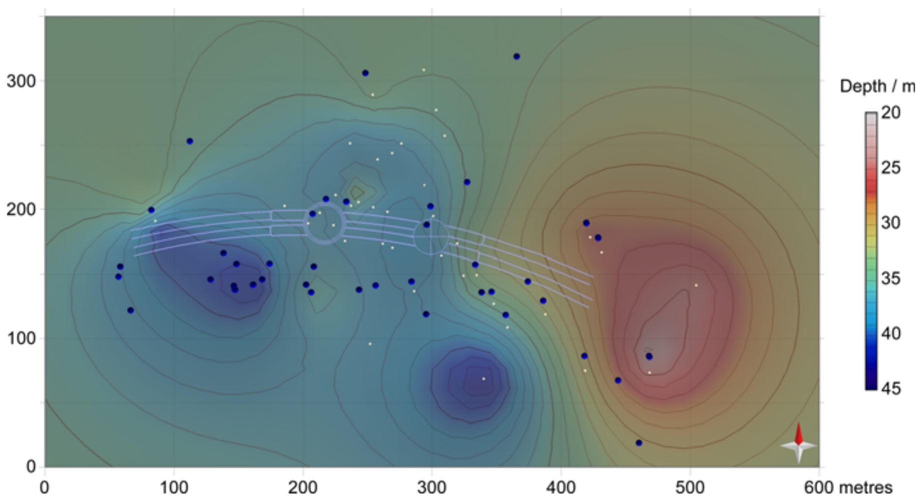

Fig. 7. Estimated contours of the base of the London Clay Formation across the Limmo site.

Fig. 8. Shear zone encountered between Chainages 9 and 12.

faults are creeping, they remain lines of structural weakness in the ground and conduits for groundwater flow.

Figure 10 represents an idealized block of this structure for clarity.

Hydrogeology

After the ground investigation, it was concluded that it was necessary to install deep wells in the Thanet Sand and the Chalk to reduce the groundwater pressures in the lower aquifer to avoid uplift in the base of the main shaft. In the auxiliary shaft, the risk of uplift failure from the groundwater pressure at the top of the Harwich Formation was mitigated with passive wells.

The piezometers installed to monitor the drawdown initially showed hydrostatic continuity between the London Clay and Thanet Sand Formations (Fig. 11). However, the dewatering from the Thanet Sands and the Chalk has altered this pattern, with dewatering having a greater effect in the lower aquifer (in the basal sands of the Lambeth Group and Thanet Sand Formation). Piezometers in the Thanet Sand Formation show less drawdown because of their greater distance from the main shaft, where most of the pumps were installed. Piezometers in the Harwich Formation and the lower part of the London Clay were slightly under-drained, reaching hydrostatic equilibrium with the lower aquifer. Instruments installed at higher levels in the London Clay Formation were not affected by the dewatering.

Hydraulic continuity between the upper and lower aquifers was a concern prior to the excavation of the tunnels, but no water was encountered in the SCL tunnels except for a slight seepage in the launch adits in the western part of the site, which are in close proximity to the River Lea. The flow of water, *c.* 1 l min^{-1}, was accommodated through a PVC drain.

Difficulties during SCL works

During construction of the SCL works, localized face instability occurred. Although primary lining deformation was consistently lower than anticipated, the presence of fissure sets created the conditions for several localized failures, which behaved similarly to a fractured rock mass. Exclusion zones were carefully implemented during SCL tunnelling activities and the use of spiles (steel bars that are inserted to pre-support the excavation) on alternate top headings also ensured the tunnels were constructed safely with ground instability reduced.

Bench failures

The tunnelling sequence was of top heading, bench and invert. The bench stage resulted in a subvertical slope *c.* 2.5 m high and several episodes of slope instability occurred during its excavation. The most frequent were planar-type failures along polished joints (greasybacks), which occurred when joints daylighted in the excavation with dip angles sufficiently high (>45°) for the bench to become unstable within hours (Fig. 12). These failures were particularly prevalent in the western section of the site and may

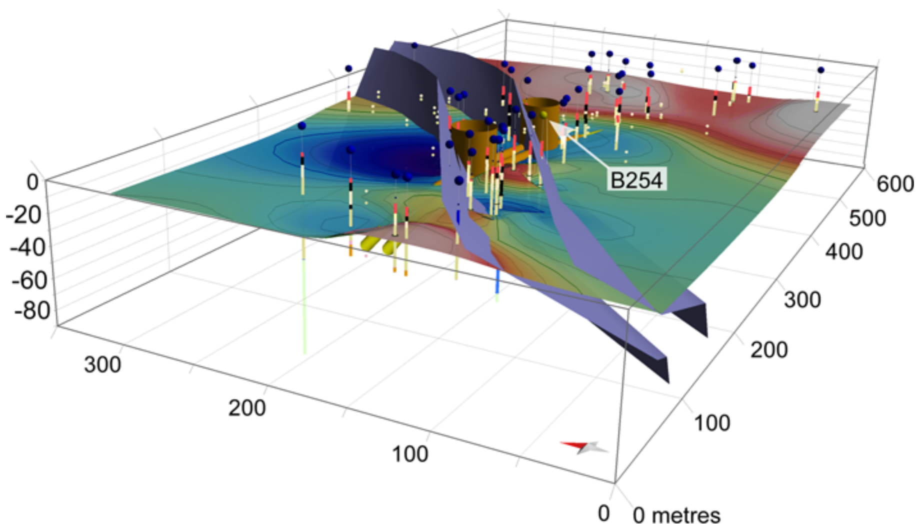

Fig. 9. Simplified geological model of the Limmo Shafts area; the nearer of the shafts (both in orange) is 30 m in diameter and 44 m deep; the farther is 27 m in diameter and 39 m deep. The 7.1 m diameter running tunnels are at 73 m ATD. Two steep reverse faults (dip-azimuth of 70 – 150) are shown propagating up from one of the faults in the Chalk (shown in Fig. 2); the change in levels to the SE (far right) is probably caused by a third similar fault, which for clarity is not shown. Minor faults and periglacial features are omitted. The northern fault intersects the tunnel, as was observed (Fig. 6), whereas the southern fault causes the observed repetition of strata in borehole B254. The coloured surface from blue at 70 m to white at 89 m ATD is the top of the London Clay Formation A2 layer (King 1980), generated by inverse distance weighted borehole data (rotary with tracks shown and percussive as spot points). The borehole tracks are coloured by strata; above the London Clay these are Made Ground in red and River Terrace deposits in beige. It should be noted that the SCL tunnels are mostly in A3 but the exterior TBM tunnels are in A2. Model constructed in Move 2016.

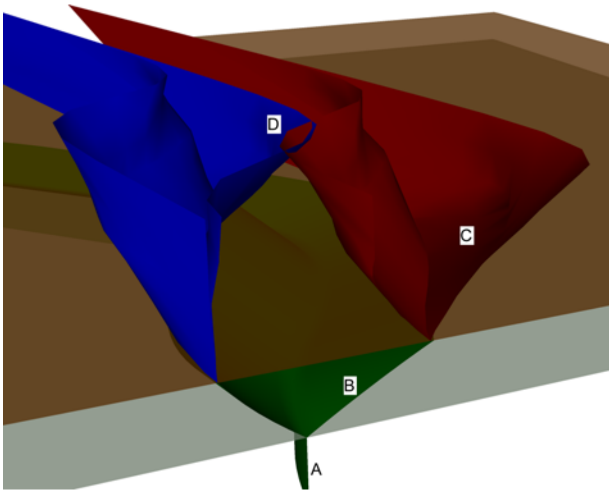

Fig. 10. Idealized section and block diagram illustrating the propagation of fractures associated with a bend on a strike-slip fault, through younger sediments. A bend (A) on a basement strike-slip fault causes a releasing bend pull-apart basin (idealized at B) in the Chalk, probably during early Tertiary extension. This is reversed by recent inversion (shortening) aligned slightly obliquely to the basement fault (note that the straight part of the blue and red fractures is oriented differently from the straight line part of the green fracture), and generates new restraining bend push-up structure (C and D) in the Tertiary sediments (Lambeth Group and London Clay Formation), one new push-up structure for each fracture in the Chalk pull-apart structure. The complexity this generates at (D) should be noted; in reality, many fractures would have formed in the pull-apart basin (B), each generating a push-up structure containing many fractures. Hence the ground in these areas becomes intensely fractured and weakened.

therefore be associated with the faults and structural weaknesses exploited by the River Lea.

In some cases it was not possible to batter the slope sufficiently as there was a requirement for a minimum separation between bench and top heading. Pocket excavation (reducing the area of the

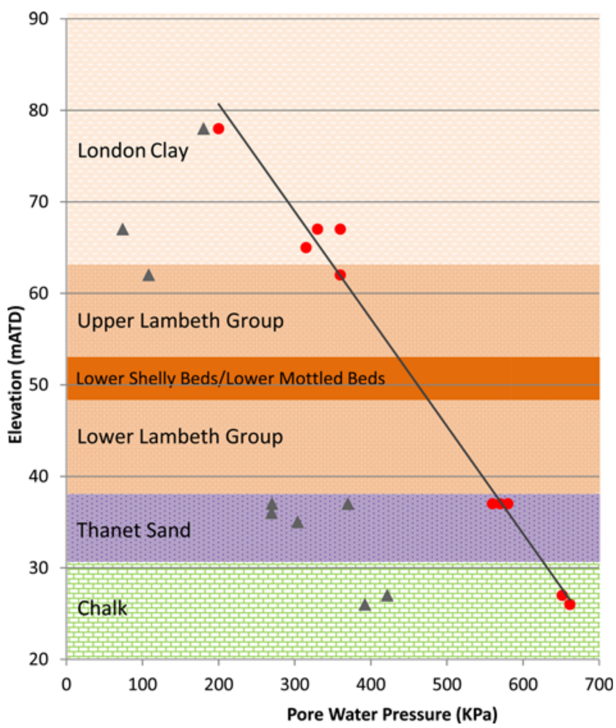

Fig. 11. Profile of porewater pressure in newly installed piezometers, pre-watering (circles) and post-dewatering (triangles).

Fig. 12. Planar failure during bench stage.

Fig. 13. Circular failure during bench stage.

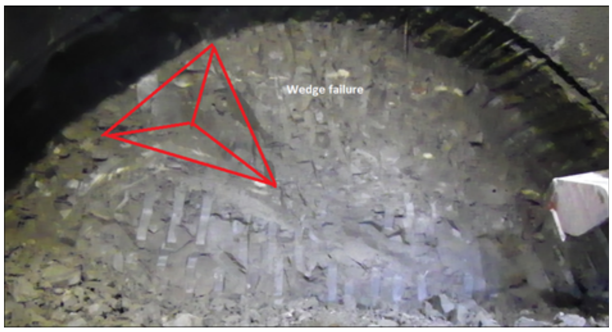

Fig. 14. Wedge failure during top heading stage.

excavation by splitting the face into two or three smaller sections that are excavated sequentially) and increasing the thickness of the 75 mm sealing layer were some of the 'tool-box' actions implemented.

In another instance, a local increase in the frequency of joints and fissures led to a rotational failure. As a consequence, the clay behaved similarly to a blocky rock mass (Fig. 13).

These instabilities of the bench stage reduced the rate of progress of the excavation.

Top heading instabilities

The presence of discontinuities also affected the stability of top headings. There were numerous examples of wedge failures during the excavation. Figure 14 shows an example of a localized wedge fall from the top heading face that occurred as a result of the intersection of three discontinuities.

Although the stability of the tunnel was not at risk, these phenomena could result in accidents for the operatives and were identified as a safety risk. Several risk reduction actions were put in place to mitigate face instability. Geotechnical logs were made of each excavated face, including descriptions of discontinuities, and the installation of 35 mm rebar spiles to pre-support the excavation was made mandatory by the designer to avoid failures in the crown and outside the face. The spiles were driven in from lattice girders in the top headings, required in the temporary condition because of the close proximity of the two running tunnels in faulted ground, and an exclusion zone was implemented until the required sprayed concrete early strength was achieved. This ensured that personnel were not put at risk during excavation and spraying. Pocket excavation was used on encountering large-scale discontinuities to mitigate the risk of face instability.

Conclusions

The information obtained during the ground investigation for the Limmo site suggested the presence of faults. Among others, some of the indications were sudden changes in the elevation of boundaries between strata, low recovery in some of the boreholes in the London Clay and presence of barriers of low permeability in the pumping tests.

The Limmo tunnels were excavated within the London Clay Formation, normally a stiff over-consolidated clay that in the Limmo area is heavily fissured with numerous discontinuities more than 1000 mm long forming several distinct joint sets. These discontinuities affected the design by causing the undrained shear strength to be much lower than usually expected for London Clay.

The faults, with throws between 4 and 7 m, are inferred to be part of a transpressional restraining bend flower structure overlying an older transtensional flower structure reactivated during present-day inversion of the London Basin.

As well as weakening the ground, the faults, fissures and joint sets also provide hydraulic connectivity between the upper and lower aquifer, although this did not lead to significant inflows of water during excavation.

The discontinuities and joint sets led to several episodes of instability during the excavation, requiring a number of measures including face logging, pocket excavation, spiling and implementation of exclusion zones to mitigate against this risk, which allowed the tunnels to be constructed safely and on programme.

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4.1.2 Drenaje para la excavación de la caverna de Stepney Green.

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Depressurisation for the excavation of Stepney Green cavern

1 **Emilio Linde-Arias** BSc, MSc, DIC, EurGeol, FGS
Senior Engineering Geologist, OTB Engineering Limited, London, UK

2 **David Harris** BSc, MSc, DIC, CGeol, FGS, CEng, MICE
Principal Engineer, Mott MacDonald Group Limited, Croydon, UK

3 **Andrew Davis** BSc, MSc, CGeol, FGS
Principal Engineering Geologist, Mott MacDonald Group Limited, Croydon, UK

At over 50 m long, 17 m wide and 15 m high the Stepney Green Cavern is one of the largest caverns ever built using sprayed concrete lining techniques in central London. The east- and westbound caverns lie approximately 20 m below the existing ground level in a constrained urban site and are located within the London Clay and underlying Lambeth Group, which is a variable series of clays, silts and sands; the higher permeability sands contain high pore pressures that are hazardous to sprayed concrete lining construction without depressurisation. Successful implementation of a practical depressurisation scheme was therefore crucial to safe construction. The overall strategy adopted for investigating these water-bearing sands within the Lambeth Group is outlined and the ground investigations and pumping tests required to determine typical material permeabilities and the response of the sands to pumping are described. The paper also reviews how that information was developed into the outline depressurisation scheme and how this was subsequently developed into the final contractor-designed depressurisation scheme that was implemented. Changes to the scheme during construction owing to ground conditions are discussed, together with the response of the ground during pumping.

1. Introduction

Crossrail is a new railway currently under construction in London, and when completed will significantly reduce journey times across the capital. It includes approximately 10 km of sprayed concrete lining (SCL) tunnels at station, ventilation shaft and junction locations. One of the largest of these tunnels is being constructed at Stepney Green, east London, to facilitate construction of a sub-surface junction. This will allow Crossrail trains from Maidenhead and the west to travel to Shenfield and Woolwich in the east. The excavation and installation of the primary sprayed concrete lining at Stepney Green was completed in August 2013.

At over 50 m long, 17 m wide and 15 m high the Stepney Green Cavern is one of the largest caverns ever built using SCL techniques in central London. The east- and westbound cavern crowns lie 17 and 21 m, respectively, below ground level in a constrained urban site and are situated within the London Clay and underlying Lambeth Group formations. The Lambeth Group is a variable series of clays, silts and sands, and the higher-permeability sands contain high pore pressures which are hazardous to SCL construction (Ciria, 2004). The significant hazard is the potential for groundwater seepage to occur within soils with limited cohesion, resulting in running sand, face instability and increased ground movements. Depressurisation of these high pore pressures was crucial to the safe construction of the caverns and compliance with the zero-harm Crossrail ethos.

On the Crossrail project a distinction was made between use of the terms ‘dewatering’ and ‘depressurisation’ in order to facilitate discussions with third parties regarding the likely impact of any measures affecting groundwater. Dewatering was considered to apply to any pumping undertaken within the Lower Aquifer, with wider environmental implications, and depressurisation for pumping undertaken in the overlying Lambeth Group. For the purposes of consistency the term ‘depressurisation’ has been used in this paper.

The SCL permanent works were designed by Mott MacDonald (contract C121), with the construction undertaken by the contractor, Dragados–Sisk Joint Venture (DSJV), under contract C305. Temporary works design was provided to DSJV by OTB

Engineering, with the well systems being provided by WJ Groundwater.

2. Geology

The geology at Stepney Green is typical for that of east central London; comprising a sequence of strata from Thanet Sand, the complex Lambeth Group strata, the Harwich Formation and the lower A2 and A3 units of the London Clay (King, 1981). A summary is provided in Table 1 and typical geological sections of the eastbound and westbound caverns are presented in Figures 1 and 2.

The eastbound SCL cavern is located predominantly within the London Clay A2 unit with Upper Mottled Beds at the invert level. The westbound cavern is approximately 4 m deeper, causing the excavation to be within London Clay/Harwich Formation, with up to 4 m of Lambeth Group between the axis level and the invert level.

Within the Lambeth Group there occur relict sand channel deposits related to the palaeo-environment present during the geological period of deposition, representing a series of meandering channels in an estuarine-type environment. These channels vary in frequency, width, depth, length and infill material, and, as a result, are spatially unpredictable and therefore problematic to identify by ground investigation with any degree of certainty. There are examples of large extensive channels proven by Crossrail ground investigations, such as below Bond Street station where identification and extrapolation are more certain. However, the channels identified at Stepney Green tend to be thinner, typically to a maximum thickness of 4 m, less extensive and encountered variably throughout the Lambeth Group. It was known from the ground investigation that to the west of Stepney Green there was a thick, persistent sand unit but towards the east, where the major caverns were to be excavated, the sand units became thinner and laterally impersistent, being represented as channels and lenses.

Strata		Typical thickness: m	Elevation to top of strata: m ATD	Typical description
Made ground		2	110	Variable
River Terrace Deposits		3	107	Medium dense, orange brown, fine to coarse sand/gravel and rare cobbles of flint
London Clay		23	104	Firm to very stiff fissured clay with silt partings and laminations. Increase in silt partings and becoming slightly sandy in the A2 unit
Harwich Formation	Swanscombe Member	1	80	Stiff to very stiff fissured sandy clay
	Oldhaven Member		80	Medium dense, slightly clayey, fine to medium sand
Lambeth Group	Upper Mottled Beds	5	80	Very stiff to hard (friable microfissured) very closely to extremely closely fissured light grey to blue grey mottled light brown and red sandy to very sandy silty clay
	Sand channels	Variable up to 4 m	79	Medium dense to very dense greenish grey fine to coarse sand
	Laminated Beds	3	75	Stiff to very stiff thinly laminated fissured grey to dark grey, sandy to slightly sandy, silty to very silty clay, with extremely to very closely spaced thin laminations of light grey slightly sandy silt
	Lower Shelly Clay	1	72	Stiff to very stiff, fissured, thinly laminated to thinly bedded dark grey and grey, silty to locally very silty clay with occasional bands of light grey sandy silt. Frequent shells and shell fragments
	Lower Mottled Beds	Variable up to 8 m	71	Very stiff to hard, multi-coloured, extremely closely fissured silty to very silty, slightly to very sandy clay
	Upnor Formation	Variable up to 7 m	66	Very stiff, dark grey-green, brown very sandy silty clay, with occasional fine to coarse rounded flint gravel Very dense, black, rounded medium and coarse flint gravel. Very dense brown or green to dark green speckled clayey to very clayey silty fine to coarse sand, with a little too much fine and medium rounded flint gravel
Thanet Sand		11	62	Very dense, grey green, silty fine and medium sand

Table 1. Geological sequence at Stepney Green

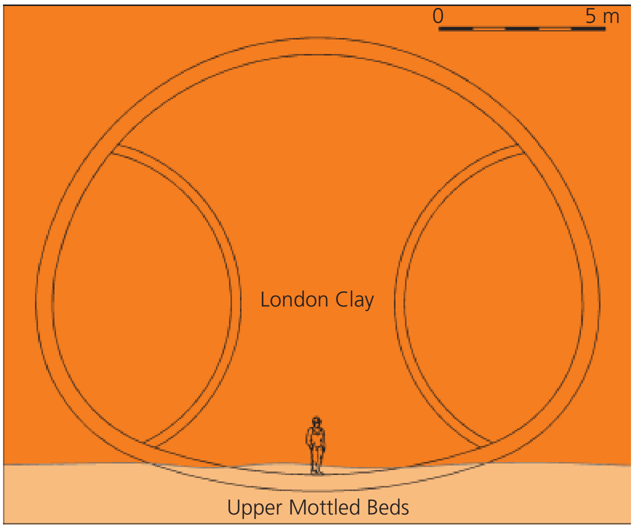

Figure 1. Typical geological section along the eastbound cavern

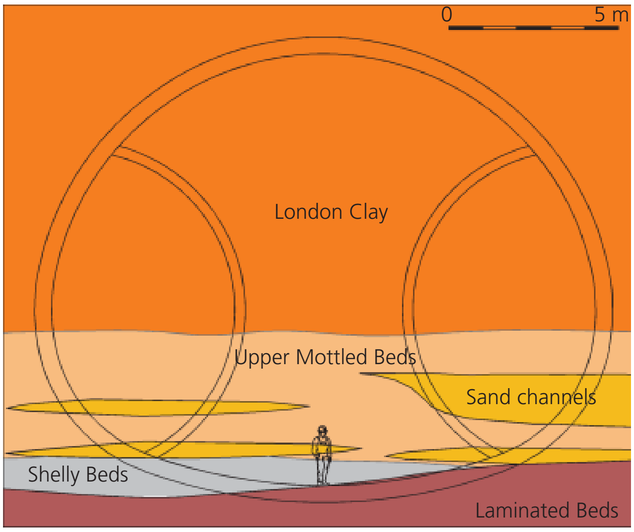

Figure 2. Typical geological section along the westbound cavern

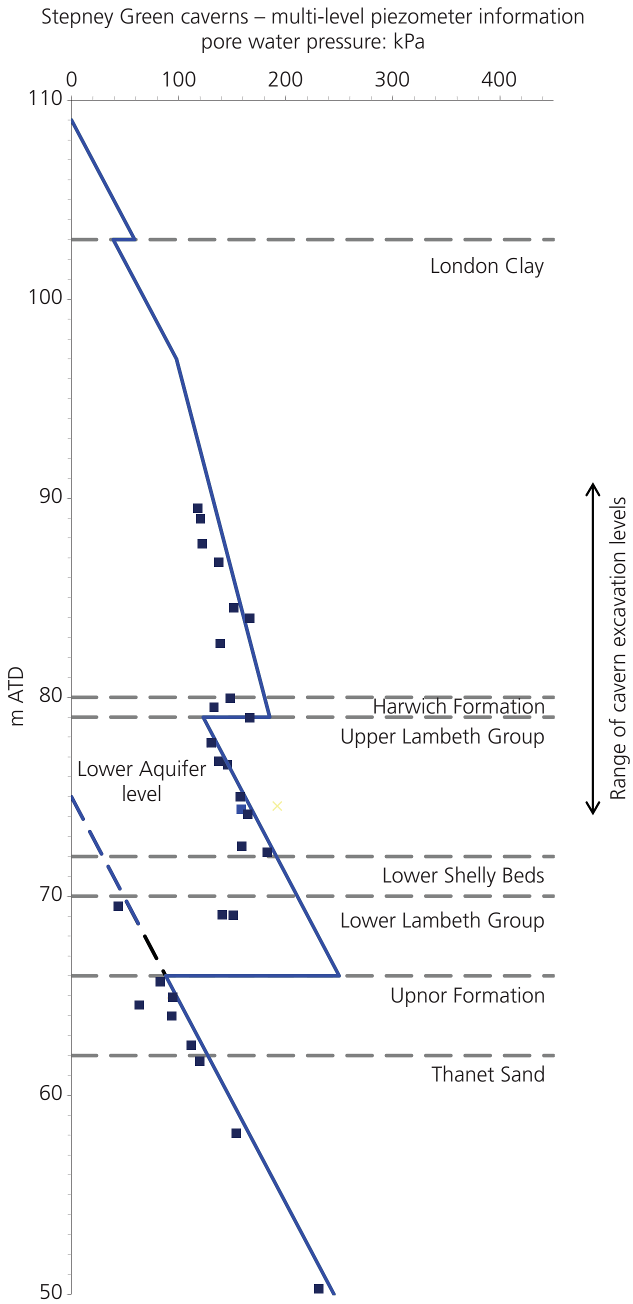

Figure 3. Multi-point piezometer readings and adopted pore pressure profile (ATD, above tunnel datum)

3. Groundwater

The groundwater profile at Stepney Green (Figure 3) is typical for the London area, comprising an upper aquifer located in the superficial deposits and a lower aquifer within the Chalk, Thanet Sand and Upnor Formation. The historical effects of groundwater abstraction from the Lower Aquifer have resulted in an underdrained profile with pore pressures within the Thanet Sand and Upnor Formation being much lower than those above in the London Clay and Upper Lambeth Group. The pore pressures within the SCL excavation depth range were variable and related to the underdrainage effects, with maximum values of approximately 180 kPa at the base of the London Clay dropping to 120 kPa at the top of the Lambeth Group. The expected groundwater pressures to be encountered in any sand channels in the vicinity of the SCL works was therefore considered to be in the range 120–150 kPa. This detailed profile was able to be determined owing to the inclusion of several high-quality multi-point monitoring systems within the ground investigation.

4. Ground investigation

Owing to the relatively long planning stages of Crossrail, four phases of ground investigation were undertaken at Stepney Green over an 8-year period to obtain geotechnical and groundwater information. The ground investigations included cable percussion and rotary boreholes and two in situ pressuremeter testing boreholes, and concluded in the final phase with a pumping test within the Lambeth Group. In total, 32 boreholes were drilled within 100 m of the proposed cavern locations.

Groundwater monitoring installations are summarised in Table 2 and consisted of conventional 25 mm Casagrande piezometers, 50 mm standpipes, vibrating wire piezometers, continuous multichannel tubing and a multi-port sampling system. The construction of groundwater monitoring installations was carefully supervised, with none of the piezometers exhibiting erroneous hydraulic connectivity between aquifers. The multi-port systems proved particularly useful, as well as economic, despite initially longer installation times.

5. Pumping test

A pumping test was specified to determine the permeability of sand channels within the Lambeth Group and to provide data to allow design of the depressurisation requirements. The pumping test comprised three pumping wells with 7 d of pumping and 7 d of recovery monitoring. This was considered to more closely reflect the likely actual multiple pumping set-up to be used during construction, rather than using a traditional single pumped well. The three pumping wells comprised 150 mm slotted pipe installed in 300 mm diameter boreholes with a sand filter pack. Each well had a screen of between 8 and 8·5 m length through the Upper Mottled Beds and Laminated Beds including sand channels. The wells terminated in the Lower Shelly Beds, which were below the invert levels of the two caverns. The Harwich Formation was sealed off from the well screens to avoid potential misleading results.

Monitoring of piezometers, standpipes, continuous multichannel tubing and vibrating wire piezometers was undertaken prior to and during the test at intervals in accordance with BS EN ISO 14686:2003 (BSI, 2003). These were supplemented by manual monitoring of key installations, to ensure that, in the event of a data-logger breakdown, manual data could be used to supplement the results.

Type	Total
19 mm piezometer	22
50 mm standpipe	5
150 mm standpipe (well)	3
Vibrating wire piezometers	14
Continuous multichannel tubing system	12
Multi-port sampling system	6
Total	62

Table 2. Summary of groundwater installation details at Stepney Green

A series of three-step tests was undertaken to determine the maximum yield of each well. A constant discharge test was then undertaken using three submersible pumps. The headworks of the three wells were sealed and a vacuum pump was used to improve the yield. The test commenced with one well pumped at approximately 1 l/s with a vacuum applied. After 1 d a second well also commenced pumping under vacuum, increasing the combined flow rate to 1·3 l/s. Finally, the third well commenced pumping under vacuum, with a slight reduction in the flow rate. Pumping stopped after 7 d at a final combined flow rate of 1·17 l/s. The maximum yields obtained during the test from the three separate wells were

- first well: 1·06 l/s with partial vacuum of $-7{\cdot}3$ m H_2O
- second well: 0·58 l/s with partial vacuum of $-8{\cdot}1$ m H_2O
- third well: 0·45 l/s with partial vacuum of $-6{\cdot}0$ m H_2O.

The maximum drawdown within the Upper Lambeth Group achieved at the end of the 7 d pumping period was typically 8 m at a distance of 10 m, and 3 m at a larger distance of 85 m. There was insignificant drawdown in the underlying Lower Aquifer.

The wells took approximately 10 d to recover to pre-test levels, with the majority recovering within the first 7 d. The test showed that the sand channels generally had connectivity within the Lambeth Group, which provided confidence in the ability to successfully depressurise the Lambeth Group from surface wells. As expected, there was insignificant drawdown in the overlying River Terrace Deposits or London Clay. Although the pumping test was not anticipated to produce surface settlement, owing to the limited duration and relatively small volumes of discharge, precise levelling was undertaken to demonstrate this, which could subsequently be used to reassure adjacent stakeholders.

The pumping test results were analysed using a range of methods. The main method of analysis was the Cooper–Jacob well field method (Kruseman and de Ridder, 1994) because of the increase of the pumping rate in steps as the second and third wells were added. The conventional Theis curve-fitting method was also used and the Theis' recovery and the steady-state Theim's method were used (Kruseman and de Ridder, 1994). The results of the analysis indicated a transmissivity of between 5 and 12 m^2/d (permeability of 6×10^{-5} to 1×10^{-4} m/s) and a storativity of 7×10^{-4} to 6×10^{-3}.

6. SCL ground-related risks

SCL tunnel excavation is undertaken by spraying concrete directly onto exposed excavated ground, forming an immediate bond with the strata. The lining can follow complex profiles and layouts, in-fill overbreak, and forms a strong, durable and watertight shell. As such it is an ideal method for the construction of complex geometrical layouts such as the Stepney Green caverns. The method does, however, require that the excavated ground remains stable in order that the sprayed concrete can be applied. Although the geology and groundwater conditions at Stepney Green are not unusual for east

London, SCL introduces greater ground risk than use of a tunnel-boring machine. The greater ground risk is related to the absence of the continuous face support provided by a slurry or earth pressure balance tunnel-boring machine. The need to control groundwater ingress and pressures where sandy materials occur in an open face or at an invert is very important if stability, and safe conditions, are to be maintained prior to application of the sprayed concrete.

If the groundwater pressures at the face of the excavation are not reduced adequately they will cause instability of sand layers, resulting in the sand running into the excavation. If sand comprises a significant proportion of the face, then direct face instability can occur. In situations where minor sand/silt strata occur (such as laminae), the wash-in of soil may cause local undermining of the face and subsequent indirect instability. Any such loss of material from the face could trigger a fall of ground from the face, overbreak and additional settlement. Where the invert is within sandy material, the presence of elevated pore pressures can lead to the development of running sand conditions. In addition to stability concerns, groundwater ingress can result in poor working conditions which are unacceptable on health and safety grounds and can affect the quality of the sprayed primary lining.

The extent of the risk and the form of the mitigation depend on a number of variables

- the size of sand units that control the quantity of potentially stored groundwater
- the permeability of the sand units which controls the rate of water inflow
- the ability of the sand units to recharge, which controls the time over which inflow may occur
- the magnitude of the pore-water pressures present, which affects the amount of inflow
- the location of the sand channel in relation to the tunnel excavation profile.

The risk of instability at Stepney Green was particularly hazardous owing to the unusual size of the excavations required to construct the 17 m diameter caverns. Previous works (as summarised by Mair and Taylor (1997)) have demonstrated that the risk of instability increases with tunnel diameter.

7. Depressurisation strategy and design

7.1 Preliminary design

Depressurisation works were considered as temporary works, and the responsibility for the design and implementation lay with the contractor. However, in order that the tender process could be assessed on an equal basis, the designer derived an indicative preliminary design for pricing purposes. The successful contractor was then responsible for taking forward the design to suit their own assessment and programme of work to satisfy a performance specification.

The key aims of the preliminary depressurisation strategy were to address both the unpredictable nature and location of the water-bearing strata and the high pore pressures that would influence the practicality of maintaining safe excavations. To adequately reduce the level of risk, there was a need to undertake SCL excavation with a degree of confidence that unexpected water-bearing sandy materials were unlikely to be encountered. In order that the risk could be maintained ‘as low as reasonably practicable’ there was a requirement that construction investigation works were comprehensive.

To address that risk, an extensive investigation and depressurisation scheme was developed combining both surface and in-tunnel works. The Stepney Green work site is small, and the adjacent third-party land (including an urban farm, protected archaeological area and an all-weather sports facility) was not considered appropriate for undertaking construction-based activities. The potential for surface depressurisation was optimised where possible by the inclusion of inclined wells drilled from within the work site, but terminating at the tunnel level beyond the site. The potential to maximise use of surface well sites was thus constrained, and it was therefore considered that it was not possible to confidently identify and depressurise all water-bearing materials by surface works alone.

Based on previous experience (Hartwell *et al.*, 1994) a maximum pore pressure of 100 kPa was adopted in the indicative designs as the limit at which sub-surface in-tunnel wells could practically be installed. This value was related to the difficulty of controlling groundwater ingress and bore stability during drilling, and the ability subsequently to install a well within the bore. Given the expected pore pressure values of up to 150 kPa, it was considered that pore-water pressures would need to be reduced, as a minimum, to less than 100 kPa by surface wells in advance of SCL tunnel excavation.

As it was considered that surface wells alone could not produce a dry excavation, it was envisaged that additional wells would be installed from within the shaft and/or tunnels. As the locations of the sand channels were not accurately known, it was considered that a robust array of investigatory probing would be taken ahead of the advancing SCL face to identify any groundwater-bearing strata. Wells would then target those locations where groundwater was identified. The number of wellpoints and wells required, and the time required to obtain the necessary depressurisation, would depend on the extent of the water-bearing materials, their permeability, pore-water pressure and amount of recharge potential. It was expected that different drilling patterns and numbers of probe holes would relate to the different excavation sequences.

The indicative tender design comprised a total of 25 surface wells and 140 in-tunnel gravity and vacuum wells. The surface well spacing was based on a typical 8 m drawdown cone being achieved around each well, such that the individual well spacing was 16 m. Surface wells were anticipated to comprise pumped and/or ejector wells, depending on the permeability of the strata to be depressurised. The combined depressurisation was considered to occur as a temporary 'moving front' of depressurisation ahead of the excavation face, with the wells being redundant once the primary lining have been completed and gained adequate strength.

7.2 Contractor's design

The contractor reviewed the preliminary design and revised the strategy to reduce the potential need for in-tunnel depressurisation measures by maximising the surface well layout. The intention was to reduce water levels to as low as possible from the surface, such that smaller quantities of in-tunnel measures were required. The increase in surface well coverage was possible due to the greater control that the contractor had upon the location and timing of site access constraints. In-tunnel depressurisation mitigation measures were retained as part of the overall design to supplement the surface depressurisation measures. This revised approach was beneficial, as it would reduce programme-critical and expensive delays to tunnel excavation works by reducing the number of in-tunnel wells required.

An ejector spacing of approximately 8 m was implemented based on the pumping test data, the experience of working in these conditions at other sites, the constraints imposed by surface access and the application of engineering judgement. This spacing was approximately half that provided in the preliminary design. Ejectors were located in two separate rings to surround the caverns where possible. The location of the ejectors depended upon the layout of the site and other existing surface constraints. Inclined wells (up to 30° from the vertical) were retained in the layout to achieve the desired coverage outside the boundaries of the site. Figure 4 illustrates the design layout of the scheme.

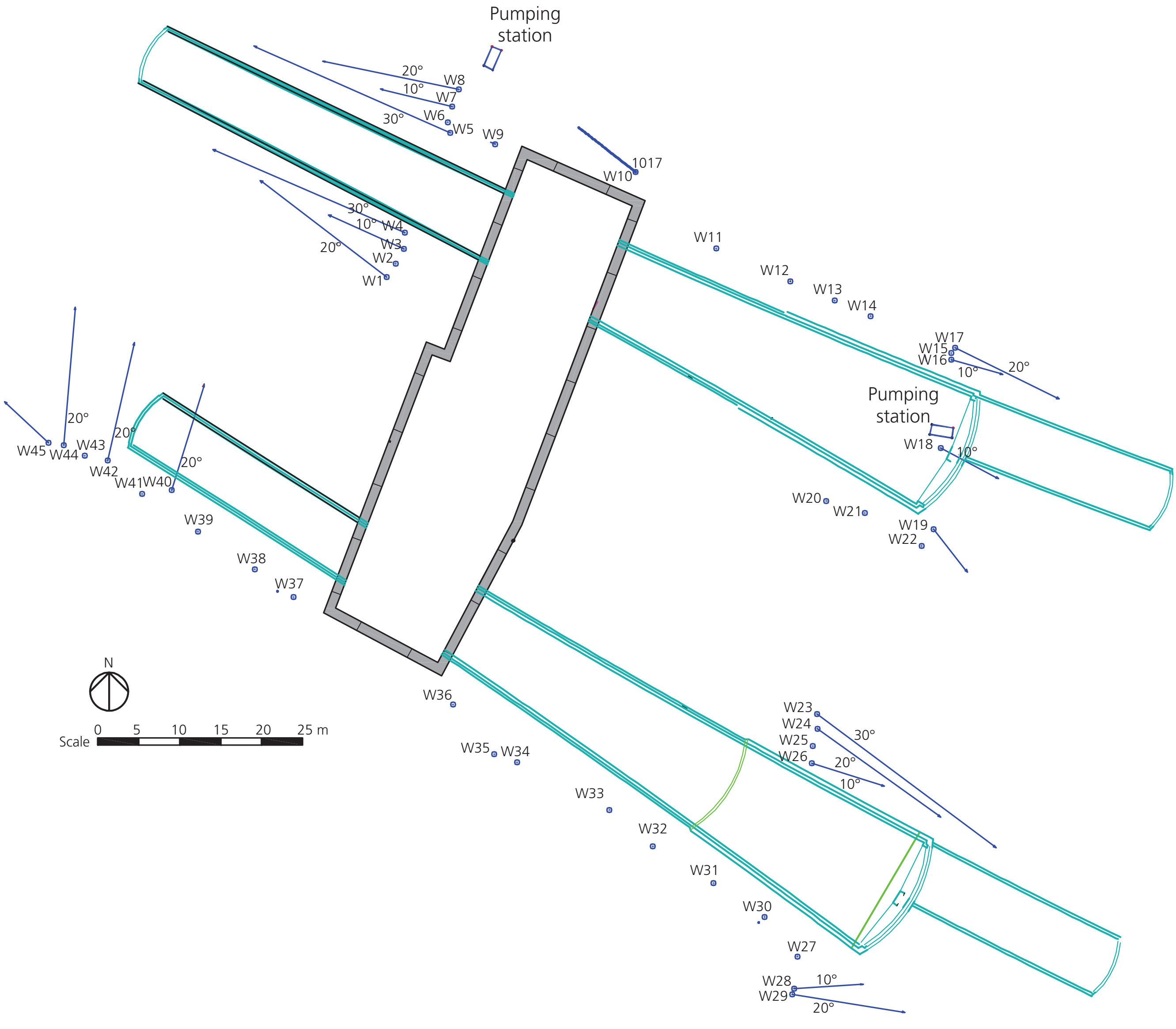

Figure 4. Surface well layout plan

The main features of the implemented depressurisation scheme were

- number of ejectors: 45
- depth: 40 m (110–70 m ATD)
- vertical installations: 27; seven inclined at 10°, eight inclined at 20°, three inclined at 30°
- bore size: 250 mm nominal
- response zone: 70–83 m ATD
- pumping ejectors operated from a surface pumping station.

Following commissioning of the surface ejectors, a drawdown equivalent to a pore pressure reduction of 140 kPa was observed in the piezometers, as shown in Figure 5. The monitoring indicated that

- most of the piezometers in the Upper Lambeth Group achieved an average drawdown of approximately 14 m (i.e. a pore pressure reduction of 140 kPa)
- piezometers in the Upper Lambeth Group located close to the base of the London Clay were not significantly affected by the pumping; the likely cause of that was that those piezometers were installed in the Upper Mottled Beds, a very stiff to hard clay and of low permeability
- there was a certain degree of scatter in the drawdown data, meaning that the effectiveness of the pumping was variable, confirming the variability of the sandy layers within the Lambeth Group.

The average drawdown achieved was considered sufficient for the eastbound cavern to be constructed without subsequent in-tunnel depressurisation works, as that was to be constructed almost entirely in London Clay. However, the drawdown was not sufficient for the deeper westbound cavern, for which groundwater levels remained typically 1 m above the invert level. In order to achieve the desired drawdown in the westbound cavern, additional drawdown from within the tunnel was needed.

Initial trials were undertaken by the contractor using wells drilled from the shaft prior to SCL excavation, and well layout proposals were derived as summarised in Figure 6 to suit the different excavation sequences. As the Lambeth Group was only encountered below the tunnel axis level, no measures were required for the excavation of the initial top heading; additional measures were only required for subsequent bench and invert excavations.

The sequence of the in-tunnel wellpoint installation is shown in Figure 7, with the main features being as follows

- approximately 105 wellpoints were installed in 90 mm auger-drilled holes
- wellpoints consisted of a 38 mm polyvinyl chloride (PVC) pipe with a 1–2 m bonded sand filter screen (54 mm outer diameter)

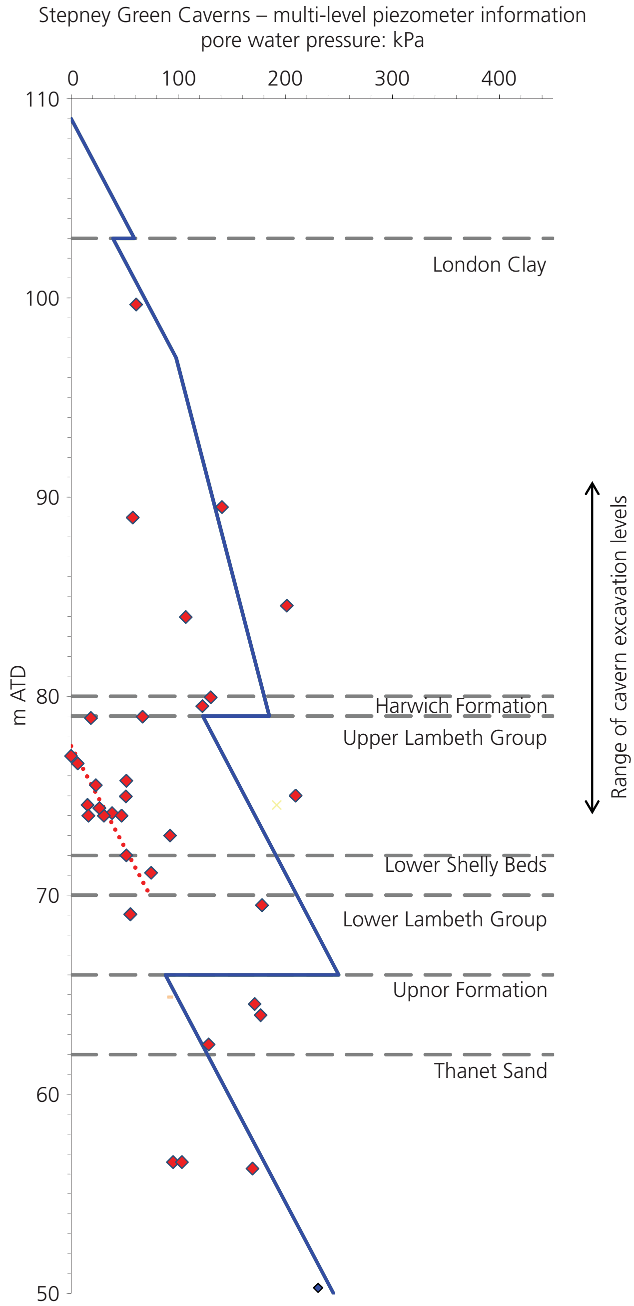

Figure 5. Pore pressure reduction achieved by surface wells

- wellpoints were of variable lengths depending on geology/elevation and angle; they were installed ahead of the face
- 25 mm PVC pipework was used to carry discharge water along the tunnel
- a V-notch tank was used to measure the flow of the system.

The inclusion of the in-tunnel wellpoints resulted in the depressurisation occurring as expected, as a temporary 'moving front' of depressurisation with the tunnel wellpoints being installed as the excavation progressed. Figure 8 shows an example of this

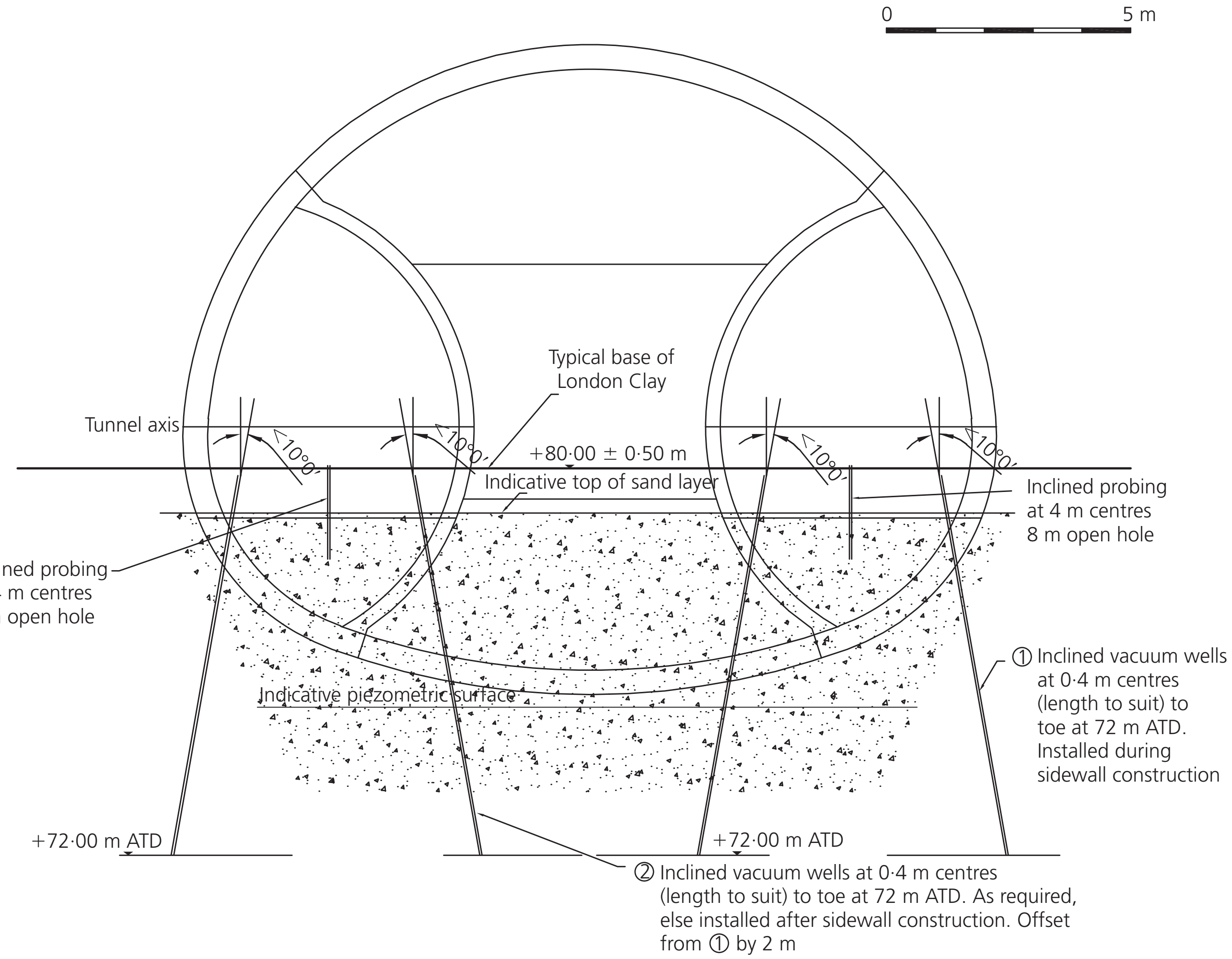

Figure 6. In-tunnel depressurisation mitigation

front of reducing water level as the excavation approaches a piezometer at the end of the westbound cavern.

8. Managing, monitoring and performance during construction

Once installed and operational, a critical aspect of the depressurisation system was the continual monitoring of the piezometric levels, the maintenance of the system and management of the information. Separate strategies were developed for the surface and in-tunnel depressurisation measures.

8.1 Surface depressurisation system

8.1.1 Maintenance

The surface system consisted of 45 ejectors divided in two stations with three pumps in each: two duty pumps and a third one acting as a back-up in case of mechanical failure (Figure 9). In case of a power failure, each station included stand-by generators. Daily maintenance was carried out by a resident operator with checks of the performance (flows, pressures, condition of the pipes and so on). To provide constant support to the 24/7 tunnelling operations, an operator was on call to attend any failure in the system during night shifts.

An automated communication system was set up such that any loss of pressure or power failure would immediately be notified by text message to the dewatering operative and the SCL works ‘person in charge’. Additionally, a sound alarm would alert the rest of the site personnel to the situation. To ensure safety, a ‘safe stop’ condition would be implemented in the SCL works, with removal of personnel, until remedial action had been taken to re-establish acceptable pore pressures.

Periodic maintenance activities included the removal of a deposited iron crust caused by the presence of *Gallionella ferruginea* bacteria, which affected the performance of the ejectors. The gradual reduction in ejector efficiency caused by this material was noticeable from the piezometer records.

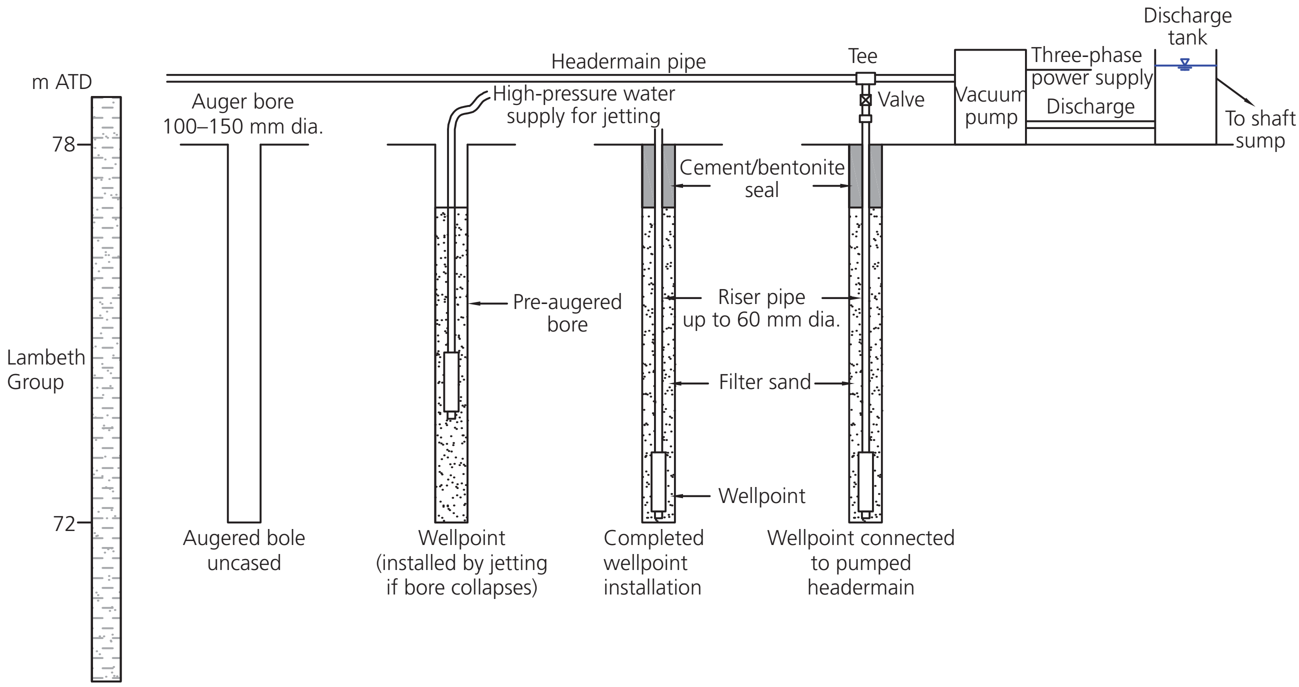

Figure 7. Typical in-tunnel well installation and set-up

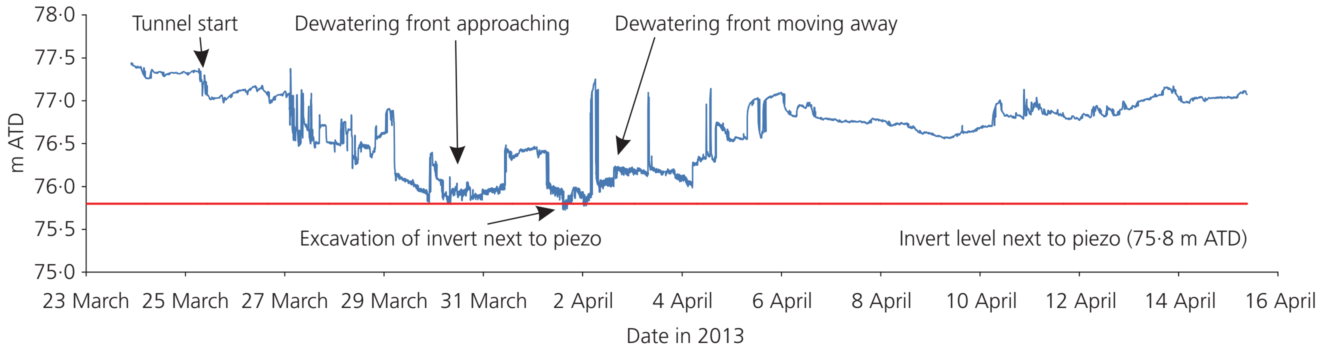

Figure 8. Pore pressure reduction achieved by additional in-tunnel wells

8.1.2 Monitoring and management

For monitoring the drawdown, 12 new piezometers were installed with piezometric level readings taken every 2 min and automatically sent to the real-time monitoring centre (Figure 10), where values were instantly visible. A back-up unit guaranteed that the real-time monitoring centre was always operational. Several gradational trigger levels were set to alert site personnel of unexpected changes of water levels that could affect the safety of SCL activities. If a piezometer reading was above a trigger level, a text was sent to the dewatering operative and the SCL works ‘person in charge’.

Review of the piezometric levels was carried out at the daily shift review group meeting in order to confirm the acceptability of continued excavation procedures.

Surface settlements were monitored on a daily basis. As expected after the pumping test, surface settlements were not large enough to cause any damage to adjacent infrastructure.

8.2 In-tunnel depressurisation system

8.2.1 Maintenance

Similarly to the surface system, daily checks were carried out on the in-tunnel system. However, during excavation, difficulties were encountered in achieving a completely dry excavation, and a series of reviews and upgrades was undertaken to improve efficiency and performance.

8.2.2 First upgrade

As a consequence of the presence of sandy material at the base of the invert together with water levels 500 mm above the excavation

Figure 9. Multiple surface well pump arrangement

Figure 10. Real-time monitoring centre

level, some sloughing was observed during the excavation. Where such sand–clay boundaries were encountered in the face, some residual water was typically observed, even when the depressurisation measures appeared to have worked satisfactorily.

On occasions, contingency measures (sump pumps, geomembrane and so on) were required, which led to delays in construction progress. As the invert level reduced as the westbound cavern excavation progressed away from the access shaft, there was an increasing risk of encountering ‘running sand’ phenomena. Owing to that increasing risk, the in-tunnel depressurisation system was reviewed in order to improve its performance. Several actions were identified and implemented

- lowering of the vacuum pipework to reduce the suction head of the system
- increasing the diameter of the pipework to increase the flow; it was increased from PVC 25 mm to PVC 50 mm
- increasing the length of the porous filter in the well from 1 m to 2 m
- installation of new wells with a layer of gravel to allow some ‘breathing’, which could help clean the screens and reduce clogging of the system
- back-flushing existing wellpoints to improve their operating efficiency.

When the above actions were completed, an immediate effect was observed in the piezometers, resulting, as shown in Figure 11, in an additional drawdown of 500 mm.

8.2.3 Second upgrade

After completion of the first side drift of the westbound cavern, it was considered necessary to upgrade the depressurisation system once again to further improve performance and reduce the groundwater level for the excavation of the second side drift and central pillar. Several actions were implemented as follows.

- New pipework was installed and the pipework was lowered to reduce the suction head of the system.
- The pipework was upgraded from 50 mm PVC pipe to 100 mm steel pipe.
- New Geho piston pumps were installed to replace the low-flow DM-Vex vacuum pumps.
- The well filter specification in fine soils is always a compromise between meeting the conflicting requirement to keep out fines and let in water; as the amount of fines was not significant, more permeable filters were specified for the wells. Figure 12 shows the revised wellpoint construction.

These changes were initially tested in three trial wells; the improved drawdown is shown in Figure 13. The changes ensured the remaining inverts of the westbound cavern were completed safely and without further requirements to reduce groundwater.

8.2.4 Monitoring and management

The performance of the tunnel wellpoints was monitored by using un-pumped wellpoints as equivalent standpipe piezometers. This allowed the monitoring location and pumped wellpoints to be altered to suit the progress of the works. In addition, where

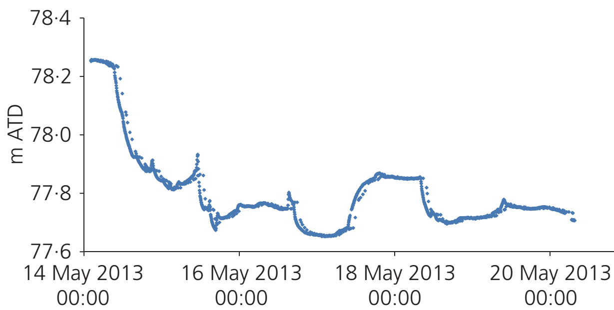

Figure 11. Pore pressure reduction achieved by additional in-tunnel measures (first upgrade)

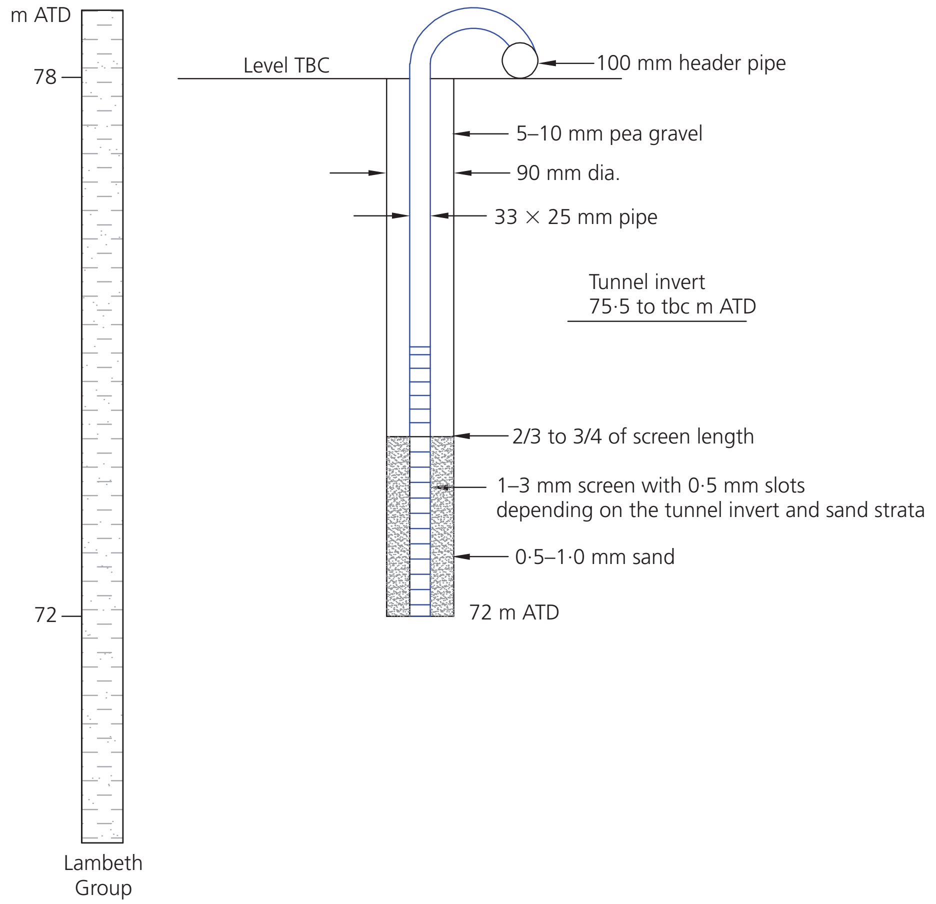

Figure 12. Revised wellpoint arrangement

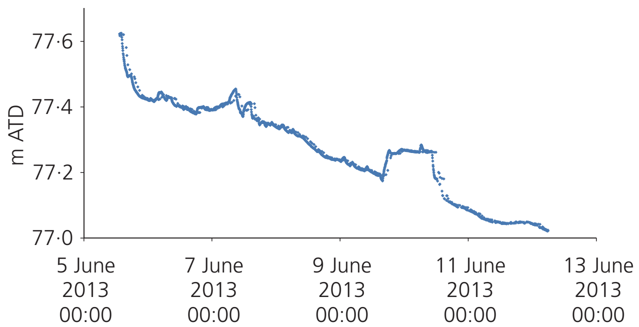

Figure 13. Pore pressure reduction achieved by additional in-tunnel measures (second upgrade)

required, dedicated piezometers were installed to 1 m below the invert. Real-time information from the piezometers was used to check trends and detect problems in any of the pumps. The invert excavation was the most critical part of the tunnel in terms of water-bearing materials, so a hold point was implemented within the excavation procedures, effectively acting as a ‘permit to dig’. The hold point required checking of water levels in the closest standpipe prior to the excavation, with the potential for revision of the depressurisation system and adoption of additional mitigation measures.

It was anticipated that, given the variability of the permeability and location of the sand channels, the design would have to be locally altered. Figure 14 illustrates an example where several contingency measures were implemented to aid in achieving a dry excavation. These included sand bags, Hy-rib, geotextile and dry-mix shotcrete. The need for such measures was infrequent and eliminated by the upgraded maintenance measures described above, which improved both safety and efficiency.

9. Conclusion

The ground investigations, pumping test, designer’s outline depressurisation design and final contractor’s depressurisation design allowed the safe SCL construction of Stepney Green caverns in the Lambeth Group. An accurate ground model of the Stepney Green site was essential in understanding where to target investigations and to understand the variability of water-bearing sandy materials in the Lambeth Group within which the deeper westbound cavern would be excavated. Close supervision and accurate measurement of pore pressures, combined with use of

Figure 14. Additional localised measures

multi-port piezometers, ensured the groundwater data were reliable to inform the depressurisation plan.

Connectivity between Lambeth Group sand channels was demonstrated by both the pre-construction pumping test and by the use of surface ejectors and in-tunnel wellpoints. This was a key factor in being able to successfully use the contractor's strategy of increased surface wells, demonstrating the benefit of the ground investigation work undertaken.

Key lessons learned included the following points.

- Multi-port piezometers can be installed successfully, allowing a reduction in the overall number of boreholes in a ground investigation while providing reliable data for design purposes. The key to achieving this was sufficient space at the surface to prepare the tubing and care during installation.
- Interconnectivity can exist between Lambeth Group sand channels, even if they are small and apparently separated by soils with very low permeability.
- Adjustments and improvements to in-tunnel wellpoints, such as increasing discharge pipe diameters, can rapidly improve drawdown.
- Regular cleaning of ejectors rapidly improves drawdown.
- A certain residual amount of groundwater in Lambeth Group sandy materials can be expected, even when depressurisation appears to be successful.
- The drawdown response varied in relation to sand channel permeability, interconnectivity and morphology.
- The construction vindicated the requirement for a multi-level depressurisation approach using both surface and in-tunnel wells.

The use of automated instrumentation and communications in a real-time context was successful in providing a robust safe action plan to forewarn of potential problems with the system.

The overall success was in part a result of the collaboration shown between the various parties in sharing knowledge of the original design intent, the perception of the risks and the practical limits of implementing the systems. The knowledge sharing was aided by the ability to ensure continuity of personnel, with staff involved in the original ground investigation and pumping test being involved on site throughout construction. The significant improvements in depressurisation achieved with relatively minor changes to the system set-up also demonstrated the benefit of using experienced sub-contractors familiar with local conditions. The efficiency of the operation was such that the SCL advance rates for the different sided drifts and central core improved from the programmed 0·75 m/d to 1·5 m/d.

Acknowledgements

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4.1.3 Desarrollo de un modelo del terreno, investigación geológica y mitigación de riesgos para la excavación con frente abierto de una galería en el proyecto subterráneo de la línea Elizabeth.

(Título original en inglés: Development of a ground model, targeted ground investigation and risk mitigation for the excavation of an open face cross passage on the underground Elizabeth Line, London)

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Development of a ground model, targeted ground investigation and risk mitigation for the excavation of an open face cross passage on the underground Elizabeth Line, London

Emilio Linde-Arias[a,*], Mark Lemmon[b], Juan Ares[c]

[a] OTB Engineering, 46 Loman St, London SE1 0EH, UK
[b] Crossrail Limited, 25 Canada Square, London E14 5LQ, UK
[c] Dragados UK, Regina House, 2nd Floor 1-5 Queen St, London EC4N 1SW, UK

ABSTRACT

The new London Crossrail running tunnels for the future London Rail Elizabeth Line were driven by Tunnel Boring Machine (TBM) and linked at regular intervals by cross passages, constructed with open face techniques. This paper describes the development of the ground model and depressurisation required for a cross passage link between the running tunnels in East London, through the analysis of the existing data and a subsequent targeted ground investigation. The cross passage was 14 m long and included a central sump with a formation level at 48 m below ground surface. Cross passages encompass two risks: Programme risk as they run along the critical path of the project and construction risk, since they are excavated with traditional open face techniques. For cross passage CP6, a pre-construction borehole indicated that the ground conditions at the cross passage horizon comprised predominantly cohesive Lambeth Group soils. The well-known variability of the Lambeth Group, often with sudden lateral and vertical change in facies, make the prediction of ground conditions extremely challenging and critical for safe open face tunnelling. The ground model evolved from the design to the construction stage, using the information provided by: the project ground investigation; TBM drive data and sampling; followed by a targeted probe drilling investigation conducted from within the running tunnels. During the latter investigation, more extensive water bearing sands were identified, than previously identified by the surface borehole, which required a revision to the ground model and subsequently a change in approach to the cross passage excavation. The new ground model allowed the Contractor to manage the ground risk appropriately, which included a change in approach to groundwater control in the form of in-tunnel well-points depressurisation and surface ejector wells.

1. Introduction

Crossrail is a railway project consisting of new tunnels, stations and associated works that will form the future, east-west running, London Rail Elizabeth Line. The Crossrail Eastern Running Tunnels contract (C305) comprised the construction of 11.9 km of twin bored TBM tunnels and associated works, between Victoria Dock and Farringdon, including ten cross passages. Cross passages are structures that connect the twin railway tunnels, at a spacing of 600–800 m, to facilitate escape for passengers and staff from one bore to the other, in the event of an emergency, in addition to access for maintenance and in some cases the location of tunnel drainage sumps. At the locations of the cross passages the TBM installed special segments (opening sets) that provided temporary support to the openings required for the excavation of the cross passages. Cross passage CP6 is located between Whitechapel and Liverpool Street, in London (Fig. 1).

CP6 is a 4.3 m diameter hand mined tunnel permanently supported with a Spheroidal Graphite Iron (SGI) lining (Fig. 2) and includes a central sump point for the tunnel drainage (Fig. 3). The design of the permanent support was the responsibility of contract C122 (Arup and Atkins). The C305 Contractor (Dragados Sisk Joint Venture) was responsible for the design of the temporary structures and any ground improvement requirements for the safe excavation of the cross passage, which was undertaken by hand mining with timber headings to support the ground prior to the installation of the SGI rings. The usual progress rate for the excavation in the contract was approximately 1 ring per day (0. 6 m/day).

The construction of CP6, carried out between April and June 2015 required completion of the TBM bored tunnels, in the cross passage location, and installation of the temporary support structure in the main tunnel bores.

Ground level at the CP6 location is approximately 114 m ATD (Above Tunnel Datum = Ordnance datum + 100) with the cross passage crown and invert at 71.2 m ATD and 75.5 m ATD, respectively (Fig. 3). The formation level of the sump is at 65.9 m ATD (48 m depth).

* Corresponding author at: Arcadis, Arcadis House, 34 York Way, London N1 9AB, UK.
E-mail address: emilio.lindearias@arcadis.com (E. Linde-Arias).

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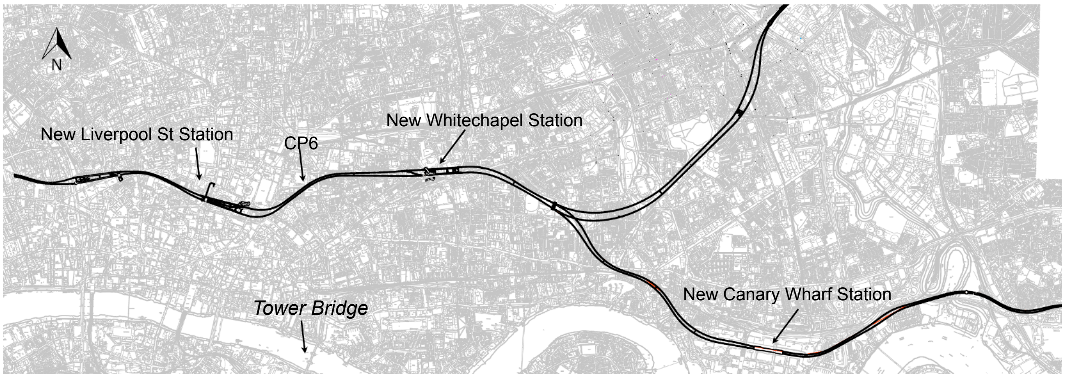

Fig. 1. Underground route of the Elizabeth Line and location of CP6.

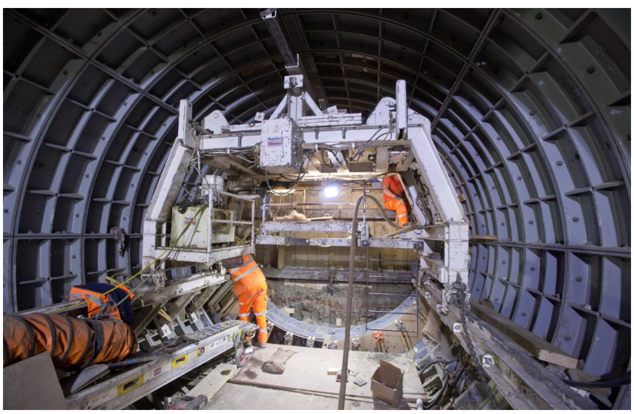

Fig. 2. Excavation of cross passage CP6 using timber headings and installation of SGI lining.

2. Geological background

2.1. Stratigraphy

The London 'Basin', as it has been traditionally named since the 19th century (Royse et al., 2012), is not a sedimentary basin but is a broad north-east to south-west trending syncline stretching from the Chalk crests of the Chilterns in the north-west to the North Downs in the south. The syncline developed from tectonic stresses that arose during the formation of the belt of mountain chains during the Alpine orogeny in the late Cretaceous, and later, during the late Oligocene to mid-Miocene (Ellison et al., 2004). The axis of the syncline is broadly coincident with the line of the River Thames in central London. The regional geological succession in the London Basin is outlined in Table 1. The floor of the basin comprises Cretaceous Chalk, overlain successively by Palaeogene deposits: Thanet Sand Formation, Lambeth Group, Harwich Formation and London Clay Formation.

The Crossrail tunnels at the CP6 location are excavated through the London Clay Formation and the Lambeth Group.

The London Clay Formation is a relatively homogeneous, stiff to very stiff dark grey/blue clay, with a varying silt content. The London Clay has been divided into successive 'Divisions' A1, A2, A3, B, C, D and E, with the divisions representing a coarsening upwards sequence (King, 1981). This formation was deposited in a deep marine environment (Ellison et al., 2004).

The Lambeth Group comprises three distinct units: the Woolwich, Reading, and Upnor Formations. The variation in the characteristics of the Lambeth Group lithologies resulted from their varied depositional environments and subsequent post-depositional changes. The Upnor and Woolwich Formations were deposited in shallow marine or lagoonal environment whereas the Reading Formation was deposited in fluvial to tidal environments (Knox, 1996). The different cycles of regression and transgression and the presence of river channels leading to the presence of sand-filled channel structures, generates a complexity of facies and consequently a great vertical and lateral variability that is challenging for engineering projects in these materials (Entwisle et al., 2013). Fig. 4 shows a summary of the most common Lambeth Group facies and their spatial relationship.

2.2. Hydrogeology

The London Basin sequence comprises two aquifers: the Lower Aquifer in the Chalk, Thanet Sand and the lower sand deposits of the Lambeth Group; and the Upper Aquifer within the Quaternary deposits. The two aquifers are divided by the low permeability London Clay and the clay members of the Lambeth Group that behave as aquitards (Preene and Roberts, 2002). Fig. 5 shows the pore water pressure profile present in central London, which is characterised by under-drained conditions in the London Clay Formation. The under-drained profile is a result of groundwater extraction from the Lower Aquifer during the 19th and first half of the 20th century, to supply water to the population of London and the requirement of industrial activities. Since the 1960́s water levels in the Lower Aquifer, in London, had been gradually recovering (Simpson et al., 1989). In 1999 an abstraction programme was developed by the General Aquifer Research Development and Investigation Team (GARDIT) to bring the groundwater levels under control (Jones, 2007).

As explained above, the variability of the depositional environment of the Lambeth Group produces the presence of sporadic lenses or layers of high permeability soil (sand or gravel) that can be charged with pressurised water. This was identified as a primary risk for the construction of the cross passage, since it had the potential to inundate the excavation without control measures. Possible mitigation techniques identified included: reduction of the water pressure with in-tunnel or surface depressurisation, as was employed in the Crossrail Stepney Green Cavern, excavated in the same horizons (Linde-Arias et al., 2015) and the new Farringdon station (Gakis et al., 2016) or stabilisation of the soil with ground improvement techniques such as grouting or ground freezing.

2.3. Faults

Traditionally in London, the presence of faulting has tended to be underrepresented due to the lack of outcrops and the perception that the London Basin is an area of very long-term tectonic stability (Aldiss, 2013). The evidence of extensive faulting was presented by De Freitas (2009) and during the last decade numerous examples of faults have been identified in underground infrastructure projects, constructed in London (Linde-Arias et al., 2018) and (Skipper et al., 2008). Some of

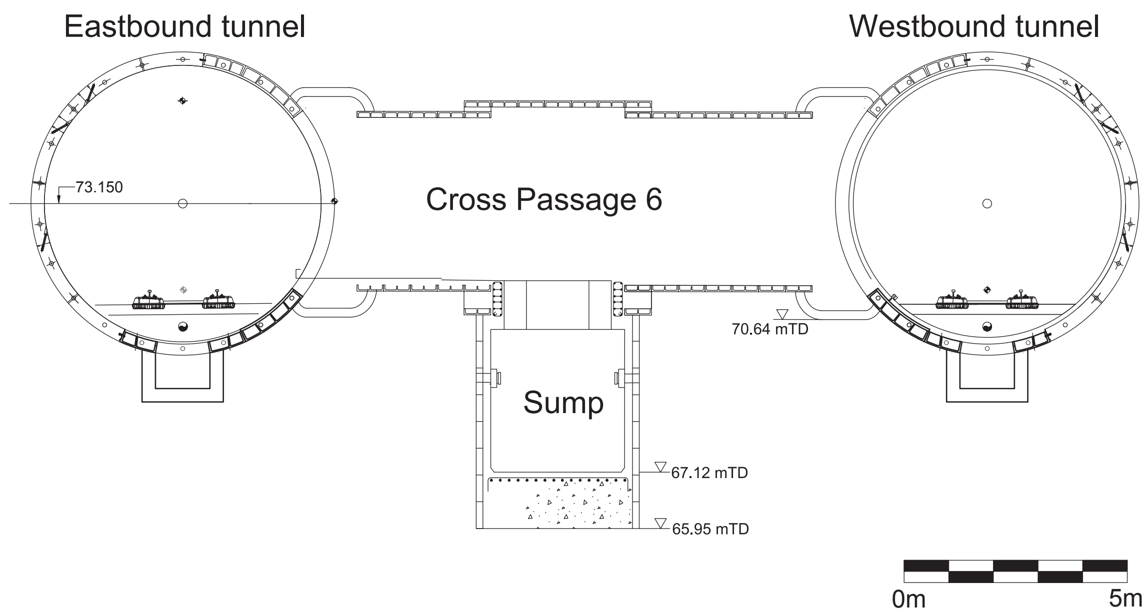

Fig. 3. Elevation of cross passage CP6 and the sump.

Table 1
London basin stratigraphy.

Period	Series	Deposits
Quaternary	Holocene Pleistocene	Made ground Alluvium Langley silt River terrace deposits
Palaeogene	Eocene Palaeocene	London clay formation Harwich formation Lambeth group Thanet sand formation
Cretaceous	Upper Cretaceous	Chalk

this faulting could be reactivations of pre-existing structures of the basement (Ghail et al., 2015).

The presence of faults is a risk to underground construction due to abrupt changes in ground conditions. Consideration of this risk is especially important for excavation in the Lambeth Group, as the transition between clays and sands can be sudden and unexpected (Gakis et al., 2016).

3. Ground investigation during design phase

3.1. Borehole information

During the Crossrail design phase, two cable percussion boreholes were drilled at the CP6 cross passage location (see Fig. 6):

- Borehole RT52 was drilled to 32 m below ground level (bgl) in 1992, as part of the early design.
- Borehole LW9 was drilled to 55 m bgl in 2003, during the subsequent design phase. The base of borehole LW9 was approximately 5 m beneath the formation level of the CP6 sump, the lowest point in the structure.

Table 2 summarises the depths and the different geological horizons encountered in those investigations and Fig. 6 presents the borehole logs in a profile with the CP6 structure.

Borehole LW9 was drilled approximately 11 m from the CP6 cross passage location and it was the only borehole to extend to a depth below the CP6 structures. Due to access constrains it was not possible to carry out additional ground investigation during later design stages.

Interpretation of the LW9 borehole log suggested that: the cross passage would be constructed in the red clay of the Upper Mottled Beds member of the Lambeth Group; and the sump would be excavated in the Laminated Beds (sand and clay) and Lower Mottled Beds (gravel) of the Lambeth Group. The Mid-Lambeth Hiatus (erosion boundary commonly identified by a precipitate – hard layer) would be roughly 1 m below the tunnel invert, with the sump extending further below. Borehole LW9 log indicated that the Lambeth Group, above the Mid Lambeth Hiatus, was mainly "*cohesive*", with only a layer of "*gravel with some matrix of sandy clay"* identified between 67. 2 m and 63. 3 m ATD.

The Harwich Formation was not encountered in any of the boreholes.

3.2. Hydrogeology

In order to investigate the piezometric level at the CP6 cross passage location, several standpipe piezometers were installed within the design phase boreholes, as shown in Table 3.

Table 3 shows that the piezometric level in the Upper Mottled Beds is lower than the levels in London Clay, which is consistent with the under-drained pore water profile discussed above.

3.3. Initial dewatering strategy

The Ground Model, based predominantly on borehole LW9, led the Designer to propose groundwater control measures only for the sump excavation, the cross passage requiring nothing as it was wholly in a clay stratum. The Designer indicated use of vacuum well points in the Upnor Formation, as an option, to reduce the water pressure in the Lower Aquifer to facilitate the excavation of the sump as shown in Fig. 7.

As mentioned above, the main contractor, C305, was responsible for the design of temporary works and therefore validating the option, which was prepared for tendering purposes.

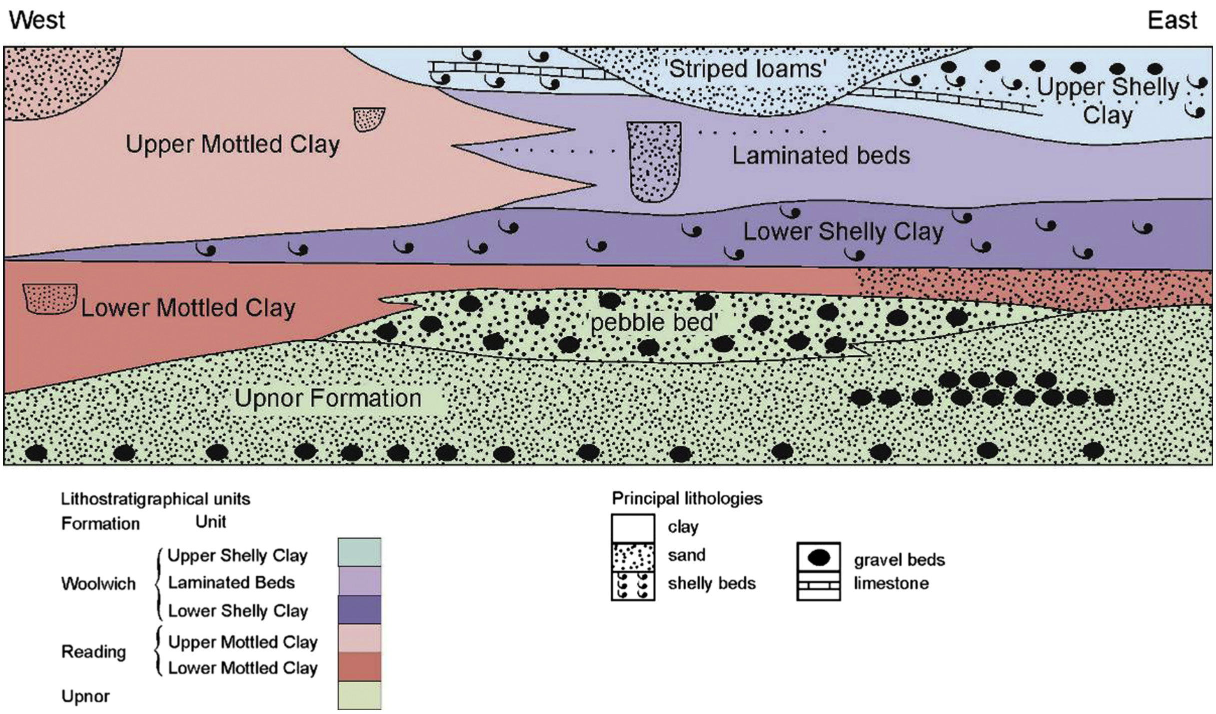

Fig. 4. Lambeth group facies and spatial relationship (Entwisle et al., 2013).

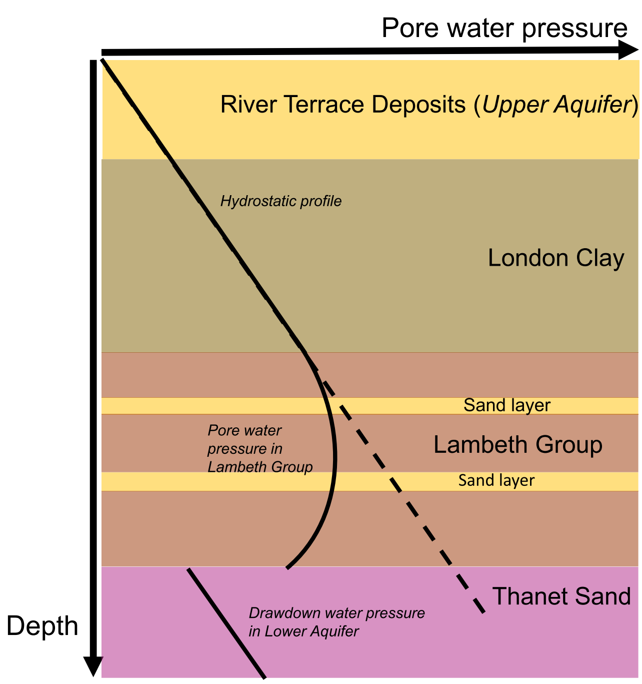

Fig. 5. Piezometric profile in central London (modified from Preene & Roberts, 2002).

4. Ground investigation during construction and development of a new ground model

4.1. Investigation during excavation of the running tunnels

During the excavation of the running tunnels with the TBMs, in the vicinity of CP6, the Contractor decided to pay particular attention to the ground conditions, with the aim of either verifying the existing borehole information or to identify variation in the Lambeth Group beds, particularly to detect the presence of water bearing sand layers.

At the CP6 location, several options were considered to investigate further the ground. The first option considered was the direct inspection of the excavated TBM face by a Geologist, however, this was ruled out as: exposing a person to hypobaric conditions was considered an unnecessary safety risk; the impact on the critical path for tunnelling was unacceptable; and there was a risk of causing excessive settlement on the surface.

A second option of investigation was proposed that avoided the impacts of a TBM face inspection, which involved direct sampling of the TBM spoil from the conveyor, moving soil away from the excavated face. This material was then testing by grading and compared with the results of similar tests, obtained from samples in other areas of the route, with known geology. This method offered a direct check on the ground conditions in the vicinity of the cross passage, however, the information obtained had limitations, as the samples collected represent an average of all the materials excavated and encountered at the 8 m diameter TBM face. While in homogenous excavation conditions, such as a full face of London Clay, the results would be representative of the ground at that location, in the instance of mixed face conditions, such as that found when tunnelling in Lambeth Group, with interbedded layers, the results do not provide the detail required to inform an interpretation for a design ground profile.

Fig. 8 shows the grading curves obtained from test results on samples from the TBM conveyor for the Eastbound and the Westbound tunnels. The curves for both tunnels showed relatively high proportions (20–40%) of sand but within the typical boundaries for the Upper Mottled Beds member in the section of the project within Liverpool Street and Isle of Dogs. However, the Westbound tunnel samples show a material with slightly more coarse particles, and consistently in the lower bound of the envelopes. Also, the shape of the curves indicated the possibility of the samples being the result of the mixture of materials from the Upper Mottled Beds, clay, and a sand channel. Fig. 8 also includes, for comparison, the boundaries of the samples tested for grading in sand channels encountered in the boreholes. This led the Contractor to conclude that the risk of encountering granular component of the Lambeth Group in CP6 was high enough to undertake further investigation.

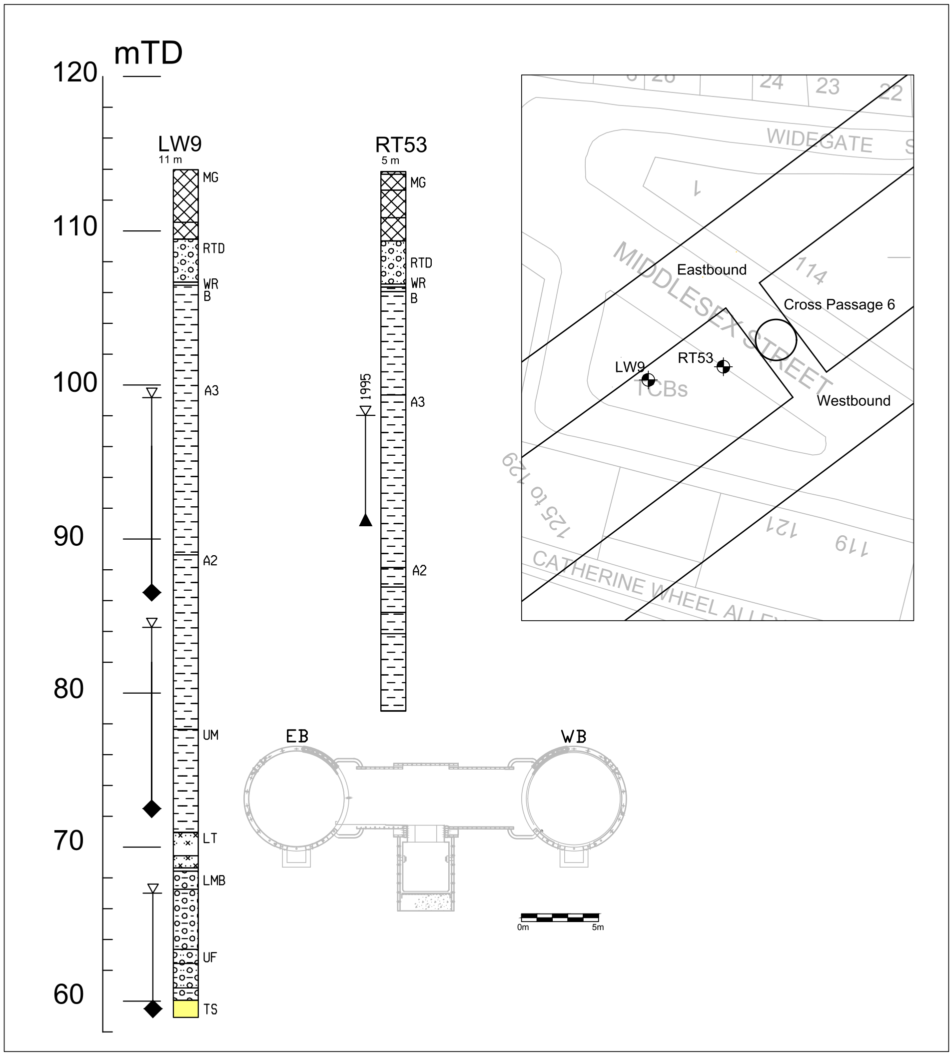

Fig. 6. Elevation of CP and boreholes logs.

Table 2
Depth of geological units encountered in the boreholes.

Unit	LW9		RT53	
	Top level (m ATD)	Top level (m bgl)	Top level (m ATD)	Top level (m bgl)
Made ground	113.8	0.0	113.9	0. 0
River terrace deposits	109.3	4.5	109.4	4.5
London clay	106.5	7.3	106.6	7.3
Lambeth group	77.6	36.3	N/A	
Thanet sand	60.0	53.9		

Table 3
Piezometers installed in boreholes LW9 and RT52.

Borehole	Tip (mATD)	Filter (mATD)	Stratum	Piezometric level (mATD)
LW9	85.9	86.7 to 85.7	London clay	99.1
	71.9	72.7 to 71.7	Upper mottled beds	84.5
	59.5	58.0 to 60.0	Thanet sand	67.0
RT53	89.6	90.6 to 91.6	London clay	98.5

4.2. In-tunnel investigation by proof drilling through segmental lining

Upon completion of the TBM tunnel rings, in the Eastbound tunnel, a series of short proof drilling holes were conducted, with the aim of sampling the ground behind the lining at the CP6 location to identify the boundaries between the different units and detect the presence of possible water bearing sand layers. The holes were approximately 1 m in length, perpendicular to the running tunnel segments and drilled through the grouting ports employing a handheld 32 mm drill, as illustrated in Fig. 9.

The drilling permitted direct sampling of the soil behind the tunnel lining, with the material recovered along the drill bit providing a small, but representative sample of the ground. The holes were drilled through a ball valve with the same thread design as the grout socket in the segment (Fig. 9). Upon encountering water in any drill hole (see Fig. 10), the operative immediately shut off the valve, to prevent excessive inflow of water to the tunnel and to avoid the risk of washing in fines from the surrounding material.

For CP6 it was decided to drill in as many ports as possible at the tunnel: invert, shoulder and crown elevations. Drilling was undertaken in ports at the cross passage location and in the five running tunnel rings either side of the cross passage (approximately 6 m in either direction). The number of probe holes was limited by the access to some of the running tunnel grout ports, due the presence of equipment in the tunnel (e. g. ventilation ducts, cabling and pedestrian walkway),

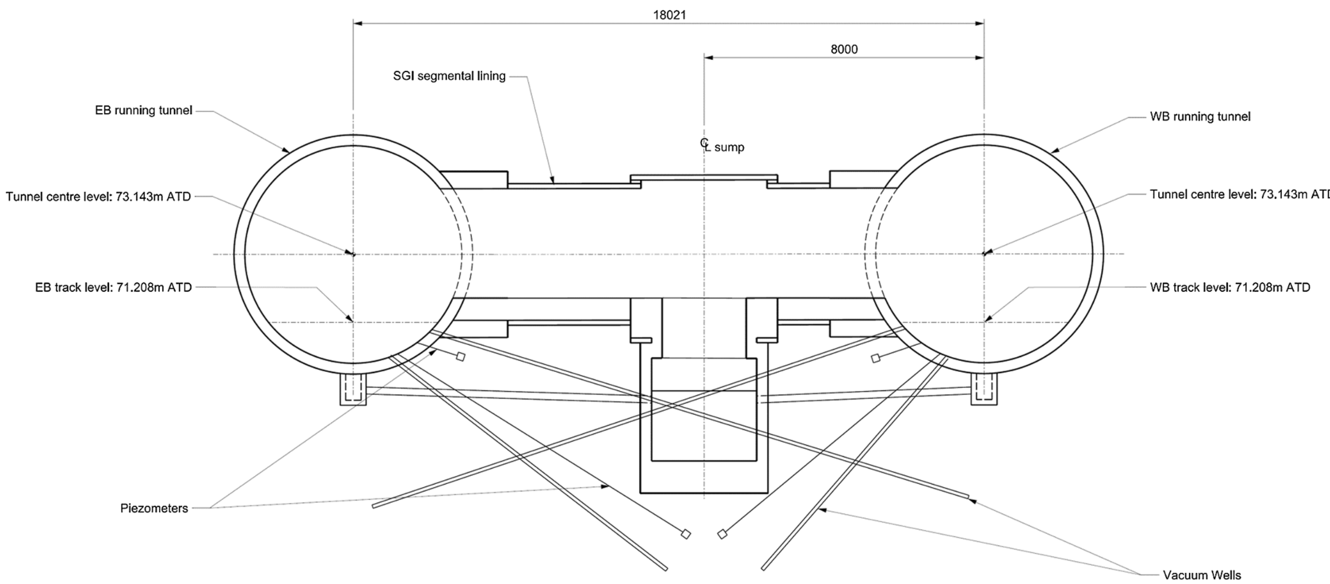

Fig. 7. Designer's indicative depressurisation strategy provided at tendering stage and based on borehole LW9.

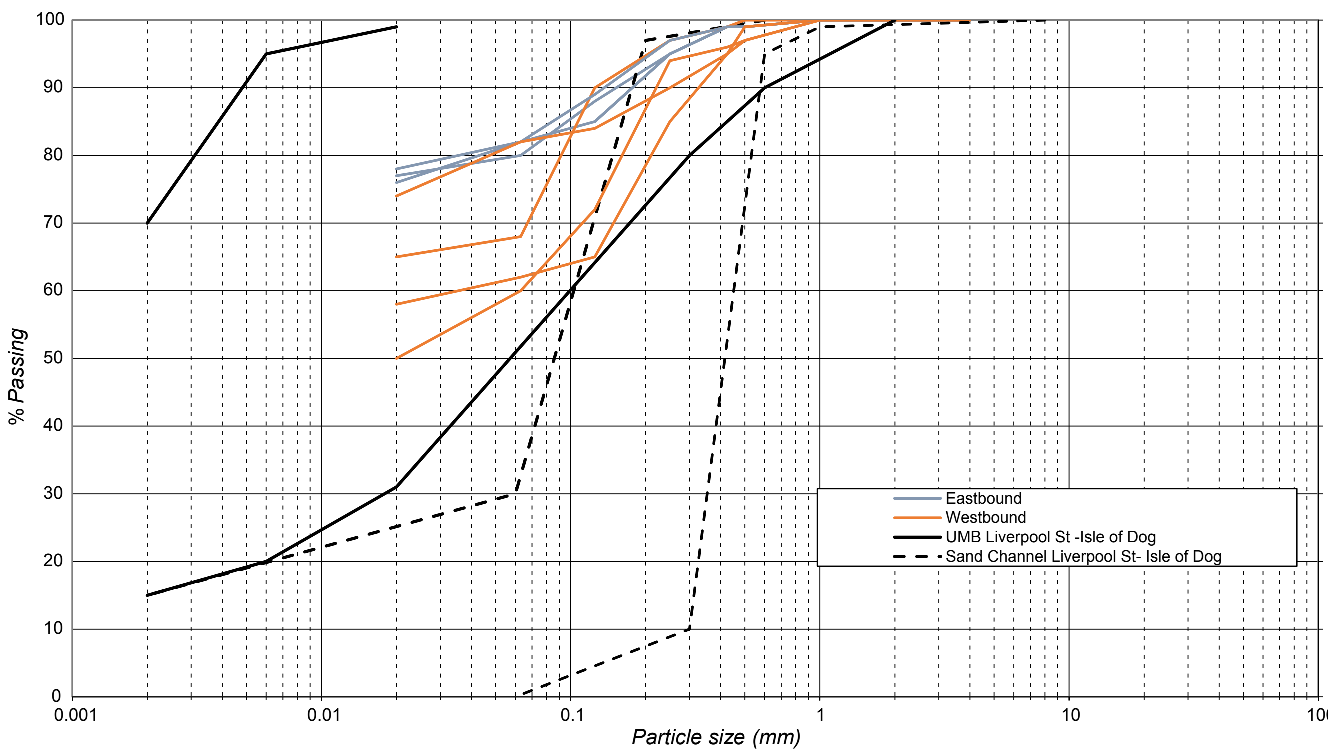

Fig. 8. Grading curves for samples taken from TBM conveyor for the Westbound (orange) and Eastbound (blue) tunnels, at the cross passage (CP6) location. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

however despite the constraints the Contractor was able to drill 11 probe holes in the Westbound running tunnel and 12 probes in the Eastbound tunnel, as shown in Fig. 10.

The drilling probed in to the ground until the recovered material changed, at which point the drilling depth was recorded so that the location of the change in material could be plotted on a cross section to allow interpretation.

The samples taken from the drilling investigation were logged and the findings allowed the Contractor to identify the stratigraphic sequence in the area surrounding the running tunnels, specifically the boundary between the London Clay formation and the Upper Mottled Beds. Fig. 11 shows two samples of the material obtained.

The investigation confirmed the presence of three units: London Clay, Upper Mottled Beds and Laminated Beds. The boundary between the Upper Mottled Beds and the Laminated Beds was encountered within the profile of the cross passage, but higher than inferred by the borehole LW9. In addition, the investigation uncovered a previously unidentified sand channel beyond the Eastbound running tunnel at crown level of the cross passage. In the probe holes drilled from the Westbound running tunnel this sandy unit occupied almost all of the upper half of the cross passage. Also, the greater presence of sand in the Westbound tunnel was consistent with the results of the grading tests from the TBM spoil.

These findings had a significant impact on the interpreted ground

Fig. 9. Proof drilling through the grouting port of the TMB tunnel segments with handheld equipment.

profile and required a different groundwater control strategy, including the requirement for additional groundwater information.

4.3. Groundwater investigation and monitoring

During the probe hole investigation, in both the Eastbound and the Westbound running tunnels, upon penetrating sand in the Upper Mottled Beds and in the Laminated Beds, pressurised groundwater was encountered, as demonstrated in Fig. 12.

In order to measure the groundwater pressure, a pressure gauge, with a range of 0–1 bar (100 kPa), was installed in a selection of the probe holes, as shown in Fig. 13. The measurements allowed the Contractor to assess the groundwater pressure and therefore the water levels, for inclusion in the ground model. The pressures recorded in the sand channel, in the Upper Mottled Beds, represented a piezometric level of 84.8mATD, similar to the level recorded in the piezometer installed at 72mATD in borehole LW9 (84.5mATD).

Due to the significance of the groundwater findings, it was decided to conduct falling head tests in the standpipe piezometer installed in the borehole (LW9) at 71.9mATD, to assess the permeability of the material. The results are presented in the Fig. 14 below using the Hvorslev method (Cashman and Preene, 2001).

The estimated permeability was 2.1×10^{-5} m/s, typical of a silt sand (Lewis et al., 2006). Permeability tests in boreholes only involve a relatively small volume of soil around the test section. If the soil is heterogeneous or has significant fabric, such tests may not be representative of the mass permeability of the soil. However, the result indicates the presence of high permeability material over the length of the standpipe piezometer filter. In earlier investigations, these strata had been logged as very stiff clay Upper Mottled Beds, however, the in-tunnel investigation and piezometer test show that the revised interpretation of silt sand to be correct, indicating possible sample recovery issues during the cable percussion drilling of borehole LW9.

4.4. The consolidated ground and groundwater model

After the completion of the in-tunnel probe drilling and groundwater investigation, it was possible to reassess the ground conditions for the cross passage CP6. The in-tunnel investigation provided much more focused and localised information directly in the vicinity of CP6. The new ground model is presented in Fig. 15, with three main differences to the original model that was developed with borehole LW9 as the main source of information:

- The presence of a water bearing sand channel that runs in a northeast-southwest direction, that would be encountered gradually as the excavation commenced from the Eastbound tunnel towards the Westbound tunnel as shown in Fig. 16.
- The top of the Laminated Beds was encountered higher than expected and consequently within the cross passage excavation profile, which had not been expected previously.
- The Upnor Formation was re-interpreted as a cemented conglomerate, based on the material being logged as a very dense gravel and the borehole recording chiselling at this level. The piezometric level in the lower aquifer was therefore less of a concern given this more competent material and the measured water levels not being significantly higher than the sump invert level.

The significance of the findings in relation to excavation of an open face tunnel is clear and the decision to undertake further ground and groundwater investigation, well justified. The new ground model led to a new approach to the control of groundwater for the cross passage construction, providing a safer working environment for the miners and prevented a delay to the programme, which would have arisen should the water bearing strata been uncovered during the works and contingency measures not been in place to address these changes.

Furthermore, all this information was gathered immediately after the installation of the opening sets segments at the cross passage location without causing any disruption to the normal progress of the TBMs. The use of a standard drilling rig from within the tunnel would have required the suspension of the works or, alternatively, delay the compilation of the data until the TBMs were in transit in Liverpool Street Station box.

5. New depressurisation strategy

The primary aim of the new groundwater control strategy was to provide a safe working environment for the miners, excavating the open face cross passage tunnel, and to avoid ingress of water and fines. The approach taken to control the ground water in the sand channel and the Laminated Beds (Fig. 14) was depressurisation, which would reduce the pressure of the water in these beds using in-tunnel vacuum wells. No other method was considered as depressurisation of the Lambeth Group had been successfully carried out in other parts of the project (Linde-Arias et al., 2015).

In-tunnel well points and piezometers were installed by a track mounted rotary drilling rig from within the Westbound and Eastbound running tunnels, as shown in Fig. 17. The data logger linked piezometers were installed in the sand channel, Laminated Beds and the Upnor Formation to provide in-tunnel automatic and real-time monitoring during pumping and to form part of an emergency alarm system to protect the workforce. The in-tunnel installations consisted of three types:

- Upward, well points between 3 and 12 m long targeting the sand channel, 1–2 m above the running tunnel crown, for the cross passage excavation.
- Downward, well points between 2 and 10 m long targeting the Laminated Beds for both the cross passage excavation and the sump.
- Downward probe hole, 12 m long, to confirm the presence of gravel in the Upnor Formation and that the water level in the Upnor Formation Lower Aquifer was below the formation level of the sump. These holes could have been switched to ejectors wells for the excavation of the last ring of the sump, if required. The dewatering subcontractor, WJ Groundwater advised against the use of well point in this case due to the difference of elevation head between the location of the pump and the formation level of the sump, approximately 5 m.

Each in-tunnel well-point bore was installed through threaded inserts in the running tunnel segments and further control measures comprising: lost bit drilling, stuffing boxes and drilling through valves,

Fig. 10. Probe drilling investigation layout in the Eastbound and Westbound tunnels and summary of soil sample findings.

were used to avoid risk of excessive ground loss during installation. Due to the high groundwater pressures in drilling long well-points there is a risk of piping of the sands. In order to mitigate this, initially shorter wells, separated from the cross passage, were installed to provide some initial depressurisation.

The 55 no. well-points, installed at 5 sections as exemplified in Fig. 18, were each immediately connected to the vacuum pumps, upon installation, as the depressurisation benefits subsequent well-point drilling and installation. Pumping with the vacuum system was maintained for the duration of the works including: the cross passage, catch-pit and sump construction.

Due to the dimensions of the rig required for the drilling of the well points (Fig. 17) the works had to be carried out during a period when the TBM was not excavating as during its operation the main tunnel is obstructed with the transport of material (segments, equipment etc.). In this case, the drilling took place during the transit of the TBM through Liverpool Street Station box.

Fig. 19, summarises the changes in the pore water pressure profile before commencing the excavation and the after the depressurisation.

The main objective of the depressurisation was to drain the sand

Fig. 11. Sample of the red clay in the Upper Mottled Beds, on the left, and the London Clay, on the right. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

Fig. 12. Pressurised water coming out of one of the probe holes.

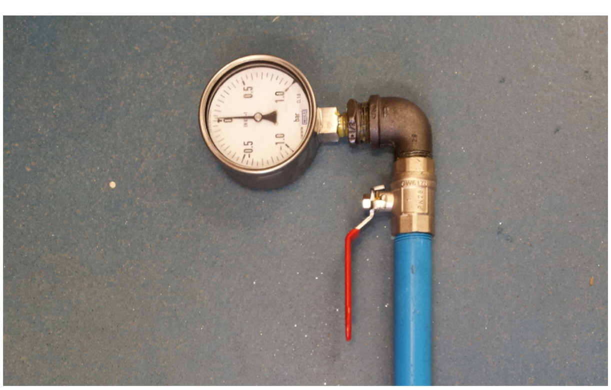

Fig. 13. Pressure gauge used to measure pore water pressure.

channel and to lower the piezometric level in the laminated bed below the invert level of the cross passage during the excavation period, up until the completion of the SGI permanent lining.

6. Ground conditions encountered during the excavation

During the excavation of CP6 the sand channel was encountered in the crown of the excavation (See Fig. 20), with the top of this layer being horizontal and coinciding with the base of the London Clay at 77.8mATD. The base of the sand channel, as encountered during the excavation, was dipping approximately 20 degrees to the South. As a consequence the observed thickness was gradually increasing from the East to the West, eventually reaching approximately 4 m at the intersection with the Westbound tunnel.

The sand channel material was medium to coarse sand and almost dry due to the dewatering. Consequently, the absence of suction caused frequent fall outs of the sand, with the material unravelling up to the base of the London Clay. As part of the normal procedure, the annulus between the timber and the ground was injected with cementitious grout.

Beneath the sand channels, a very stiff brown clay of the Upper Mottled Beds was encountered, which was underlain by Laminated Beds, between 72.2mATD and 67.8mATD.

The sand in the Upper Mottled Beds is described as clean, well sorted, fine quartz sand with grains subrounded to rounded and no presence of dark grains suggesting a depositional environment of relative high energy (See Fig. 20).

The sand in the Laminated Beds is fine quartz sand with yellow grain grains probably due to iron coating or impurities of the quartz. Relative to the sand lense in the Upper Mottled Beds, there is a higher percentage of dark grains (approximately 5%). The aspect of this sand shows similarities with the Thanet Sand as shown in Fig. 21.

Lower Mottled Beds were encountered between 67.8 and 66.5mATD at the bottom of the sump and consisted of cemented clay (including a layer of calcrete), Fig. 22. The top of the Upnor Formation is located 66.5mATD. It consisted of silica gravel, cemented with glauconitic sandy clay. Both the Lower Mottled Beds and the Upnor Formation required the use of mechanical breaking equipment to excavate effectively.

The presence of the sand layers and the hard bands caused delays during the excavation. The breakouts of sand and consequent increase of grouting accounted for approximately 7 days of delay. The mechanical breaking of the cemented conglomerate (Fig. 22) caused an additional delay during the sinking of the sump.

7. Settlement

One of the main risks associated with the construction of tunnels in urban areas is the effect of ground movement on any third party asset. On the surface of the highway, above the CP6 cross passage location, a line of precision levelling points were installed to monitor the ground movement from the Crossrail construction.

Fig. 23 shows the recorded settlement, at the levelling point located in the middle of the two running tunnels, above CP6, between January 2015 and December 2015.

Fig. 23 shows two clear episodes of settlement that are explained by the lag between the two TBMs. A timeline of the construction activities that could have had an influence on the settlement above CP6 is provided in Fig. 24.

The first episode during early January, resulted in a settlement of approximately 4 mm, was a consequence of the excavation of the Eastbound TBM tunnel. The second episode, between February and June 2015, was caused by the excavation of the Westbound TBM tunnelling and the cross passages groundwater control pumping that generated 9 mm of additional surface settlement. Due to the two activities taking place at about the same time, it is difficult to distinguish magnitudes of movement for each individual event, however, given that the Eastbound TBM produced 4 mm of settlement, it can be inferred that the settlement due to groundwater pumping was in the range of 4–5 mm. It is of note that the settlement due to the cross passage

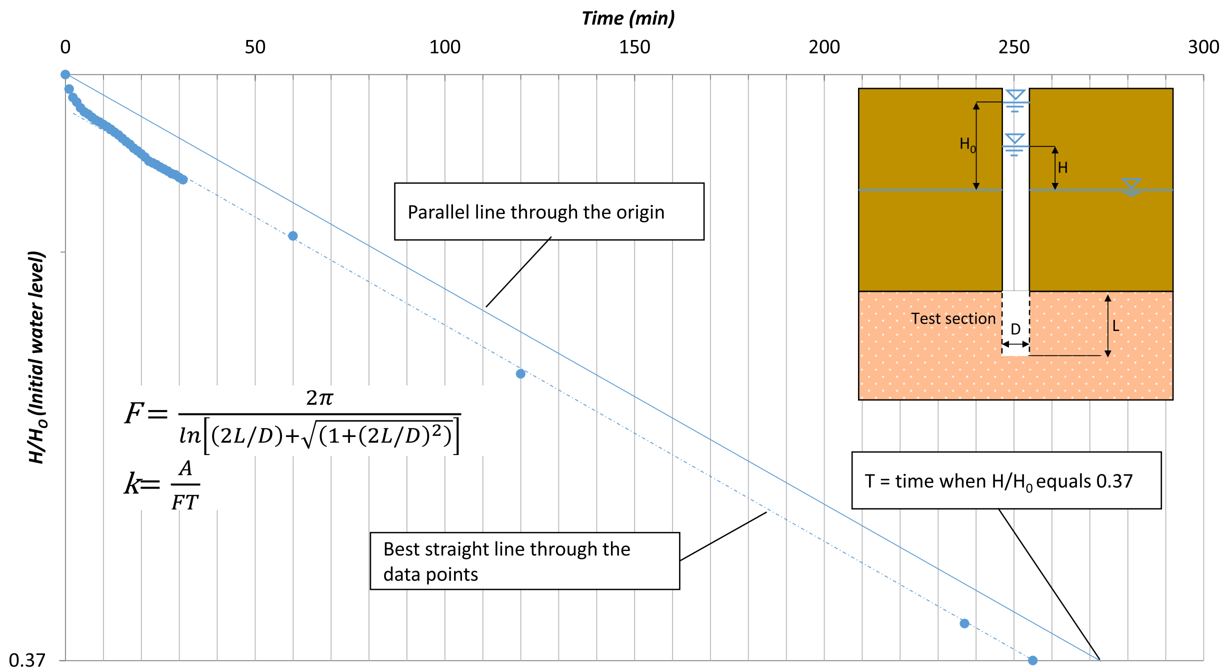

Fig. 14. Results of the falling head tests in the borehole LW9.

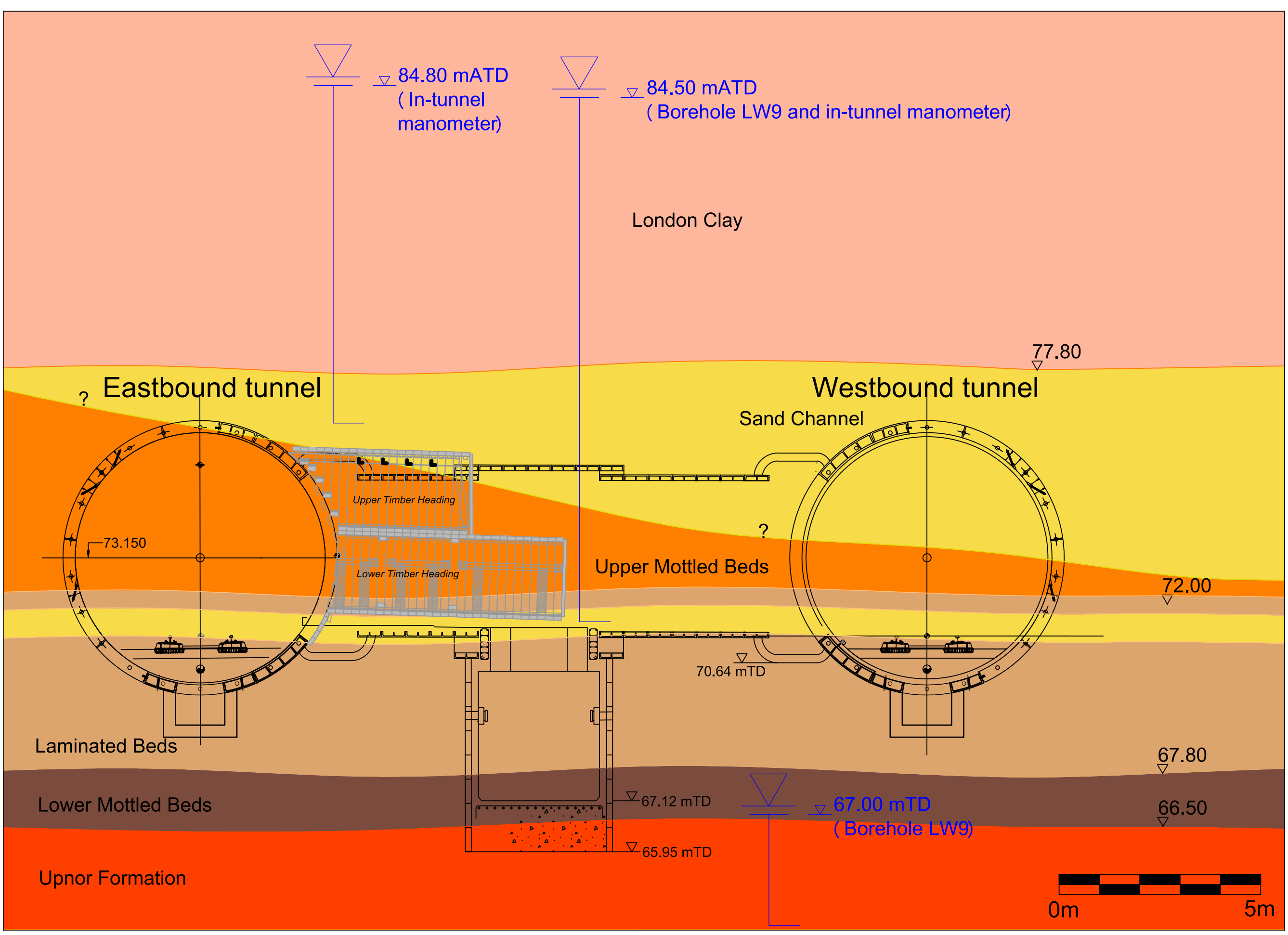

Fig. 15. New ground model for CP6 and sump.

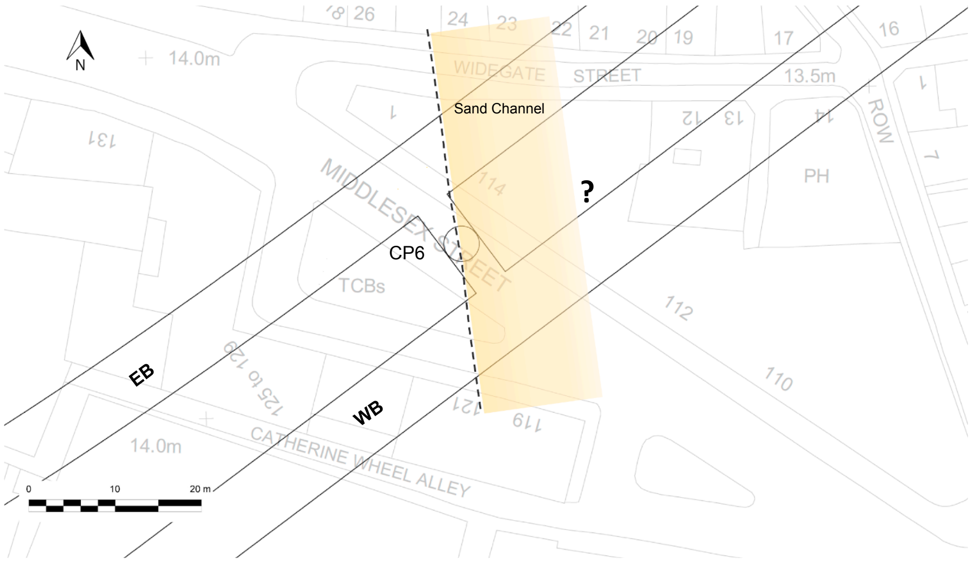

Fig. 16. Probable footprint of the sand channel at cross passage crown level.

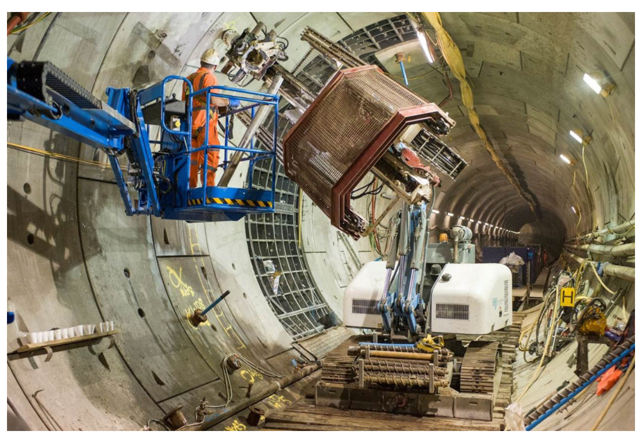

Fig. 17. View of the plant deployed to drill the well points next to CP6 opening sets in the Eastbound running tunnel.

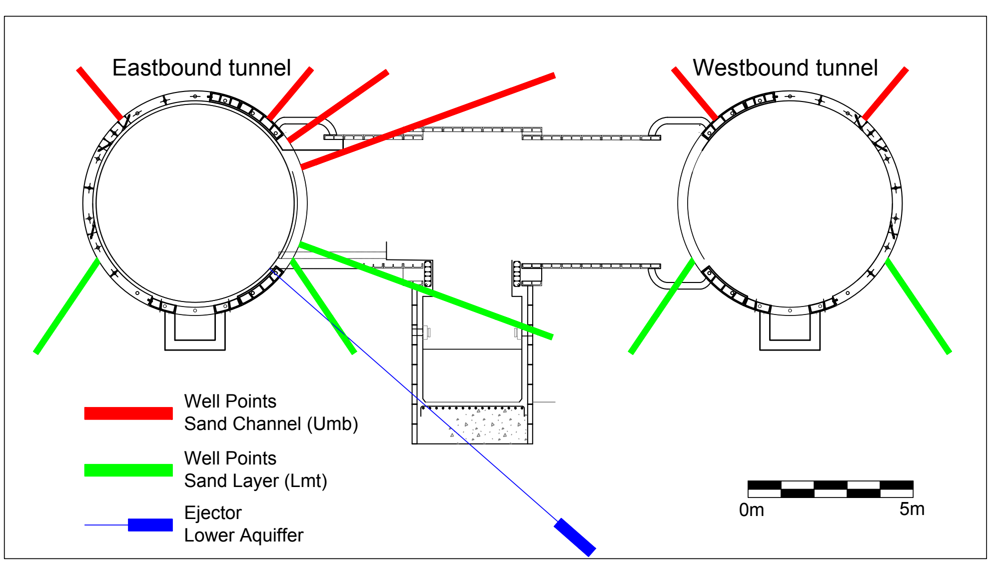

Fig. 18. Depressurisation strategy at construction stage. (Umb: Upper Mottled Beds. Lmt: Laminated Beds).

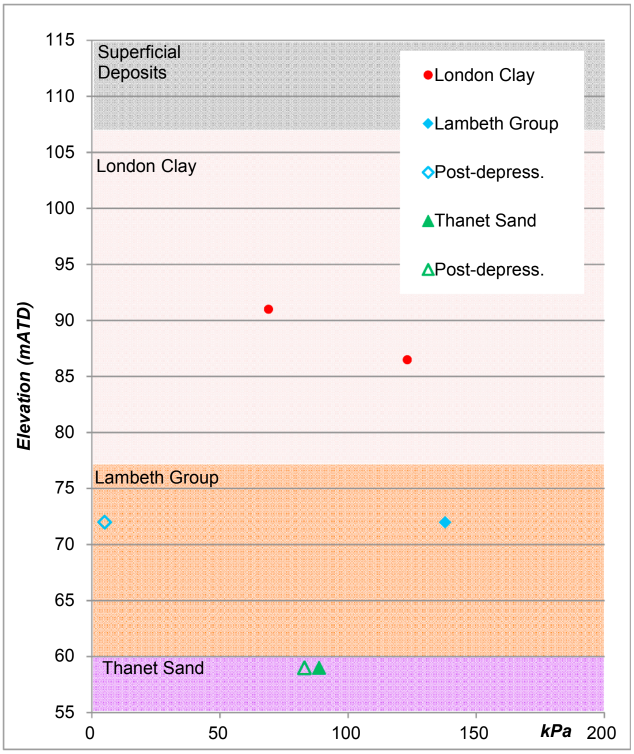

Fig. 19. Pore water pressure profile with the values pre- and post-depressurisation.

excavation was minimal.

8. Conclusions

The challenge in complex parts of tunnelling projects, such as cross passages, is to avoid or mitigate as much as possible the associated risks whilst still meeting the commercial targets. Delays in the completion of a cross passage can entail severe impacts in the programme as they are usually the last part of tunnelling to be carried out in the projects.

Open face tunnelling is still the favoured option for cross passages as it is the most efficient method, however, this technique carries inherent risk that is higher, in relative terms, to more modern mechanised techniques. The risks are further elevated where the geological conditions include water bearing granular materials or where the project is commissioned in an urban area. In London, the Lambeth Group is particularly challenging, in this regard, as in addition to it including water bearing granular strata, it contains sudden lateral and vertical changes in facies, as a consequence of its depositional environment. A carefully planned ground investigation is the main tool to reduce to anticipate these variations.

Within an urban environment, the limited availability of suitable access, above proposed underground structures, restricts the locations available for intrusive investigation. The Crossrail cross passage CP6, case exemplifies how limited access resulted in only two surface boreholes being drilled for the detailed design phase, which led to limited sampling of the ground.

The findings of this paper demonstrate that the urban constraint on ground investigation can have significant implications on the risks associated with open face tunnelling in variable ground. In the case of the CP6 cross passage, the differences identified between the borehole investigation and the later in-tunnel investigation, namely: sand beds higher in the structure profile and the undetected sand channel, resulted in a very different risk profile for the Client and Contractor. The opportunities to carry out intrusive investigation should be maximized to obtain the best possible quality of information and minimize the possibility of obtaining misleading information, following a 'right first time' philosophy.

It is the recommendation of the authors that ground investigation for the excavation of cross passages is approached in a phased manner, with surface boreholes used as an initial investigation and a later investigation planned from within running tunnels to further develop the ground model. The in-tunnel investigation at cross passage, CP6, with numerous probe-holes using hand held equipment and installation of

Fig. 20. View of excavation of the sand channel in the crown of the cross passage,on the left. On the right, view of boundary of the Upper Mottled Beds underlain the Laminated Beds.

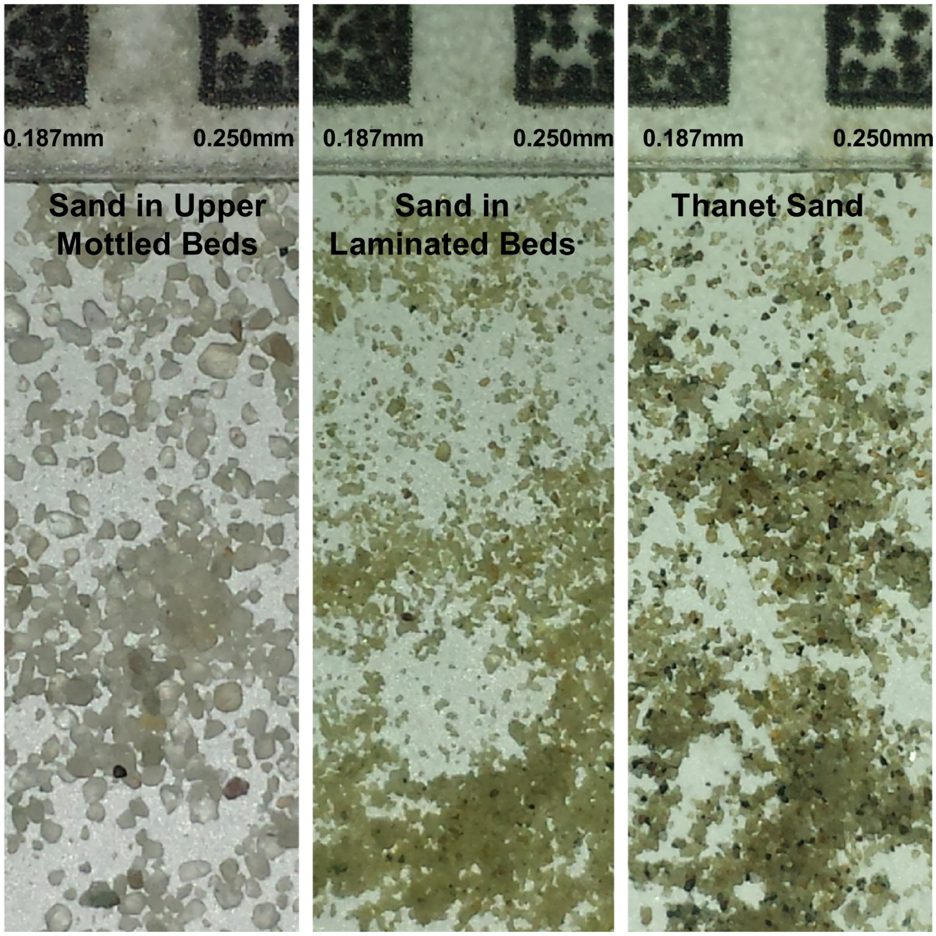

Fig. 21. Sand encountered in CP6 and also Thanet Sand for comparison.

Fig. 22. View of some calcrete from the Lower Mottled Beds (left and right below) and the Upnor Formation conglomerate exposed in the cross passage sump.

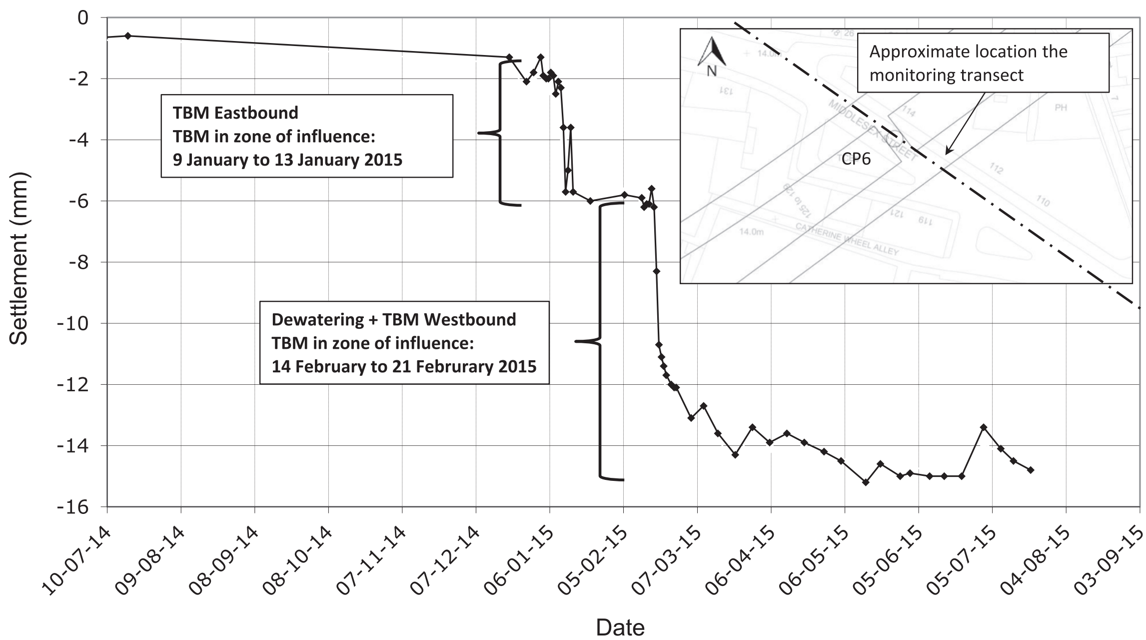

Fig. 23. Maximum settlement along Middlesex Street.

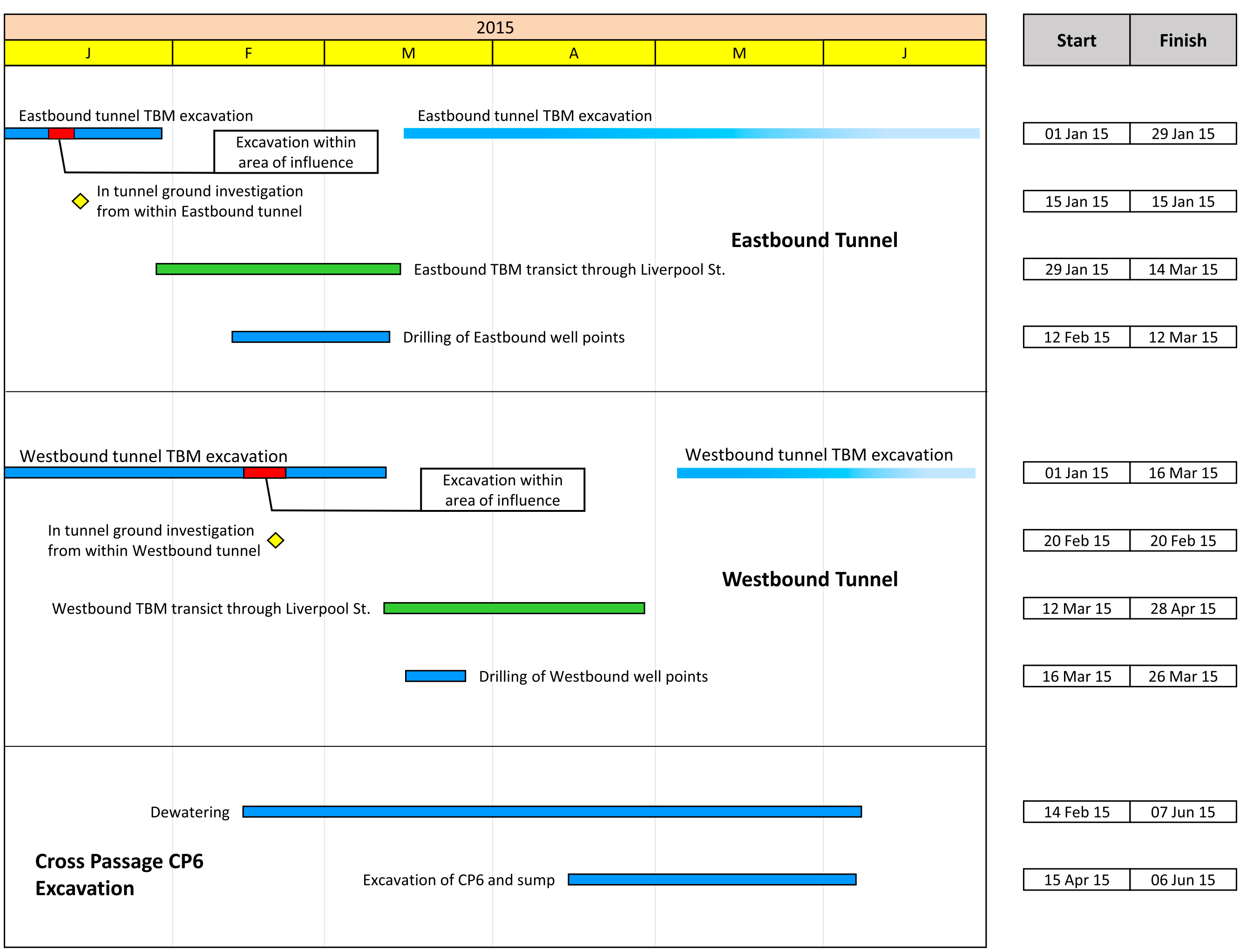

Fig. 24. Gantt chart of construction activities that affected the settlement.

pressure gauges to record piezometric levels illustrates an example of how to obtain key additional information to update the ground profile without disrupting the TBM operations.

Acknowledgements

The authors would like to show their gratitude to Crossrail Ltd, Dragados-Sisk JV, WJ Groundwater, OTB Engineering, Tomas Horak (Dragados UK) and Barry Barnicle (Dragados UK) for their assistance on the elaboration of this paper.

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4.1 Otras publicaciones

4.1.1 Multi-aquifer pressure relief in east London.

(Título original en inglés: Reducción de presiones hidráulicas en un acuífero multicapa en el este de Londres)

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Multi-aquifer pressure relief in east London

Contrôle de pression dans multiple aquifères dans l'est de Londrès

T.O.L Roberts[*1], E. Linde[2], C. Vicente[3] and G. Holmes[1]

[1] *WJ Groundwater Ltd, London, UK*
[2] *OTB Engineering, London, UK*
[3] *Dragados Sisk JV, London, UK*
[*] *Corresponding Author*

ABSTRACT Two adjacent deep shafts in east London involved excavation to 30 m below standing groundwater level and required temporary control of pressures in the Lower Aquifer below the London Clay. Three distinct aquifer horizons were present comprising the Lambeth Group channel sands, Thanet Sand and Chalk. One of the shafts was constructed using a circular diaphragm wall with a reinforced concrete base slab and the second shaft was constructed using SCL techniques with a domed base. The different structural approaches plus slightly differing depths led to radically differing requirements for groundwater control which are explored in this paper. One of the shafts required substantial lower aquifer pressure relief which was achieved using deepwells targeting the Chalk plus a combination of external pumped Thanet Sand wells and internal passive wells separately targeting the Thanet Sand and Lambeth Group. The second shallower shaft was successfully constructed using internal passive relief wells.

RÉSUMÉ Deux puits profonds adjacents dans l'est de Londres impliquant une excavation à 30 m sous le de niveau de la nappe et nécessitant contrôle des pressions dans l'aquifère inférieur situe au dessous de l'argile de Londres. Trois aquifères distincts sont présents: les sables du Lambeth Group, les sables du Thanet Sand et enfin la craie. L'un des puits a été construit à l'aide d'une paroi moulée circulaire avec un radier en béton armé, et le deuxième puits a été construit à l'aide de la technique SCL avec un fond bombé. Les différentes approches structurelles avec des profondeurs différentes conduisent à des exigences radicalement différentes pour le contrôle des eaux souterraines, ce qui est décrit dans le présent document. L'un des puits a nécessité une importante dépressurisation de l’aquifère inférieur qui a été réalisé à l'aide de puits équipes de pompes ciblant la craie, plus une combinaison de puits équipes de pompes dans le Thanet Sand , plus une série de puits passifs ciblant le Thanet Sand et le Lambeth group. Le deuxième puits moins profond a été construit avec succès à l'aide de puits passifs situe dans l’enveloppe du puits.

1 INTRODUCTION

Crossrail is a major new east-west rail link through central London. The twin 6.2m diameter running tunnels under central London were driven by tunnel boring machines (TBM) from the west and east to meet at Farringdon. This paper is concerned with the dewatering and pressure relief measures required for the eastern drive and auxiliary shafts located at Limmo peninsula between the Isle of Dogs and the Royal Docks, Figure 1. The site is bounded by the tidal waters of Bow Creek to the west, Canning Town Station to the east (Jubilee Line and Docklands Light Railway) and the Lower Lea Crossing (road bridge) to the south. The location was chosen to facilitate material delivery and muck-away for the TBM drives by river.

The main shaft at Limmo is 30m internal diameter formed by a circular, 1.2 m thick, diaphragm wall. Ground level at the shaft site is at 107mTD (0.00mOD = 100m Tunnel Datum). The shaft dig depth was 44.3m and the toe depth of the diaphragm walls was to 55m below ground level (bgl). The shaft includes a 500mm thick ‘no fines’ concrete drainage layer below the base slab. This layer is pumped by sumps during the 3 year tunnel construction period to avoid build-up of pressure below the base slab. Once the TBM drives are complete this main shaft will be fitted out to provide ventilation and pedestrian access to rail level. The finished shaft will have sufficient structural integrity and dead weight to resist the groundwater uplift pressures. In order to facilitate access to launch and service the TBMs for the main 8.3km drives to Farringdon and secondary 0.9km drives to Victoria Dock portal a second auxiliary shaft was required. The auxiliary shaft is 27m internal diameter formed by sheetpiles through the overburden with Sprayed Concrete Lining (SCL) below. Dig depth for the shaft was 39.8mbgl and it was completed with a domed base to resist uplift groundwater pressures during the period of the TBM tunnelling works. On completion of tunnelling the auxiliary shaft will be backfilled. The layout of the shafts is shown in Figure 2.

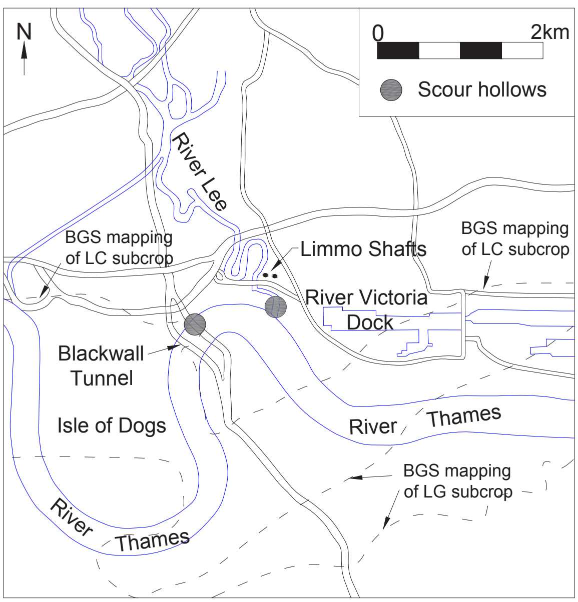

Figure 1. Site Location in east London.

The main and auxiliary shafts were connected by twin SCL tunnels plus back and fore shunts at rail level at approximately 68 mTD (39 m depth).

1.1 Borehole data

A large number of boreholes were drilled at the site to investigate the ground profile and these have been summarised in the schematic soil profile in Figure 3. This shows the typical geological sequence present across central London comprising recent superficial deposits of Made Ground, Alluvium and Terrace Gravels underlain by London Clay, Lower London Tertiaries with Chalk below. Variations in the strata interface levels suggest evidence of faulting in the vicinity, mainly to the west of the shafts.

Figure 2. Aerial photograph of site from the south. (Photograph courtesy of Crossrail)

1.2 Hydrogeology

The Terrace Gravels are water bearing, forming a discreet Upper Aquifer. The London Clay provides a confining layer for the regionally important Lower Aquifer comprising the Chalk and Thanet Sand (Royse et al. 2012). The granular horizons in the lower Lambeth Group are also in good hydraulic connection with the Lower Aquifer system. The granular horizons in the upper Lambeth Group above the Mid-Lambeth Hiatus (MLH), and in the Harwich Formation, have limited hydraulic connection to the Lower Aquifer and are sometimes referred to as the Intermediate Aquifer. The granular horizons in the Lambeth Group are known to be laterally intermittent (Page & Skipper 2000). Groundwater levels in the Upper Aquifer are approximately 100mTD and are controlled by levels in the adjacent tidal river.

Recharge to the Lower Aquifer is via the Chalk outcrops to the North and South of London and also directly from the Terrace Gravels/Thames where the London Clay and Lambeth Group subcrop 2 to 3km to the south of the site. Known scour hollows in the surface of the London Clay are shown on Figure 1. The scour feature at the Blackwall Tunnel is known to penetrate to the Lambeth Group whereas the one at the mouth of the River Lea is thought not to penetrate through the London Clay. Excessive abstraction for water supply in central London led to a substantial lowering of the natural groundwater level in the Lower Aquifer between about 1850 and 1965. Prior to 1850 groundwater levels at Limmo were thought to be up to approximately 110mTD (Simpson et al. 1989 and Environment Agency 2014), falling to about 85mTD by 1965 and recovering to the levels shown in Figures 3 and 4 by the start of the works. The profiles in Figure 4 have been interpreted from monitoring data obtained from the pumping test data, from experience and from the Crossrail data base.

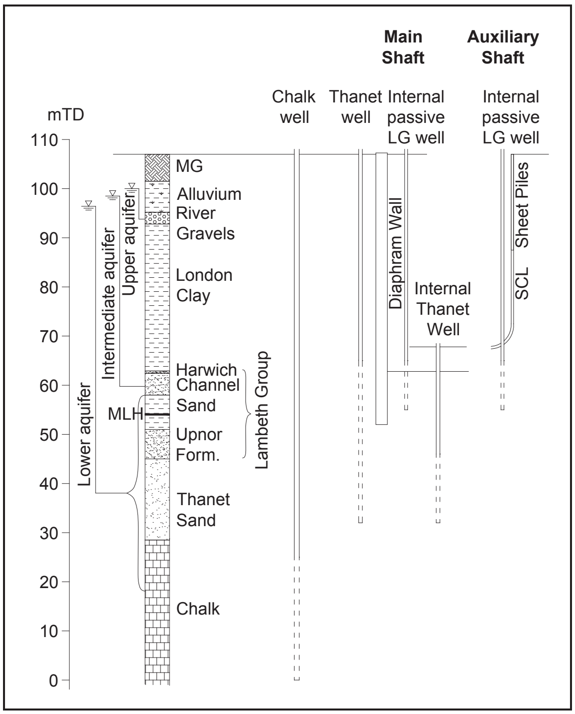

Figure 3. Schematic section showing ground profile.

1.3 Pumping test

A comprehensive programme of pumping tests was commissioned by Crossrail prior to the works. This involved installation and test pumping of wells which individually targeted the Chalk, Thanet Sand, the Lambeth Group granular horizons (two wells) and the Harwich Formation (two wells). The test regime is summarised in Table 1 and the distance drawdown data are summarised in Figure 5. The following should be noted with respect to Figure 5.

- A vacuum was applied to the Harwich Formation and Lambeth Group wells in order to maximise drawdown.
- Data for the first Harwich test well has not been included as there was no discernible response in any of the piezometers.
- The Lambeth Group wells targeted the Lambeth Group above the MLH. The data for the two Lambeth Group wells have been combined and normalised to an average flow of 0.45l/s.
- Piezometers which showed no response are not included.
- For the Harwich Formation tests, Lambeth Group piezometers above the MLH which responded to pumping are included, and vice versa for the Lambeth Group tests.
- The Thanet Sand data includes piezometers with response zones in the Upnor Formation (generally granular horizon of the lower Lambeth Group, which usually has a good hydraulic connection to the Thanet Sand).
- The Chalk test includes piezometers in the Thanet Sand and Upnor Formation.

Table 1. Pumping test regime.

Stratum	Well depth	Flow rate	Pumping
Harwich	47.5m	0.18 l/s	69 hrs
Harwich	49.3m	0.14 l/s	46 hrs
Lambeth	55.5m	0.4 l/s	39 hrs
Lambeth	57m	0.52 l/s	42 hrs
Thanet	79m	2.5 l/s	40 hrs
Chalk	114m	20 l/s	9 days 10 hrs

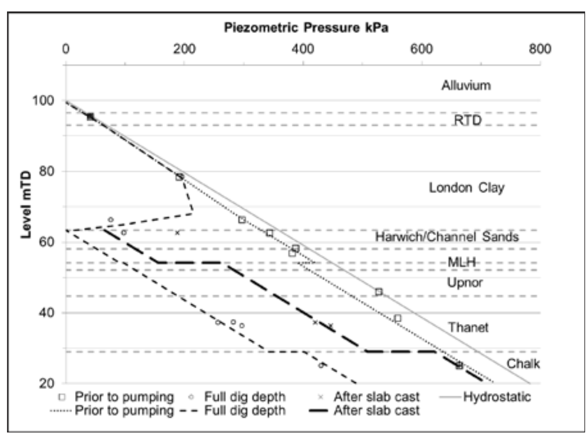

Figure 4. Summary of pore pressure profile.

Figure 5 shows the generally good response to pumping of all principal horizons. The successive order of magnitude increase in flow for the Harwich/Lambeth Group to the Thanet and again from the Thanet to the Chalk is apparent. Also evident are the more variable localised response of the Harwich/Lambeth Group (above the MLH). The relatively localised response of the Thanet wells compared to the more extensive response for the Chalk pumping is also apparent.

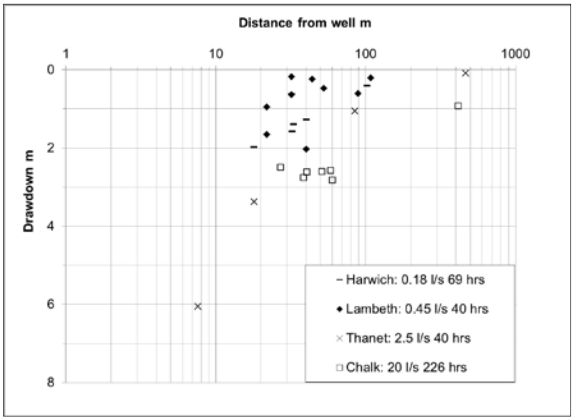

Figure 5. Summary of pumping test data.

2 DEPRESSURISATION REQUIREMENTS

Groundwater in the Upper Aquifer is cut-off by the diaphragm wall and sheet piles around the main and auxiliary shafts respectively.

During construction of the main shaft there was a concern that high groundwater pressures in more permeable layers trapped under less permeable layers would cause uplift failure of the soils in the base of shaft. This could occur at the base of the London Clay and at the various granular horizons in the Lambeth Group down to the Upnor Formation.

The requirement was to reduce the groundwater pressure in these strata such that the design groundwater pressure multiplied by 1.1 is less than the overburden pressure multiplied by a partial factor of 0.9, BS EN 1997-1:2004 2.4.7.4. In other words the groundwater pressure needs to be reduced to less than 0.82 times (0.9/1.1) the over burden pressure at every level below the base.

3 TARGET PIEZOMETRIC LEVELS

The target piezometric level can be calculated for any horizon and depth of excavation by taking the strata interface levels as indicated in Figure 3 and assuming a density for all of the soils present of 20kN/m^3. The results for the main shaft are given in Table 2.

Table 2. Target groundwater levels at selected horizons for main shaft. Values in italics are above, and values in bold are below, pre-construction groundwater levels.

Excavation level	Harwich 63.1mTD	Upnor 50.4mTD	Thanet 44.5mTD
90.0mTD	*108.0*	*116.4*	*120.4*
80.0mTD	**91.3**	*99.8*	*103.7*
70.0mTD	**74.6**	**83.1**	**87.0**
62. 7mTD	**62.4**	**70.9**	**74.8**

For the main shaft, due to uncertainty regarding diaphragm wall-soil interface shear strength, it was difficult to take account of soil shear strength and wall friction in a defensible manner. This was much easier to justify for the smaller auxiliary shaft which was constructed by SCL methods with a domed base. A calculation for the auxiliary shaft was carried out, which took soil shear strength into account, and this indicated a minimal requirement for pressure relief in the Upnor Formation and below, and a reduction to 81.5mTD in the Harwich Formation. Once complete, the SCL lining for the auxiliary shaft was designed so that no pressure relief was required for the 3 year tunnel construction period.

The main shaft base was keyed into the diaphragm walls in order to mobilise the base slab and shaft weight to resist uplift pressures. Taking the installed concrete into account resulted in a target piezometric pressure for the 3 year tunnel construction period of 93.5mTD, which amounted to only a few metres of drawdown.

4 DEPRESSURISATION STRATEGY

The requirement for substantial pressure reduction in the Upnor Formation for the main shaft construction necessitated pressure reduction in the Thanet Sand and Chalk. Although the Chalk and Thanet are in good hydraulic connection the results from the pumping test indicated that appreciable local drawdown in the Thanet could be achieved without a corresponding reduction in the Chalk. In order to minimise both the abstraction flows and the aerial extent of drawdown, the Chalk dewatering was only intended to achieve part of the required drawdown in the Upnor with the remainder achieved by local Thanet wells.

The pumping test showed that the Lower Aquifer pumping had minimal impact on pore pressures in the Lambeth Group above the Mid-Lambeth Hiatus. For the main shaft the granular horizons above the Mid-Lambeth Hiatus are cut-off by the diaphragm wall, so that only a minimal passive relief well system was needed.

Some drawdown was required in the Upnor after the base slab was cast and it was considered that this could be achieved with internal Thanet passive relief wells.

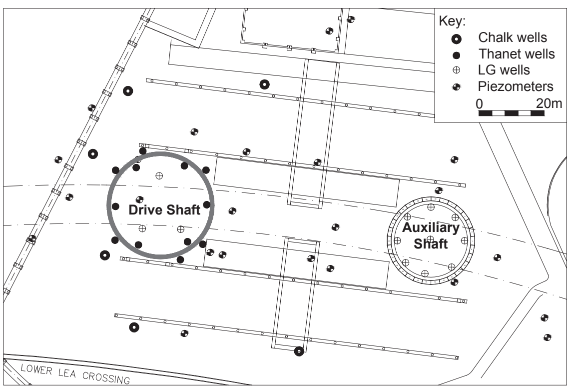

Figure 6. Site layout showing shafts, wells and piezometers.

The auxiliary shaft is founded towards the base of the London Clay. The wide impact of the Chalk well dewatering for the main shaft provided the required lower aquifer pressure relief for the auxiliary shaft during shaft construction. Internal passive relief wells provided the pressure relief needed in the Harwich Formation and Lambeth Group channel sands. This had to be a more extensive system than for the main shaft because the SCL method of construction provides no cut-off below dig level.

A key benefit of this strategy was that it avoided the need for continuous pumping from the Chalk throughout the 3 year tunnel construction program. The strategy exploits the poor hydraulic connection between the intermediate aquifer and lower aquifer and the restricted hydraulic connection between the Thanet Sand and Chalk which have caused difficulty on previous construction projects undertaken in a similar geological setting (Linney & Withers 1998).

The proposed well array is shown in Figure 6 and summarised in Table 3.

Table 3. Summary of dewatering well array.

	External Chalk	External Thanet	Internal Lambeth	Internal Thanet
No. of wells				
Main shaft	7 No.	6 No.	3 No.	6 No.
Aux. shaft			9 No.	
Distance from shaft	<50m	<5m	Internal	Internal
Install level	107mTD	107mTD	107mTD	68mTD
Well depth	107m	75m	52m	36m
Pump size	<25 l/s	3 l/s	Passive	Passive

Installation of the internal Lambeth Group passive wells from ground level allowed a program of test pumping prior to excavation. For the main shaft this demonstrated the effectiveness of the well array and the integrity of the diaphragm wall cut-off below dig level.

5 MONITORING RESULTS

Manual and data logger monitoring, using vibrating wire transducers, was undertaken on the piezometer array in Figure 6. The array included instruments with response zones in all strata present. A sample of the data collected is presented in Figure 7. Pumping started with three of the Chalk wells in December 2011. These wells were switched off, and were then restarted in February 2012 with pumping from the external Thanet wells commencing mid-March 2012.

It can be seen that the Thanet and Chalk initially responded together, with an additional 8 m of drawdown generated in the Thanet Sand when the Thanet well pumping commenced. Abstraction flows from the Chalk were 160 l/s with flow from the external Thanet Sand wells at 11 l/s. Data for the Lambeth Group is intermittent. The Lambeth Group showed minimal response to the Chalk pumping. The Thanet wells are screened up into the Lambeth Group which did respond when pumping started on the Thanet wells. Note that the data plotted in Figure 7 are only for instruments installed around the outside of the shafts. Figure 4 shows pore pressure data prior to construction, when the main shaft had reached full excavation level and when the external Chalk and

Thanet wells were shut down so that only the passive internal Lambeth and Thanet wells were operational (which was the case during the 3 year tunnel construction period). The lines show the interpreted pore pressure profile within the main shaft at the same program points in the course of construction.

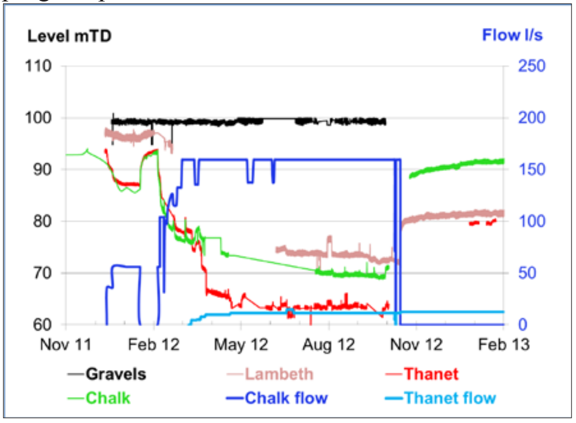

Figure 7. Sample of monitoring data in vicinity of main shaft.

Surface level monitoring indicated that there was no discernible settlement (i.e. <2mm) when pumping commenced from the Chalk wells only. Some settlement became evident local to the shafts when pumping commenced from the Thanet Sand wells and the auxiliary shaft passive relief wells. This suggests that the settlement in this vicinity was primarily related to the depressurisation of the upper Lambeth Group. Surface settlements of the order of up to 20 mm were recorded 50 m from the shafts by the end of the 8 month shaft construction period with no measurable settlement 400 m from the shaft.

6 CONCLUSIONS

A comparison between the estimated abstraction flow based on simple linear superposition of the pumping test data and the measured flows is given in Table 4. The high actual abstraction flow for the Thanet wells relative to the estimated flow is due to the recharge from the Chalk below. The rate of recharge would be expected to be disproportionally higher as greater drawdown (compared to the pumping test) is achieved in the Thanet Sand relative to the Chalk which is in very close proximity below.

Table 4. Comparison between estimated and actual flows.

	Thanet	Chalk
Pump test flow rate	2.5 l/s	20 l/s
Centre of shaft to wells (average)	18m	30m
Pump test drawdown at average dist.	3.4m	2.5m
Drawdown achieved	8m	24m
Estimated flow (linear superposition)	5.8 l/s	192 l/s
Actual abstraction flow rate	11 l/s	160 l/s

The situation for the Chalk is reversed with flows less than suggested by linear superposition. This is because the limited recharge to the Chalk results in an increasing distance of influence and corresponding reduction in flow over time. An analysis based on the initial 20 m of drawdown achieved gives a very close match between the estimated and measured flows.

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Many individuals contributed to the success of this project and the authors would like to acknowledge the project client Crossrail, the permanent works designer for the main shaft Mott Macdonald, the main contractor Dragados Sisk Joint Venture, the temporary works designer OTB Engineering and the dewatering subcontractor WJ Groundwater Ltd.

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4.1.2 Design and construction of Crossrail Stepney Green sprayed concrete lined caverns.

(Título original en inglés: Diseño y construcción mediante hormigón proyectado de las cavernas de Stepney Green, para el proyecto de Crossrail)

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Design and Construction of Crossrail Stepney Green Spayed Concrete Lined Caverns

Nicholas Tucker Eur Ing, BSc, CEng MICE, Mott MacDonald Ltd.
Katherine Scevity BEng, MCQI CQP, Bechtel
John Comins BEng, HND, ACSM, Dragados-Sisk JV
Emilio Linde BSc, MSc, DIC, EurGeol, FGS, OTB Engineering Ltd.

Abstract

At over 50 m long, 17 m wide and 14 m high the Stepney Green Caverns are the largest caverns ever built using Sprayed Concrete Lining techniques in Central London. This paper describes the design, construction and monitoring of the Primary Sprayed Concrete Lining together with lessons learnt that can be applied to other Crossrail structures.

The lining system consists of a fibre reinforced Primary Lining, a waterproofing membrane and a fibre reinforced Secondary Lining. A combination of spray applied and sheet membrane waterproofing was utilised. The permanent Primary Lining was designed to take the full short-term applied ground load and surface surcharges. Additional long-term loads, subsequent to the installation of the Secondary Lining are shared between the two linings.

The excavation and support methodology varies along the length of the cavern. The launch chamber was constructed using top-heading, bench & invert, the smaller section of cavern using single side-wall drift and the largest part of the cavern using double side-wall drifts with a central gallery.

The cavern profile at each advance was controlled using target-less remote laser scanning equipment, set up at a safe distance from the excavation face. The performance of the lining was constantly monitored using deformation monitoring points, pressure cells, inclinometers, extensometers and piezometers and reviewed daily.

Project Description

At over 50 m long, 17 m wide and 14 m high the Stepney Green Caverns are the largest caverns ever built using Sprayed Concrete Lining techniques in Central London.

The Stepney Green Caverns were excavated under the East End of London at Stepney Green to create the underground rail track junctions on the city's Crossrail project that will permit train services to run either north towards Stratford or south towards Canary Wharf. See Figure 1.

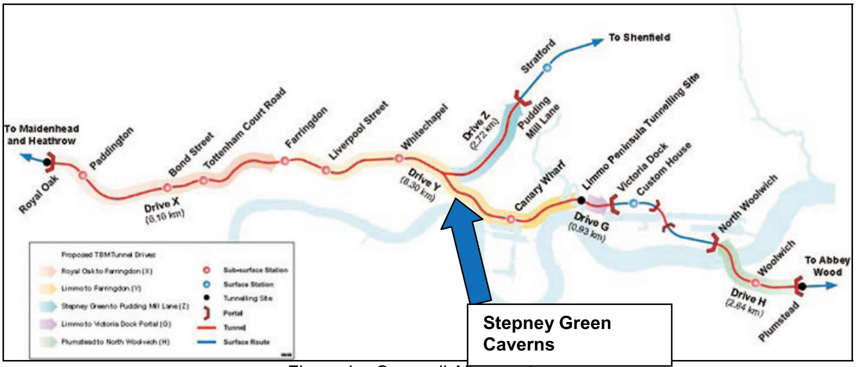


Figure 1 – Crossrail Alignment

The vertical alignments of the caverns are different; with the westbound alignment being approximately 5m deeper. This meant that the eastbound cavern was wholly excavated within London Clay whilst the westbound cavern encountered up to 4m of Lambeth Group below axis level.

Excavation of the caverns progressed from a rectangular access shaft, built across the twin TBM running tunnels to the west of the junctions that will house M&E utility rooms, a ventilation facility and provide an emergency ingress and egress for Crossrail during operation. The SCL cavern design was carried out by Mott MacDonald Ltd. supported by Gall Zeildler Consultants. Temporary works were designed by OTB on the contractor's behalf. The caverns form part of the Eastern Running Tunnels Contract C305 that was awarded to the Dragados/Sisk JV in December 2010 for approximately £500 million.

The contractor took possession of the site in March 2010 to set up and start excavation of the large access and operations shaft. The shaft construction used diaphragm slurry walls of up to 40m deep, 15m wide and 60m long with bar reinforcement cages and over 9,500m^3 of concrete. Shaft construction was completed in January 2013 and SCL excavation commenced on the EB launch tunnel in November 2012 and the EB cavern in January 2013. The EB cavern Primary Lining was completed in May 2013, followed by the westbound in August 2013. To construct the caverns and launch tunnels, the project team had to excavate approximately 38,000m^3 (bulked) of material and apply over 6,000m^3 sprayed concrete to the temporary and permanent walls.

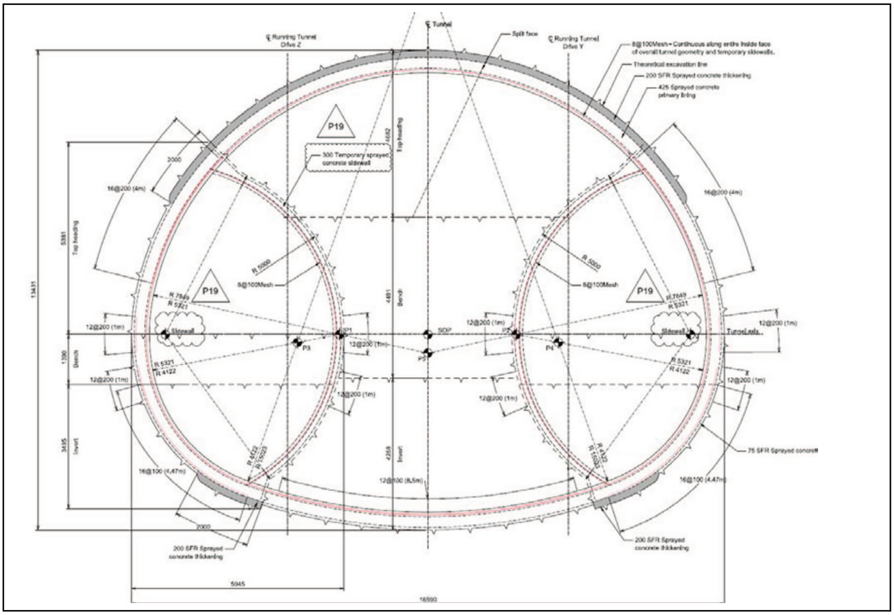


Figure 2 – Stepney Green Cavern – Primary Lining for Largest Section

Risk Management

The construction of the Stepney Green Caverns was one of the highest risks on the CRL programme. The size and location of the caverns are unique; caverns of this size had never been built in the same geological conditions in London. The risks associated with the construction of the caverns needed to be carefully managed in order to meet the Client's and other legal obligations and followed the principles of the Joint Code of Practice for Risk Management of Tunnel Works.

The period of time when the face or crown is left unsupported was when the high risk events were most likely to occur. In accordance with the requirements of CDM legislation, the Designer identified risks that could not be mitigated during design and passed them on to the construction phase.

The main risks included;

- Instability and/or collapse of the tunnel through water ingress due to presence of sand channels.
- Local face instability with block falls or collapse due to highly fissured and slicken-sided zones of clay.
- Potential for isolated pore water pressure in granular layers causing instability of excavation face and inundation.
- Unknown hydrology causing inundation of the tunnel causing drowning or injury.
- Inadequate depressurisation of non-cohesive strata leading to potential tunnel excavation instability, tunnel collapse and impact on third parties and the general public.

In order to manage these risks effectively a suite of documents, review meetings and other tools were utilised.

The SCL Action Plan was the main risk management document used by the Contractor and contained details on how to mitigate risks. The action plan described how the ground and tunnel movements, water inflows and other unexpected situations would be controlled during the works. Specific Action Plans were also developed for high risk areas of the works, e.g. for the Westbound Cavern – Groundwater and Depressurisation.

Due to the high risk nature of these works, key personnel were required to meet strict competency requirements and approved by the Client's Project Manager. To ensure all parties were aware of the works, their responsibilities and the impact of other interfaces a suite of meetings were put in place. Namely the Shift Review Group (SRG), the Contract Technical Committee (CTC) and the Engineering Review Panel (ERP).

The SRG was held daily. Representatives from the Client, the Contractor, the Designer and the Senior SCL Engineer were required to attend the meeting. The role of the group was to review the last 24 hours of work, discuss any observations (health & safety, quality and technical), review the monitoring data, face logs, probe holes etc. Having reviewed all the data, the works for the next 24 hours were then planned and recorded on the Required Excavation and Support Sheet (RESS), acting as the permit to work system.

The CTC met bi-weekly to review the effectiveness of SRG process. Senior representatives from the Client, Contractor, Designer and the Senior SCL Engineer met to review the summary of the previous week's progress. The meeting was also used to discuss technical issues such as excavation methodology and the effectiveness of the depressurisation system. The CTC helped manage high risk operations by highlighting areas which could be improved upon.

The ERP met every four weeks and included members from the CRL Central Engineering Group. The ERP played an overseeing role during the works. They reviewed the output of the SRG and the CTC, gathering and disseminating any lessons learnt. High risk activities undertaken on other parts of Crossrail were discussed at the ERP and any lessons learnt incorporated in to the cavern works.

The CRL Expert Panel provided valuable peer review, allowing the overall team to be questioned on the robustness of the systems in place for controlling the works. They visited the site on a number of occasions, in advance on key risk activities commencing, for which the Contractor and the CRL site team and Designer prepared presentations for discussion.

Primary Lining Analysis & Design

SCL Lining System - The lining system developed for the SCL tunnels at Stepney Green comprised a steel fibre reinforced Primary Lining, a waterproofing membrane and a non-steel fibre reinforced Secondary Lining with bar-reinforcement. The lining is a double-shell lining with both linings considered part of the permanent load bearing structure. A combination of spray applied waterproofing and sheet membrane was designed for SCL structures for Stepney Green.

Material Durability - Key performance criteria that determined the durability of the sprayed concrete material are as follows:

- Concrete mix design and reinforcement cover to meet the exposure class according to BS8500 to achieve 120-year design life.
- Water to cement ratio targeted between the 0.4 and 0.45 range to create low permeability.
- Active pozzolans, namely silica fume slurry, to further decrease the permeability to less than 1×10^{-12} m/s enhancing chemical stability of the concrete.
- Structural steel fibres to promote homogenous distribution of micro-cracks.
- Micro-synthetic fibres in the Secondary Lining to provide concrete anti-spalling mechanism in the event of a fire.
- Alkali-free set accelerators at dosages less than 8% by weight of total cementitious materials to promote early and long term compressive strengths.

Performance Requirements – Key performance requirements for the Primary and Secondary SCL are given in Figure 3;

Performance Requirement	**Primary SCL**	**Secondary SCL**
Design life	120 years	120 years
Early age strength development (minimum)	Greater than modified J2 requirements shown in Figure 4.	Greater than J2 requirements shown in Figure 4.
Long term compressive strength	28 days>C28/34 90 days>C32/40	28 days>C28/34 90 days>C32/40
Structural fibre performance	28 days: Class D1 S1.8 ; D3S1.4 90 days: Class D1 S2; D3S1.4 (After BS EN 14487-1)	Residual flexural strengths as for Primary SCL
Concrete shrinkage	Less than 0.03% at 28 days	Less than 0.03% at 28 days
Exposure class	DC-2 as per Table A.11 of BS 8500-1	XC3 as per Table A.5 of BS 8500-1
Water permeability (chemical durability)	< 35mm penetration – See clause c) in sprayed concrete trial section of KT20.2201	Not applicable
Cementitious content	> 380kg/m³ Type: CEM II A-D, CII A-D	> 380kg/m³ Type: CEM I, CEM II A-D, CII A-D
Water-cement ratio	< 0.45	< 0.5
Fire protection	Not required	Fire protection requirements to EUREKA time-temperature curve

Figure 3 – SCL performance Requirements

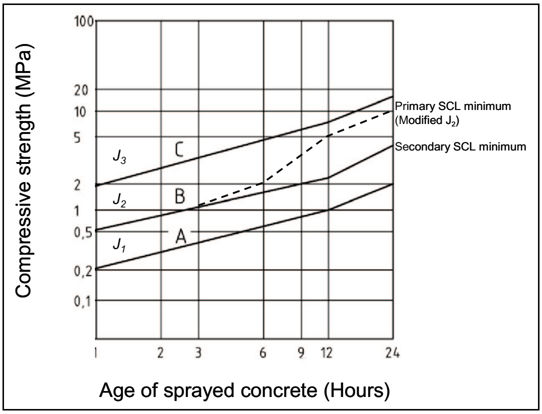


Figure 4: Early Age Strength development for sprayed concrete - modified (based on EN 14487-1:2005)

Primary lining – the Primary Lining was designed to take the full short-term applied ground load and any other loads, such as from compensation grouting and surface surcharges, expected in the two to three years prior to Secondary Lining installation. Additional long-term loads, such as those applied through on-going consolidation of the London Clay, subsequent to the installation of the Secondary lining were shared between the two linings.

The Primary Lining comprised two layers; P1 steel fibre reinforced sprayed concrete and P2 bar reinforced sprayed concrete. Steel fibres were provided in order to increase the ductility of the concrete and provide toughness and post crack resistance in the long term. A minimum of 40kg/m³ of fibres were specified to reduce crack widths in sprayed concrete to 0.3mm. In addition bar reinforcement was provided to accommodate calculated tensile forces. Lattice girders were provided to ensure shape control and additional robustness to the design although not actually required by structural calculation.

A 75mm thick initial concrete layer, sprayed directly against the ground, was provided. This was included as load bearing in the short term but was considered sacrificial and not considered in load capacity calculations post installation of the Secondary Lining.

Secondary Lining - Taking into account the loads and stresses already taken by the Primary Lining, the Secondary Lining was designed to carry:

- The full long term water pressure,
- Internal loads, such as mechanical and electrical equipment,
- Long term load conditions; e.g. the effects of consolidation,
- The effects of temperature and shrinkage,
- Fire loads,
- The effects of degradation of the 75mm sacrificial layer,
- An allowance for additional loads that might be applied in the future.

The proportion of consolidation loading applied to the Secondary lining was calculated using FLAC numerical modelling. The Secondary Lining was reinforced with conventional bar reinforcement.

Secondary linings were designed to carry sufficient residual capacity to resist ground loading after a EUREKA time/temperature fire curve, as defined in the Technical Specification for

Interoperability – Safety in Railway Tunnels TSI-SRT. The EUREKA curve was developed for the rail industry in Germany. The Secondary lining concrete (cast in-situ or sprayed) contains micro-synthetic fibres in order to limit explosive spalling and maintain structural integrity. The quantity of fibres was determined by pre-construction fire testing.

Waterproofing Systems - Waterproofing membranes were installed between the Primary and Secondary linings. The waterproofing system provided is "un-drained"; i.e. the groundwater is contained by the waterproofing membrane and not managed by an internal tunnel drainage system, the Secondary lining carrying the full hydrostatic loads.

Spray applied waterproofing membranes offer benefits by bonding to both the Primary and Secondary linings. This property offers maintenance and repair benefits in the long term by preventing the movement of water, either behind or, should it be breached, in front of the membrane. Sprayed membranes are not suitable to be applied directly to active water ingress and if this is the case sheet membranes are recommended. The Stepney Green caverns were constructed in both London Clay (sprayed membrane utilised) and Harwich Formation and Lambeth Group (sheet waterproofing membrane utilised).

Due to the presence of structural fibre reinforcement in the Primary concrete linings, a regulating layer (fine aggregate non-fibre reinforced concrete) is applied to the substrate prior to application of the waterproofing layer.

Structural Parameters - The parameters used in the design of the SCL tunnels are presented in Figure 5 below:

Sprayed & Cast In-Situ Concrete	Value
Characteristic compression cylindrical strength of sprayed concrete	32 MPa
Mean Elastic Modulus	33.9 GPa
Coefficient of creep reduction	2
Creep reduced (effective) modulus	16.95 GPa
Steel Bar Reinforcement Parameters	Value
Yield Stress	500 MPa
Elastic Modulus	205 GPa

Figure 5: Structural Parameters

In order to model the beneficial effect of steel fibres in concrete sections under bending, the RILEM TC 162–TDF2 guidelines were used to model the behaviour of concrete under tension. The behaviour of concrete under compression followed Eurocode 2. This resulted in the stress strain diagram shown in Figure 6.

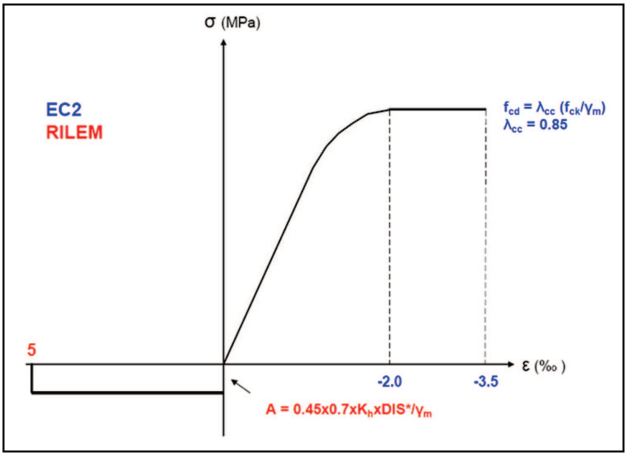


Figure 6 - Stress Strain diagram for SFRS

Where, following RILEM TC162:

- A is flexural tensile stress for ε = 0.0 - 0.5%
- A = 0.45x0.7xKhxD1S*/γm
- Kh (shape factor) = 1.0-0.6x(h-0.125)/0.475
- D1S* is 2.0 MPa at 90 days and 1.8 MPa at 28 days

The following points should be noted:

- The compressive side of the stress strain curve follows Eurocode 2 (Section 3.1.7, Figure 3.3);
- The residual strengths in tension are calculated using a simplification of the RILEM model.
- The residual strength in tension was set as a constant value up to a strain of 0.5%;
- The partial material factor of safety, γm is 1.5 for concrete and fibre-reinforced concrete and 1.15 for steel bars, in accordance with Eurocode 2.

Structural Analysis – The following numerical analyses were undertaken using both FLAC 2-D and 3-D:

- Calibration of 2-D numerical analysis to establish ground models and relaxation parameters.
- Preliminary analyses to confirm the lining shapes, thicknesses and proposed excavation sequences were structurally adequate for the Primary SCL linings.
- Modelling of ground-structure interaction using the finite difference package FLAC was carried out.

A modelling simplification inherent in 2-D modelling is the relaxation of the ground ahead of an excavation face, a largely 3-D effect, and the values used in the 2-D modelling were calibrated against 3-D models and previously constructed tunnels in London Clay, at Heathrow Airport and Kings Cross Station. Time dependent development of sprayed concrete strength and stiffness were included in the models.

In line with London Underground (LU) requirements a surcharge loading of 75kPa was applied to both the Primary and Secondary Lining cases. The train loadings were included in accordance with LU Engineering Standard 1-052 – Civil Engineering – Bridge and Railway Group Standard GC/RT5112.

Excavation Modelling – A FLAC a 3-D extruded model was used to analyse the Stepney Green Caverns. A typical excavation sequence is described below;

The narrower section of the turnout caverns utilised a single sidewall drift construction methodology. The advance lengths were 1 metre for top heading and bench and 2 metres for invert. The following construction sequence was assumed in the analysis;

Sequence Number	Construction Sequence (assumed in design)
01	Construct whole length of left hand side
02	Construct whole length of right hand side

<table>
<tr><th>Part of lining</th><th>Activity</th><th>Advance length (m)</th><th>Duration (hours)</th></tr>
<tr><th colspan="4">(1) Left hand side</th></tr>
<tr><td rowspan="2">TH</td><td>Excavation, scanning, mapping</td><td rowspan="2">1.0</td><td>4.0</td></tr>
<tr><td>Primary lining</td><td>4.0</td></tr>
<tr><td rowspan="2">BCH</td><td>Excavation, scanning, mapping</td><td rowspan="2">1.0</td><td>2.0</td></tr>
<tr><td>Primary lining</td><td>4.0</td></tr>
<tr><td rowspan="2">INV</td><td>Excavation, scanning, mapping</td><td rowspan="2">2.0</td><td>4.0</td></tr>
<tr><td>Primary lining</td><td>6.0</td></tr>
<tr><th colspan="4">(2) Right hand side</th></tr>
<tr><td rowspan="4">TH</td><td>Excavation</td><td rowspan="4">1.0</td><td>4.0</td></tr>
<tr><td>Split 1 initial lining 200mm</td><td>2.0</td></tr>
<tr><td>Sidewall removal</td><td>4.0</td></tr>
<tr><td>Primary lining incl. joint formation</td><td>6.0</td></tr>
<tr><td>BCH</td><td>Excavation incl. removal of sidewalls</td><td>1.0</td><td>6.0</td></tr>
<tr><td rowspan="2">INV</td><td>Excavation incl. removal of sidewalls</td><td rowspan="2">2.0</td><td>8.0</td></tr>
<tr><td>Primary lining incl. joint formation</td><td>8.0</td></tr>
</table>

Advance rates / cycle times

The excavation sequence is shown diagrammatically in Figure 7 taken from the 3-D model output;

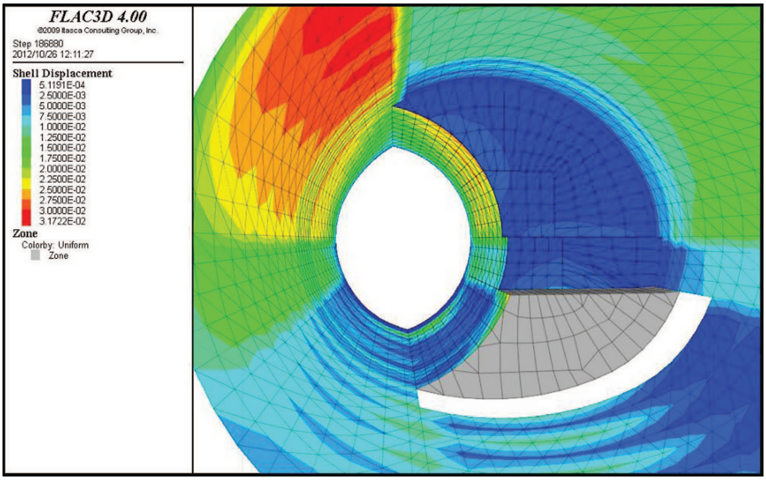


Figure 7 – Excavation Sequence for Single Sidewall Drift – showing Shell Displacement

The calculated volume losses were in the order of 1.0–1.2 %. The maximum predicted displacement for this section was 32mm.

The results of the analysis for long-term condition for the Primary Lining bending moments in the hoop direction are shown in Figure 8.

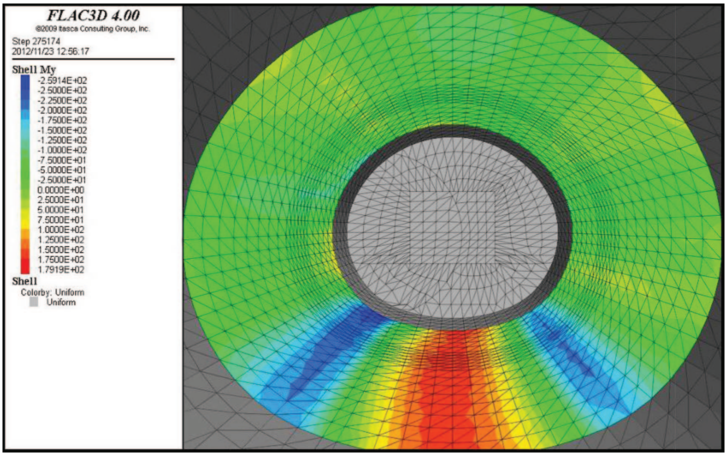
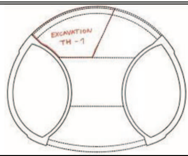
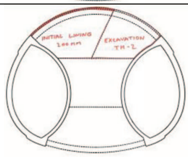
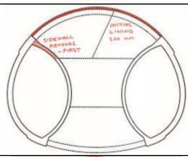
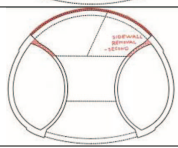
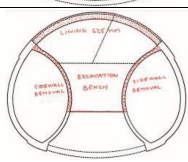
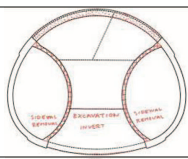
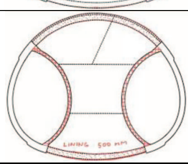


Figure 8 – Bending Moments in Long-term Condition

The red/orange colour in the invert shows tension on the inside face. The green/blue colour shows tension on outside face, with the largest moment typically being at the knees.

The wider section of the turnout caverns utilised a double sidewall drift with central gallery construction methodology. The advance lengths were 1 metre for top heading and bench and

2 metres for invert. The following calculation steps for the central gallery were assumed in the analysis;

Calculation Step	Scheme	Description
Stage 1		Top Heading – Split 1 – Excavation
Stage 2		Top Heading – Split 1 – Initial lining 200mm Top Heading – Split 2 – Excavation
Stage 3		Top Heading – Split 1 – Sidewall removal Top Heading – Split 2 – Initial lining 200mm
Stage 4		Top Heading – Split 2 – Sidewall removal
Stage 5		Top Heading – Primary lining 625mm Bench – Excavation Bench – Sidewall removal
Stage 6 until Stage 10	–	As Stage 1 until Stage 5 (second advance)
Stage 11		Invert – Excavation and Dewatering
Stage 12		Invert – Primary lining 500 mm and again Stage 1 (third advance)

The calculated volume losses were in the order of 1.3–1.4 % for the sidewall drifts and 1.0 for the central gallery. The maximum predicted vertical displacement at cavern crown was 60mm.

Construction of Cavern Primary Linings

Ground Stabilisation and Depressurisation

As identified during the design stage, the main ground hazard during the construction of the SCL works at Stepney Green was the presence of high pore water pressures in the sandy layers of the upper part of the Lambeth Group. In the Eastbound Cavern, which was fully excavated in the stiff clays of the London Clay and Upper Mottled Beds, this pressure could lead to heave of the base and consequently rupture the impermeable clay layer during the invert stage. In the Westbound Cavern, where sand layers were encountered at bench and invert stages, the pore pressures could cause instability of face and invert due to running liquefied sands.

Based on the results of pumping tests, a surface ejector scheme was developed to dewater the upper part of the Lambeth Group. The strategy was to encapsulate the tunnels within a ring of ejectors. A spacing of approximately 8 metres was initially set by WJ Groundwater during their design based on previous pumping test data and the constraints imposed by surface access. These constraints required the use of inclined ejectors to achieve the drawdown outside the site boundaries. The dewatering layout is shown on Figure 9.

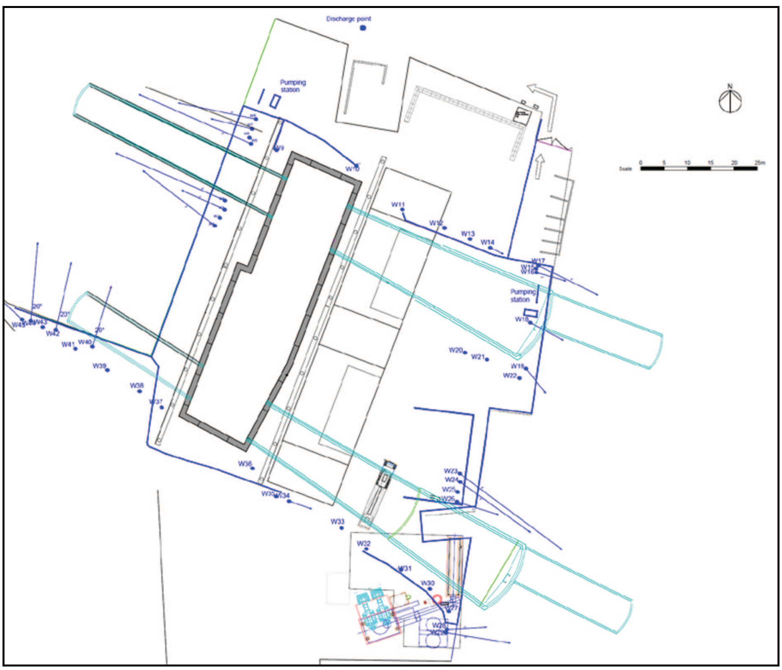


Figure 9 – Layout of Dewatering Scheme

The drawdown achieved by this surface based system was approximately 12m (from 90m to 78m ATD). As the deepest invert excavation level for the Eastbound and Westbound was 79m and 75m ATD respectively it was concluded that an additional in-tunnel dewatering system would be required for the Westbound Cavern excavation.

The nature of the layers to be dewatered (being randomly distributed sand channels with changes in thickness, length and frequency) meant that an intense probing regime was needed to detect these formations. A preliminary design for the in-tunnel dewatering was developed as a starting point, with wells drilled every 4 metres at bench level after the excavation and completion of the primary lining in the invert. The depressurisation worked as a temporary "moving front" of depressurisation with the tunnel wellpoints installed as the excavation progressed. The performance of the tunnel wellpoints was monitored using un-pumped wellpoints as stand pipe piezometers. This allowed the monitoring location and pumped wellpoints to be altered to suite the progress of the works. In addition dedicated piezometers, comprising shallow wellpoints, were installed to just 1 metre below the invert. See Figure 10

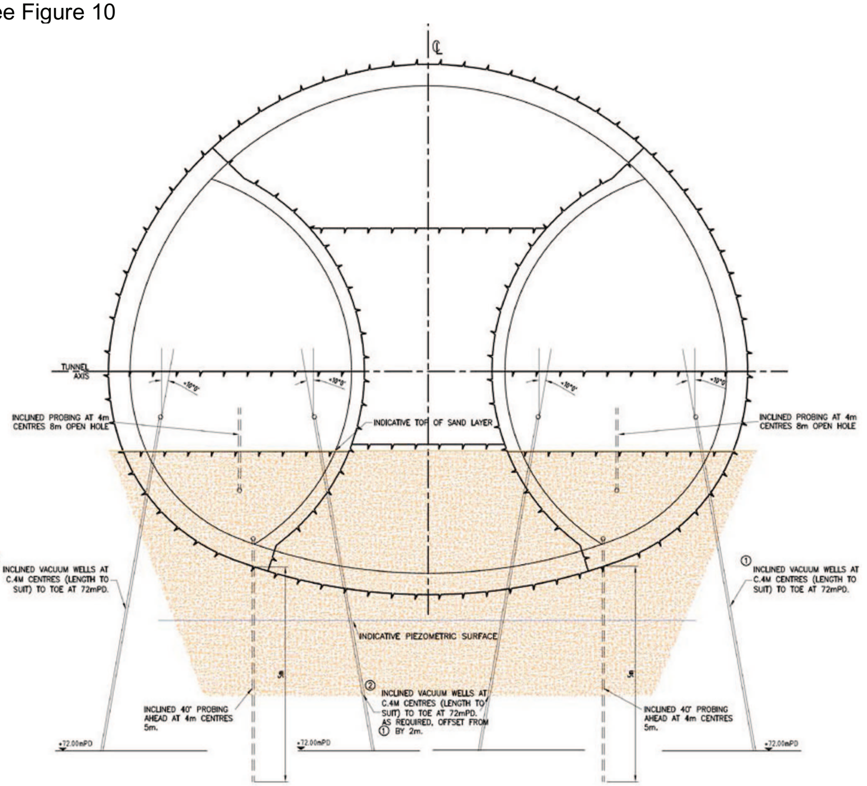


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Figure 10 – Layout of Dewatering Wellpoints

Due to the inconsistent distribution of the sand layers, the initial wellpoint layout and frequency were altered following review of information obtained from the probing, face logs and piezometer monitoring. The selection of screen and filters for the wells was modified to accommodate the changes in permeability of the sand lenses with due consideration to the groundwater abstraction flow rate.

The additional drawdown obtained by the surface wells was sufficient for the excavation of most of the tunnels. However, it was not always possible to achieve a full drawdown at the interface boundary between a sand horizon and underlying clay stratum. Flowing water was sporadically encountered leading to challenging construction conditions when invert excavations were “open”. A permit to dig system was successfully implemented which ensured certain key checks had to be undertaken before excavation of the invert was permitted. The permit was signed by all of the interested parties and the Senior SCL Engineer indicated the contingency measures that had to be employed depending on the water levels and geology.

Excavation Sequence and Ground Support

Three gangs were mobilised in November 2012 to work the 24/7 shift pattern of 7 days on - 3 days off dayshift and a 7-4 nightshift. At peak times, during concurrent construction of both EB and WB Caverns, over 80 staff and operatives worked at site on each shift.

The EB launch tunnel, situated wholly within the relative “comfort” of the London Clay, was excavated first to familiarise the crews and create more working space at the pit bottom for manoeuvring the large SCL plant before breaking out through the diaphragm wall for the EB cavern. The primary excavation and spraying plant were proved during this juncture and bespoke on-the-job training was given to the crews by the suppliers and manufacturers of the plant.

The invert and crown joint connections for the lattice girders and steel reinforcing bars on the first side drift to the adjacent enlargement proved to be particularly challenging to construct. Despite best efforts during preparation of the joints some remedial works involving the breaking back of previously sprayed concrete proved to be necessary. The acute angle of each joint in section profile proved difficult to excavate and spray accurately in the close confines of the side drift. Lessons learnt from the first side drift in the EB tunnel led to a series of improvements in joint preparation that were quickly implemented in the subsequent double side drift section and the WB Cavern.

The following plant was used to construct the SCL caverns:

- Primary excavation of all tunnel elements by a pair of Liebherr 924 compact excavators c/w various hydraulic tools including Wimmer drilling units, crusher pincers, road-header, breaker, claw & various excavating buckets;
- Pair of Liebherr 928 loading shovels for removal of spoil from face into skips at pit bottom;
- Five Utranazz 5.5m3 car-mixers for conveying fresh concrete from site batch plant to column skip on surface and from pit bottom to the face;
- Primary concrete spraying by three Meyco Potenza spraying mobiles (one for maintenance as spare);
- Two Meyco Suprema shotcrete pumps and two Meyco Oruga spraying robots;
- Meyco Piccola and GM dry bag hand spraying equipment;
- L2C Atlas Copco twin boom drilling jumbo for installation of spile bars, vacuum wellpoints, ground probing and pre-split holes within Drive Y tunnel eyes of headwall;
- Combination of RT 4WD Genie boom-lift MEWPs and two tracked boom-lifts for all working at height activities.

Figure 11 – EB Cavern – Removing temporary sidewall of top heading

A dedicated SCL Plant Manager was appointed to ensure that the equipment was utilised as much as possible. Prestart checks, daily maintenance and cleaning of each item of plant by the responsible trained operative were incentivised by the implementation of a production bonus scheme. Preventative maintenance and reactive repairs as necessary were carried out underground by fitters on each shift. Equipment was removed from service for scheduled maintenance by manufacturers and specialist plant technicians.

Design Development during Construction

In order to improve the constructability of the design the following early changes were agreed with the Contractor, CRL and the Designer and implemented on site:

- Bench excavation rounds for side wall and enlargement increased to 2 metres to improve speed of ring closure;
- Lattice girders were removed from all temporary side walls to minimise handling and erection risks and improve quality of the sprayed concrete lining;
- Alternate lattice girders were removed from Primary SCL main outer shell to provide girders at 2 metre centres;
- The P2 layer (comprising bar reinforcement but no steel fibres) for invert, bench and top-heading was constructed directly after the P1 layer invert in 2 metre long sections to minimise mesh lap joints and improve the quality of the sprayed concrete lining;
- The P2 mesh in main outer SCL shell at the crown and invert of the side drifts was left exposed for 1 metre to enable a better lap connection later during the enlargement construction;
- Split face construction and 200mm capping of the initial SCL layer on the top heading in larger spans of cavern central enlargement (i.e. wider sections of the apple core) was replaced by driven steel spile bars and a 75mm initial layer. This facilitated installation of the lattice girders and reduced further construction joints in the P1 layer

Tunnelling Progress

The average tunnelling rates achieved for the tunnel elements were as follows:

Tunnel Element	Sequence of construction	Eastbound (m/day)	Westbound (m/day)
Launch tunnel	1	1.7	0.9*
Single side drift section (Cavern 1)			
LH side drift	2	0.9	1.3*
RH enlargement	3	1.3	1.4*
Double side drift section (Cavern 2)			
LH side drift	4	1.4	0.9*
RH side drift	5	1.1	1.0*
Central enlargement	6	0.6	0.8*
TBM Reception stub tunnels			
Stub tunnels	7	1.5	1.5*

* includes installation of vacuum well-points for groundwater depressurisation

The average cycle times that were achieved for the single side drift sections were as shown below;

Part of lining	Activity	Advance length (m)	Actual Duration (hours)
(1) Left hand side drift			
TH	Excavation, survey, ramping	1.0	1.5
	Primary lining (P1 only)		3.2
BCH	Excavation, survey, ramp removal	2.0	2.5
	Primary lining (P1 only)		2.5
INV	Excavation, survey, backfill out	2.0	3.5
	Primary lining (P1 only)		4.0
P2	P2 layer for INV, BCH & TH	2.0	7.5
(2) Right hand enlargement			
TH	Excavation, survey, ramping & removal of sidewall	1.0	2.0
	Primary lining (P1 only)		2.7
BCH	Excavation, survey, ramp & sidewall removal	2.0	2.8
	Primary lining (P1 only)		2.2
INV	Excavation, survey, backfill & sidewall removal	2.0	3.2
	Primary lining (P1 only)		3.2
P2	P2 layer for INV, BCH & TH	2.0	6.5

Figure 12 – WB Cavern - Installing Lattice Girders in TH of Central Enlargement

Generally the SCL caverns were constructed faster than expected. It is considered that this was attributed mainly to:

- High motivation of work crews (mainly due to bonus pay);
- Design development during construction;
- Synergy and collaborative working between DSJV, OtB, Crossrail, C121 site teams and Designer as evidenced during the effective daily SRG and weekly CTC meetings;
- Good care, maintenance & rotation of equipment;
- Implementation and continual improvement of effective ground dewatering systems;
- Consistent supply of fresh quality concrete from the Hanson on-site batching plant;

Lessons Learnt

The main lessons learnt, relating to health and safety, have been identified as follows:

- Minimise manual hands-on work where personnel must enter the exclusion zone at the face (e.g. for joint preparation and installation of steel continuity bars, lattice girder connections, survey instrumentation);
- Minimise the duration of time that people have to be within the exclusion zone at the face for each round;
- Promote fully mechanised work where possible for joint preparation, excavation & spraying. Selection of plant and tools is an important contributing factor when planning and designing the works. Consider size, weight, power, gradeability, reach, emissions and engine fire suppression. Where possible time should be allowed for trialling each item of plant;
- Provided good shape control can be maintained by other means, remove lattice girders from the design due to the various manual handling, working at height and falling objects risks associated with their transport, handling, hoisting and erection at the face. They slow ring closure, increase potential for voids in the lining at the butt

plate connections and their fixed position pre-determines face size and round length leading to an inflexible design when considering face support. Due to the flaring and changing shape of the cavern each girder had to be uniquely designed, fabricated and installed in pre-determined position.

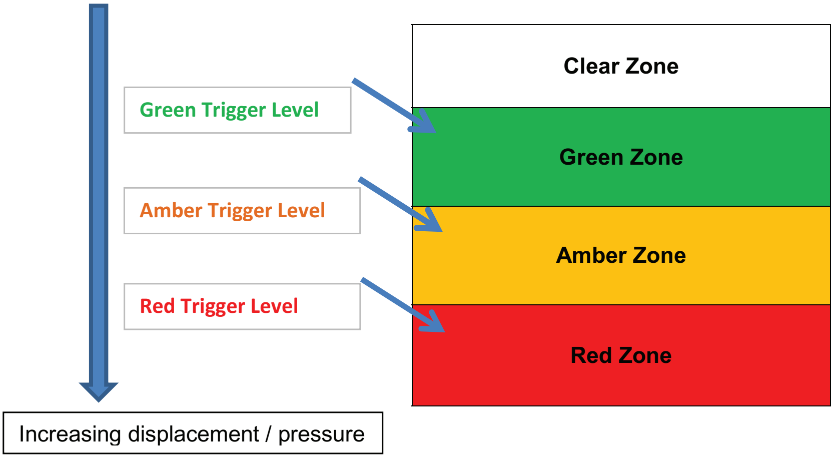
- Minimise the use of steel bars and mesh by utilising steel fibres where possible. The bars tended to increase potential for shadowing and voids when spraying mechanically using robots;
- Avoid tight corners in the SCL profile (e.g. where the temporary sidewall meets the main outer lining shell in the crown and invert) as they are difficult to excavate and spray accurately using mechanised means;
- Use 2 metre advances for bench and invert as they create better access for excavation and spraying, minimise handling of backfill for ramping and generally enable quicker ground support and reduced ring closure time;
- Minimise construction joints that require preparation, rework and inspection;
- Remove pressure cell instrumentation from the lining as it takes time to install and read, and the results proved to be inconsistent. Other means such as strain monitoring should be considered to supplement the deformation monitoring.

Figure 13 – Removing Backfill from Stepney Green Cavern

SCL Performance

Instrumentation & Monitoring Plan

In line with the CEDS Instrumentation & Monitoring Standard; Green, Amber & Red zones were identified as below;



The definition of the zones and trigger levels with associated actions are described below;

- **Clear Zone** – the zone when monitoring was initiated, prior to any triggers being reached. No actions.
- **Green Trigger Level** – this trigger alerted that the Green Zone had been reached. The Green Trigger level was set at 75% of the FLAC calculated lining displacement. This took account of an estimated 25% lining displacement occurring prior to installation and first reading of a monitoring point. The action was to review instrumentation and increase frequency if necessary and agreed at the SRG.
- **Amber Trigger Level** – this trigger alerted that the Amber Zone had been reached. The Amber Trigger level was set at 105% of FLAC calculated lining displacement. The action was to inspect the area of the relevant tunnel and review by the ERP both within 24 hours. The ERP could recommend changes to monitoring and construction methodology.
- **Red Trigger Level** – this trigger alerted that the Red Zone had been reached. The Red Trigger level was set at 150% of FLAC calculated lining displacement. The action was to stop the tunnelling process and make the excavated face safe pending an Emergency Engineering Review within 4 hours.

Predicted vs. Actual Lining Displacements

Given the variable diameter of the cavern, the Designer provided two sections for trigger level monitoring points for the caverns. In each section each monitoring point had its own set of trigger values for both horizontal and vertical deformation.

In general terms, it was observed that the vertical component was similar to the expected one (between 90% and 105% of the amber trigger value), whereas the horizontal component was less than expected (between 30% and 70% of the amber trigger value).

The following tables summarise the green, amber and red trigger values and the average values obtained;

	SINGLE SIDEDRIFT SECTION							
	Actual	Horizontal			Actual	Vertical		
CB1	-0.8	10	14	20	-25.8	-21	-29	-41
CB2	-0.6	11	15	22	-19.4	-19	-26	-38
CB3	-0.5	14	19	27	-15.3	-13	-18	-26
CB4	-1.4	9	13	18	-8.6	-6	-8	-11
CB8	0.6	5	7	10	-18.9	-11	-15	-21
CB9	2.0	7	9	13	-14.1	-9	-13	-18
CB10	3.0	6	8	12	-7.2	-5	-7	-10

	DOUBLE SIDEDRIFT SECTION							
	Actual	Horizontal			Actual	Vertical		
CB15	-				-24.0	-13	-19	-27
CB1	-5.4	5	8	11	-20.3	-11	-15	-21
CB2	-7.3	16	22	31	-24.4	-21	-29	-42
CB3	-4.0	17	23	33	-17.2	-16	-23	-32
CB4	-9.6	9	13	19	-13.8	-9	-13	-18
CB8	2.7	5	7	11	-21.1	-10	-14	-20
CB9	7.4	10	14	20	-23.1	-18	-25	-36
CB10	4.1	14	20	28	-21.0	-15	-21	-30
CB11	4.8	8	12	17	-13.8	-8	-11	-16

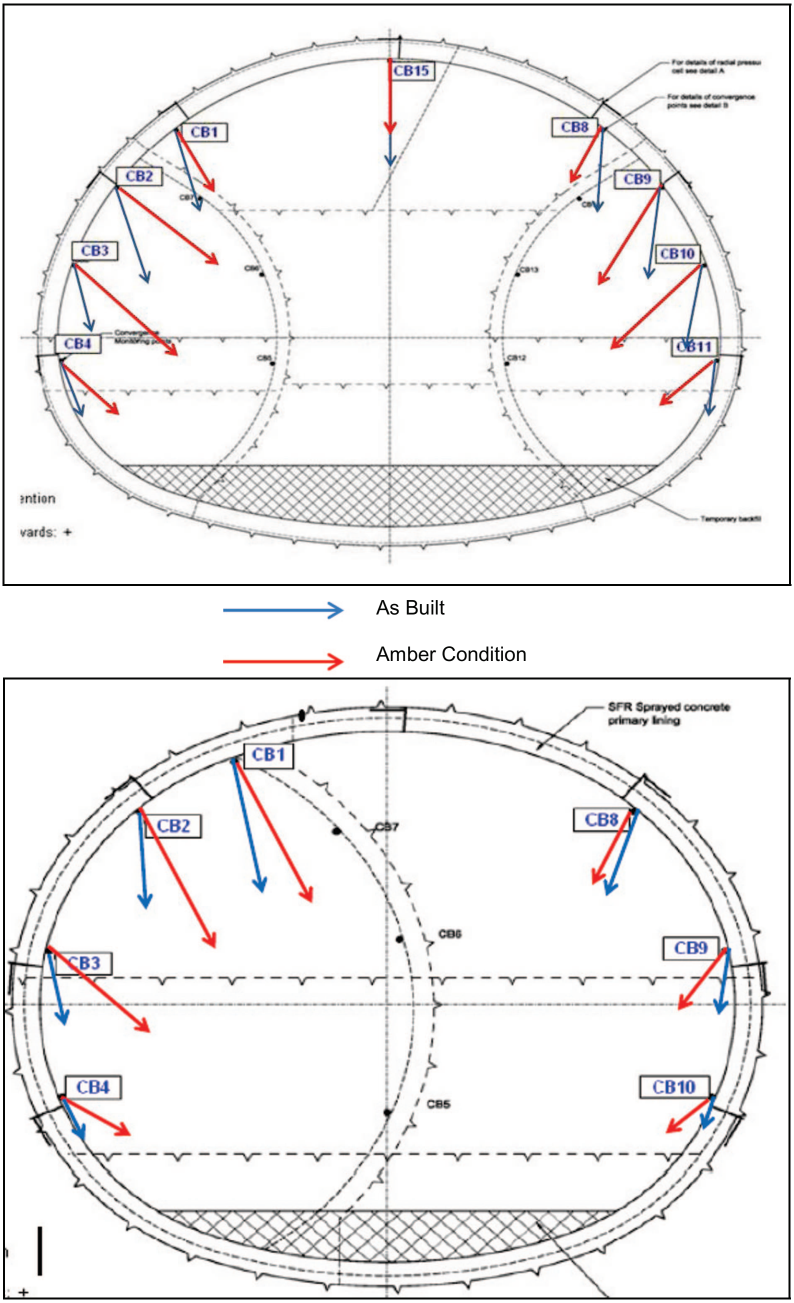


Figure 14 – The expected movement in amber condition and average as-built movement for the single and double side-drift sections.

The discrepancy between vertical and horizontal is considered to be due to the effect of the dewatering that was carried out in the sandy layers beneath the tunnel invert, causing whole-body downward movement of the cavern shell.

Surface settlement

Surface movement was monitored with several arrays of precise levelling points. Readings were taken on a daily basis. Maximum settlement was 60mm above the largest section of the Eastbound Cavern. This value includes the movement due to the SCL works and dewatering. From the analysis of the timeline in settlement, a value of approximately 10mm was determined to be related to the depressurisation and 50mm due to SCL works. The predicted surface settlement, assuming a volume loss of 1.5% was 62mm.

Above the Westbound Cavern the maximum recorded settlement was 51mm, approximately 40mm due to SCL works and 10mm due to depressurisation. The predicted surface settlement for the Westbound Cavern was 30mm.

Conclusions

Design and construction of the Stepney Green SCL Caverns has demonstrated that it is possible to construct large caverns successfully in London Clay and the Lambeth Group, with structure and ground movements controlled to acceptable levels. This was done with a collaborative team effort between Client, Contractor and Designer. There are four further caverns to be constructed on Crossrail and key lessons learnt from Stepney Green can be applied to these structures.

References

1) BTS/ABI 2003 The joint Code of Practice (JCoP) for Risk Management of Tunnel Works in the UK. Joint publication by the Association of British Insurers & the British Tunnelling Society.
2) Technical Specification for Interoperability – Safety in Railway Tunnels TSI-SRT.
3) RILEM TC162-TDF; Test and design methods for steel fibre reinforced concrete. Vol 36, October 2003.
4) LU Engineering Standard 1-052 – Civil Engineering – Bridge and Railway Group Standard GC/RT5112

5 DISCUSIÓN

A continuación, se aborda una breve discusión sobre los resultados de los artículos científicos publicados en revistas de impacto que componen este trabajo de Tesis Doctoral, así como de lo que representan en el avance del conocimiento.

5.1 Ingeniería Geológica y túneles en la península de Limmo, Este de Londres

(Título original en inglés: Engineering geology and tunnelling in the Limmo Peninsula, East London)

En este artículo se analiza la información geológica y geotécnica obtenida en la campaña de investigación y durante la fase de excavación de los túneles en la Península de Limmo (Londres). Este espacio corresponde a la llanura de inundación situada al este del río Lee y su carácter peninsular se debe, precisamente, a que en este tramo el cauce hace un cambio brusco de dirección Sur a Oeste y propicia la existencia de dos orillas que forman un ángulo recto.

Esta península constituyó el punto de acceso de las tuneladoras para la construcción de la sección Este del túnel de la nueva línea de Crossrail. Para ello, se excavaron dos pozos y una serie de túneles con sostenimiento de hormigón proyectado para la interconexión entre los pozos y para el lanzamiento de las tuneladoras. El emboquille de los mismos se había proyectado inicialmente desde los pozos, dentro de la formación Arcillas de Londres.

Durante la campaña de investigación geotécnica, numerosos sondeos confirmaron la existencia de diferencias significativas en la profundidad a la que se alcanzaban las diferentes formaciones y niveles estratigráficos de referencia. Por otra parte, el perfil de la presión intersticial de este sector, al contrario que en el resto de Londres, muestra un comportamiento hidrostático similar a los del Grupo Chalk o Arenas Thanet, observándose durante el bombeo del acuífero principal cómo el descenso del nivel del piezómetro no era uniforme. Esto permitió apuntar a la existencia de una serie de fallas con un rumbo NO-SE que podrían interceptar a los túneles, así como otras de rumbo E-O situadas más hacia el Sur. La excavación de uno de los pozos permitió observar que las Arcillas de Londres estaban más fisuradas de lo habitual, estando presentes diversas familias de diaclasas con una distribución de orientaciones bastante dispersa, si bien se identificaron dos familias ONO-ESE y otra NE-SO, ambas subverticales. Fookes y Parrish (1969) indicaron que puede existir una relación entre la orientación principal del régimen de esfuerzos tectónicos contemporáneos y pasado.

Durante la excavación de los túneles se confirmó la existencia de dos fallas principales, la primera, identificada por observación directa de una banda de arcilla, brechificada, con orientación 070/15 y la segunda, inferida por un cambio brusco del miembro de las Arcillas de Londres A3 al A2. La información sobre estas estructuras se interpreta como el resultado de la reactivación de las fallas de cuenca de tipo *pull-apart* que afectaban a Chalk, generándose una estructura tipo flor. Este modelo de tipo extensional como estructuras en flor negativas y positivas ya había sido propuesto por Cosgrove y Ghail (2010) para explicar algunas de las estructuras geomorfológicas de Londres. Aldiss (2013) sugirió que los mapas del BGS, históricamente, habían infrarrepresentado las fallas en Londres. Dado que esta localidad se encuentra al norte de un tramo de cauce del rio Támesis, en el que se sitúa una de las depresiones rellenas por sedimentos cuaternarios descritas por Berry (1979), y en la confluencia del Támesis, éstas confirman la hipótesis de Freitas (2009) sobre la red de drenaje como manifestación en superficie de la compartimentación tectónica de la Cuenca de Londres.

Desde el punto de vista geotécnico, según los resultados de los ensayos las Arcillas de Londres, en esta zona afectada por estas fallas se constata que el límite inferior del rango de valores de la resistencia al corte sin drenaje (S_u) presenta unos valores más bajos que el resto del proyecto. En concreto, los valores mínimos mostraban una función: $S_u = 20+3z$ (kPa), mientras que en el resto del proyecto la función era: $S_u = 20+5z$ (kPa). En el sector Oeste, se constataron valores aún más altos, con una función: $S_u = 40+3.33z$ (kPa). Este hecho demuestra que las Arcillas de Londres evidencian un incremento en el grado de fisuración en zonas afectadas por fallas resultando en unos parámetros resistentes más deficientes.

En cuanto al comportamiento durante las obras, los frentes de excavación fueron predominantemente estables. Sin embargo, ocasionalmente se observaron desprendimientos de dos tipos: (i) ligados a inestabilidades de tipo cuña en los avances y planares en la destroza, en las que las Arcillas de Londres se comportan como una roca competente; (ii) por deslizamientos de tipo rotacional en las destrozas, en el que las Arcillas de Londres evidencian un comportamiento de tipo suelo mostrando inestabilidad a corto plazo. Estos incidentes, aunque de pequeña importancia para el túnel como estructura, pudieron haber ocasionado la ocurrencia de accidentes laborales de carácter grave. Por ello, como medida de mitigación del riesgo se aplicó en los frentes de excavación un machón central y se abatió la inclinación de la fase de destroza.

Por tanto, la excavación de túneles de Arcillas de Londres afectadas por fallas, es factible, siempre y cuando se tome en consideración que (i) el diseño de del sostenimiento tenga en cuenta las propiedades resistentes de la arcilla más fisurada, y (ii) la secuencia de excavación y de aplicación del sostenimiento se planifiquen de forma que el riesgo de inestabilidades del frente y de los bancos de excavación sea mitigado.

5.2 Drenaje para la excavación de la caverna de Stepney Green.

(Título original en inglés: Depressurisation for the excavation of Stepney Green cavern)

En el proyecto de la nueva línea de Crossrail en Londres, Stepney Green representa la zona de confluencia de los túneles del ramal norte procedente de Pudding Mill Lane y el ramal de sur de Woolwich (Figura 31), acomodando la transición de una sección de 4 a 2 túneles. Además, era una zona de tránsito para las tuneladoras, por lo que se encontraba dentro de una fase crítica del programa de obra. Para ello, se diseñaron dos cavernas de 50 m de longitud, con morfología troncocónica, que llegan a unas dimensiones máximas de 17 m de anchura por 15 m de altura, en lo que suponía hasta entonces la mayor sección de túnel excavada en Londres. Para acceder a los emboquilles se construyó un pozo al amparo de un muro pantalla.

La caverna de los túneles de sentido Este se excavó en las Arcillas de Londres sin especiales complicaciones, a lo que ayudó la baja permeabilidad de esta formación. En la caverna de los túneles de sentido Oeste, con un eje a una cota 5 m inferior, el techo del Grupo Lambeth coincidía aproximadamente con el eje horizontal de la caverna. Por debajo de las Arcillas de Londres, ocupando la mitad inferior del frente de excavación, aparecían las en las arcillas abigarradas de la Formación Reading del Grupo Lambeth. Durante los sondeos para la redacción del proyecto se detectó la presencia de varias capas de arenas limosas a cota de excavación, de espesor variable hasta un máximo de 4 m y cuyo comportamiento hidrogeológico se asimilaba a acuíferos colgados y confinados, con presiones intersticiales de hasta 150 kPa. Adicionalmente, como parte de la investigación se realizó un ensayo de bombeo para profundizar en la caracterización hidrogeológica.

Con los resultados obtenidos: (i) se confirmó que las capas de arenas eran bastante continuas y que mostraban cierta continuidad hidráulica al registrarse un descenso del nivel del agua en los piezómetros instalados en los sondeos; (ii) se estimaron sus parámetros hidráulicos, con una permeabilidad de entre $1x10^{-4}$ a $6\ x10^{-5}$ m/s y un coeficiente de almacenamiento de entre $7x10^{-4}$ y $6x10^{-3}$; (iii) se constató que la rebaja del nivel freático no ocasionó asientos en las edificaciones más próximas.

Con esta información se estableció la necesidad de adoptar medidas para evitar inestabilidades durante la excavación que pudieran afectar a la integridad de la estructura y de las edificaciones cercanas, por entradas descontroladas de arena y agua en la fase de excavación. Dado el resultado del ensayo de bombeo, se decidió abordar la excavación mediante la instalación de un sistema de eyectores desde la superficie que permitiera drenar los paquetes de arena. En total se perforaron de 45 pozos con eyectores hasta una cota inferior a la de contrabóveda de las cavernas. Algunos de los eyectores se instalaron en pozos inclinados ya que no era posible perforar desde la proyección vertical de las cavernas por restricciones de acceso. Estos equipos extraen agua por la generación de presiones negativas mediante el efecto Venturi al inyectar agua a gran presión (>750 kPa) y están indicados para materiales limosos (Preene *et al.*, 2000). La eficacia de este sistema se monitorizo con una red de piezómetros, adicional a los ya instalados para la fase de diseño, que permitió obtener medidas en tiempo real. La instalación de estos equipos en varios niveles en el mismo sondeo optimizó el aprovechamiento el limitado espacio disponible para la perforación.

Tras la conexión de los eyectores, los piezómetros registraron un descenso medio del nivel freático de 14 m, sin embargo, algunos de ellos indicaban presiones de poro no adecuadas para excavaciones de frente abierto. Consiguientemente, se decidió implementar un sistema de adicional drenaje dentro del túnel mediante un sistema de pozos de bombeo (*well-points*) conectados a una bomba de vacío situada en el interior del túnel; para ello se aprovecharon los equipos disponibles y no se requirió una sonda especial. Los taladros de investigación perforados desde el emboquille del túnel confirmaron que las capas de arenas se disponían en lentes, en especial hacia el nivel de contrabóveda, por lo que los pozos fueron perforados subverticalmente en la parte inferior de los hastiales de los túneles a medida que se avanzaba con la excavación. Este sistema funcionaria como un "frente de drenaje" provisional.

El sistema subterráneo se modificó numerosas veces para adaptarse al número de pozos, la profundidad de las capas de arenas, a la distancia del frente de excavación con las bombas y a la granulometría de las capas de arenas finas que pueden requerir cambios en el diámetro de las tuberías y el tipo de filtro. Ambos sistemas, superficial y subterráneo, precisaban de una gran labor de protección y de mantenimiento. El sistema de *well-points* implementado en el túnel requirió numerosas operaciones de mantenimiento, ya que el constante tránsito de equipos provocaba roturas en las tuberías y por tanto pérdida del vacío y parada de las bombas. En superficie, las tuberías de los pozos con eyectores se debían extraer y limpiar con frecuencia ya que la presencia de la bacteria *Gallionella ferruginea* formaba concreciones férricas que obturaban el sistema. Como sistema de contingencia el caso de fallos mecánicos o problemas con el suministro eléctrico se creó una redundancia del sistema de bombeo. Finalmente, un sistema de monitorización en tiempo real alertaba de cualquier desconexión de las bombas o de subidas del nivel piezométrico para activar un sistema de contingencia y de sellado del frente.

Las cavernas de Stepney Green demostraron que es posible la excavación de túneles de gran diámetro a frente abierto de los materiales arenosos del Grupo Lambeth, siempre y cuando se efectúe una adecuada reducción de la presión de poro. Es recomendable aplicar una estrategia secuencial, comenzando por: (i) una primera fase de drenaje desde la superficie utilizando un sistema de eyectores, adecuados para materiales limosos-arenosos y que no requieran grandes caudales y (ii) una siguiente fase de bombeo con *well-points* conectados a una bomba de vacío que permita efectuar un drenaje de detalle, diseñado a partir de los taladros de investigación. Asimismo, se debe prever que el rendimiento de la excavación disminuya por lo que el programa de obra debe considerar una duración mayor de los trabajos, dado el tiempo destinado a las labores de instalación y mantenimiento.

El caso de estudio de Stepney Green muestra también que la implementación de un sistema de drenaje activo requiere un mantenimiento intensivo y una monitorización exhaustiva y permanente del sistema para anticipar posibles problemas y adaptarse a los cambios geológicos.

5.3 Desarrollo de un modelo del terreno, investigación geológica y mitigación de riesgos para la excavación con frente abierto de una galería en el proyecto subterráneo de la línea Elizabeth.

(Título original en inglés: Development of a ground model, targeted ground investigation and risk mitigation for the excavation of an open face cross passage on the underground Elizabeth Line, London)

Los dos túneles principales que componen la sección subterránea de la línea Elizabeth (Crossrail) requieren, por razones de seguridad, su interconexión mediante galerías transversales. Estas estructuras tienen un diámetro de 4,3 m y su excavación implica dos riesgos: el constructivo, ya que se excavan con técnicas tradicionales a frente abierto dado su reducido diámetro, y el riesgo de retrasos, ya que las galerías se encuentran en el tramo crítico del proyecto al tratarse de los últimos frente de excavación.

La galería CP-6 (*Cross Passage*) se sitúa entre Liverpool Street y Stepney Green en el Este de Londres, a unos 40 m de profundidad, tiene una longitud de 14 m e incluye un sumidero central a 48 m debajo de la superficie. Para el diseño de CP6, la principal fuente de información geotécnica provenía de un único sondeo LW9, perforado a unos 11 m del eje de la galería, cuya testificación geotécnica indicaba la presencia de arcillas rígidas abigarradas de la Formación Reading del Grupo Lambeth a la cota de excavación de la galería y algunos niveles de gravas pertenecientes a la Formación Upnor en el sumidero.

Debido a la conocida variabilidad del Grupo Lambeth, durante la perforación de los túneles principales, dirección Este y Oeste, se estudió la granulometría en varias muestras del material excavado por las tuneladoras a la altura de la galería CP-6. Las curvas granulométricas de las muestras de ambas tuneladoras mostraban la existencia de una fracción arenosa (diámetro >0,02 mm) en una proporción variable y que era indicativa de la posible presencia de una o varias capas de arena en la sección de excavación de la galería, consecuentemente contradiciendo el modelo geológico creado a partir del sondeo LW9. Para poder corroborar la testificación, se decidió realizar unos ensayos de permeabilidad en el sondeo. La permeabilidad estimada en las cotas equivalente al metro superior de la excavación, en un tramo descrito como "*arcilla*" fue $2{,}1\times10^{-5}$ m/s, típica de un limo arenosos (Lewis et al., 2006). Esto sugería la existencia de errores en la descripción de los testigos, pudiendo haber pasado inadvertidas capas de arenas interestratificadas. Este hecho confirma que en medios urbanos, con pocas oportunidades de realizar investigaciones intrusivas, éstas deben de ser de máxima calidad para evitar problemas en la validez de la información, máxime cuando se exploren formaciones con frecuentes cambios laterales de facies, tales como el Grupo Lambeth.

Tras la instalación los anillos del túnel perforado con tuneladora, en el sector situado al Este de la ubicación de la galería CP-6 se realizaron una serie de perforaciones a través de los puertos de inyección en las dovelas, empleando un taladro de mano de 32 mm, con el objetivo de muestrear el terreno situado a trasdós del revestimiento; esto permitiría confirmar los límites entre las diferentes unidades y detectar la presencia de posibles capas de arena saturadas en agua. Mediante estas investigaciones se detectó la presencia de una capa de arena de un 1 m de espesor a cota de la clave de la galería. En los taladros del túnel Oeste, esta unidad de arena ocupaba prácticamente la mitad superior de la sección. Además, a través de la instalación de manómetros de presión su pudo determinar una presión piezométrica máxima de un 1 bar a la cota de contrabóveda de la galería.

Estas investigaciones realizadas en fase de obra permitieron anticipar un cambio en las condiciones del terreno sin necesidad de detener el avance de las tuneladoras y de retrasar el plazo de ejecución. Con esta información se diseñó un drenaje activo para la galería mediante *well-points* conectados a una bomba de vacío. La perforación de estos pozos tuvo dificultades iniciales debido al carácter sub-artesiano de las arenas ($\leq$1 bar). Por ello se optó por una estrategia de conexión gradual de unos pozos cortos, situados a mayor distancia de la galería para ir reduciendo la presión hasta que fuera posible perforar hasta la profundidad requerida, a cota de contrabóveda.

El rebaje del nivel del agua se fue monitorizando mediante varios piezómetros, que se mantuvieron operativos durante la excavación para detectar posibles subidas repentinas debido a fallos en el sistema o en los equipos. Se instalaron, asimismo, alarmas sonoras conectadas a las bombas para alertar en caso de desconexiones de las mismas y que se procediera a cerrar el frente mediante madera.

Este sistema resultó exitoso, ya que cuando se comenzó la excavación, tras la retirada de las dovelas las arenas se encontraban secas y se pudo realizar la excavación en las condiciones deseadas. Sin embargo, en ausencia de succión, se produjeron desprendimientos en la clave de la perforación que llegaron a alcanzar la base de las Arcillas de Londres, lo cual retrasó el avance de la excavación ya que requirió las inyecciones de lechada para rellenar el hueco creado. Este aspecto puede convertirse en un problema de consideración en aquellos casos donde la potencia de las arenas sea importante.

Durante la excavación se caracterizaron granulométricamente las arenas, constatándose su variabilidad, lo que se traduce en la importante oscilación que demuestran los valores de permeabilidad. Este hecho aconseja que el diseño de estos sistemas de drenaje activo en capas de arenas del Grupo Lambeth se lleve a cabo mediante métodos empíricos, debiendo ser modificado de acuerdo al desarrollo de las obras.

Otro importante factor a tener en cuenta es el asiento motivado por la consolidación relacionada con la reducción de la presión efectiva de las arenas. En las secciones de auscultación se registró un incremento del asiento máximo de 15 mm, del que se estima que aproximadamente 5 mm se deben a la consolidación de las arenas. Estos valores suponen un asiento de 1 mm por cada metro de rebaje del nivel freático, perfectamente admisibles por cualquier estructura. Sin embargo, en otras condiciones, donde el espesor de las arenas sea mayor y la compactación sea mayor, resulta necesario evaluar el riesgo de asientos superiores a los admisibles.

6 INFORME DE IMPACTO DE LAS PUBLICACIONES

6.1 Publicaciones SCI

Artículo: Engineering geology and tunnelling in the Limmo Peninsula, East London

Revista: Quarterly Journal of Engineering Geology and Hydrogeology

DOI: https://doi.org/10.1144/qjegh2016-116

ISSN: 1470-9236

Año publicación: 2017

Datos sobre la revista:

Año	Factor impacto	Cuartil	Índice inmediatez	Vida media citas	Factor impacto 5 años	Ranking categoría
CATEGORÍA: GEOSCIENCES, MULTIDISCIPLINARY						
2015	1,058	Q4	0,045	>10	0,989	141/184
2016	1,102	Q3	0,273	>10	1,036	138/188
2017	0,818	Q4	0,4	>10	1,163	168/189
2018	1,171	Q4	0,727	10,3	1,263	158/196
CATEGORÍA: ENGINEERING, GEOLOGICAL						
2015	1,058	Q3	0,045	>10	0,989	21/35
2016	1,102	Q3	0,273	>10	1,036	22/35
2017	0,818	Q4	0,4	>10	1,163	32/36
2018	1,171	Q4	0,727	10,3	1,263	31/38

Artículo: Depressurisation for the excavation of Stepney Green cavern

Revista: Proceedings of the Institution of Civil Engineers - Geotechnical Engineering

DOI: https://www.icevirtuallibrary.com/doi/10.1680/geng.14.00061

ISSN: 1353-2618

Año publicación: 2015

Datos sobre la revista:

Año	Factor impacto	Cuartil	Índice inmediatez	Vida media citas	Factor impacto 5 años	Ranking categoría
CATEGORÍA: GEOSCIENCES, MULTIDISCIPLINARY						
2015	0,577	Q4	0,93	7,7	0,6	32/35
2016	0,544	Q4	1,333	7,7	0,686	31/35
2017	0,906	Q4	1,429	8,2	1,034	30/36
2018	1,274	Q4	1,455	8,1	1,219	30/38
CATEGORÍA: ENGINEERING, GEOLOGICAL						
2015	0,577	Q4	0,93	7,7	0,00067	171/184
2016	0,544	Q4	1,333	7,7	0,00059	178/188
2017	0,906	Q4	1,429	8,2	0,001	163/189
2018	1,274	Q4	1,455	8,1	0,00104	152/196

Artículo: Development of a ground model, targeted ground investigation and risk mitigation for the excavation of an open face cross passage on the underground Elizabeth Line, London

Revista: Tunnelling and Underground Space Technology

DOI: https://doi.org/10.1016/j.tust.2019.01.004

ISSN: 0886-7798

Año publicación: 2019

Datos sobre la revista:

Año	Factor impacto	Cuartil	Índice inmediatez	Vida media citas	Factor impacto 5 años	Ranking categoría
CATEGORÍA: CONSTRUCTION & BUILDING TECHNOLOGY						
2015	1,741	Q1	0,239	6,7	2,005	15/61
2016	2,192	Q1	0,678	6,9	2,562	15/61
2017	2,418	Q2	0,62	6,3	2,822	16/62
2018	3,942	Q1	1,029	5,9	4,356	10/63
CATEGORÍA: ENGINEERING, CIVIL						
2015	1,741	Q2	0,239	6,7	2,005	32/126
2016	2,192	Q1	0,678	6,9	2,562	28/125
2017	2,418	Q1	0,62	6,3	2,822	27/128
2018	3,942	Q1	1,029	5,9	4,356	10/132

6.2 Otras Publicaciones

Artículo: Crossrail Sprayed Concrete Lining Depressurisation at Stepney Green Caverns

Libro: Crossrail Project: Infrastructure design and construction.

DOI: http://doi.org/10.1680/cpid.60784.385

ISBN: 0978-0-7277-6078-7

Año publicación: 2015

Editorial: ICE publishing

Artículo: Multi-aquifer pressure relief in east London

Libro: Proceedings of the XVI ECSMGE Geotechnical Engineering for Infrastructure and Development

DOI: https://doi.org/10.1680/ecsmge.60678

ISBN: 978-0-7277-6067-8

Año publicación: 2015

Editorial: ICE publishing

Indexado: Scopus

7 CONCLUSIONES

Este trabajo de Tesis Doctoral ha abordado la investigación de dos de las principales incertidumbres para la perforación de túneles urbanos en Londres, cuya ejecución ha estado especialmente condicionada por factores geológicos: las características geológicas y geotécnicas del Grupo Lambeth y la presencia de fallas no conocidas en toda la cuenca. Para ello se han analizado tres casos reales de túneles excavados con frente abierto que potencialmente podrían ser afectados por estos problemas geotécnicos.

Como principales aportaciones de este trabajo de Tesis Doctoral, se pueden establecer las siguientes:

- La investigación geotécnica e hidrogeológica en el sector la península de Limmo, a la que se suman los datos obtenidos durante las excavaciones para diferentes estructuras subterráneas en esta zona, confirman la presencia de un sistema de fallas que afectan a los materiales del Paleógeno. Se interpreta que estas estructuras son resultado de una inversión tectónica de una cuenca *pull-apart* transformada en una estructura de flor positiva, lo que resulta consecuente con las hipótesis previas que apuntaban a una compartimentación de la cuenca, que se manifiesta en superficie a través de la red de drenaje.

- En la Formación Arcillas de Londres se produce un aumento de su fisuración en el entorno de las fallas reduciéndose, en estas zonas, de forma significativa los parámetros resistentes representativos de estos materiales. Esto conlleva una problemática asociada en las obras subterráneas, constatada en los frentes de excavación y en las destrozas, si bien no condiciona su integridad estructural.

- Afrontar la excavación de capas de arenas saturadas del Grupo Lambeth en frente abierto en cualquier diámetro de excavación (cavernas y galerías) es viable siempre que se adopten medidas de drenaje activo de las mismas. Debido al reducido caudal de bombeo requerido y de la distribución espacial en capas lenticulares, es necesario utilizar eyectores si se desea hacer desde superficie. La capacidad de realizar un bombeo desde el frente de excavación está limitada por la presión sub artesiana, siendo recomendable reducir esta presión a valores menores de 1 bar.

- El drenaje de las capas de arenas del Grupo Lambeth puede ocasionar asientos en superficie, cuya magnitud depende del espesor de la capa drenada, de su módulo de deformación y de su permeabilidad, aspectos a fijar en futuros proyectos. En los casos analizados se documentaron asientos reducidos, de 1mm/1m de rebaje, que no comprometen la seguridad de las estructuras superficiales.

- La utilización de sistemas de bombeo para la excavación de túneles precisa de un mantenimiento intensivo para evitar ascensos repentinos del nivel freático debido a pérdidas de presión por fugas o problemas de suministro eléctrico, siendo aconsejable disponer de sistemas redundantes y de monitorización.

- Se constata la importancia de actualizar los modelos geotécnicos correspondientes a proyectos de obras subterráneas deben de actualizarse permanentemente con la información obtenida durante el transcurso de las obras. Este aspecto ha quedado ejemplificado la mejora del modelo geológico a partir del uso de datos granulométricos y los proporcionados por los taladros de prospección y los manómetros, sin necesidad de detener el avance de la tuneladora o de acceder al frente de excavación en condiciones hiperbáricas.

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