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Title: **Three-dimensional interaction of twin tunnels: numerical analysis of the Waterloo International Terminal case study**

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Abstract

Being able to predict with precision and certainty how existing tunnels respond to new tunnelling works in urban areas is vital for the safety of the existing tunnels and for minimising the cost and environmental impact of the new tunnels. The three-dimensional interaction of tunnels in stiff, overconsolidated clays has mainly been restricted to field studies, with only a few generic numerical studies. Nonetheless, a large part of underground tunnel construction has happened and continues to occur in overconsolidated clays. The paper bridges this gap by using the case study of Waterloo International Terminal, where two new tunnels were excavated beneath two 70 year-old tunnels, to validate a numerical model. The validated numerical results provide new, valuable insights into the differences and similarities of the response of the existing tunnels depending on their typology (running or station tunnels) and on the time after the excavation of the new tunnels. Furthermore, they reveal the significance of the stiffness reduction factors that need to be applied to account for the segmental nature of the tunnel linings, highlighting the need for further research into the operational value of the tunnel stiffness.

Keywords: tunnelling, overconsolidated clays, segmental linings, tunnel interaction, lining stiffness

Introduction

Metropolitan areas around the globe are continuously expanding along with the ever increasing concentration of population in these areas. To cope with the new societal and economic demands, urban transport systems are growing accordingly, with underground transport, e.g., railways and roads, playing a major role as it can alleviate the congestion and shortage of space in the surface. One of the primary challenges when constructing new tunnels in urban environments is minimizing ground movements and the potentially detrimental effects on existing surface and subsurface infrastructure. In this context, the present work focuses on the impact of tunnel construction on existing tunnels.

When assessing the potential impact of new tunnel construction on existing tunnels, it is common industry practice (e.g., TfL, 2017) to perform an initial assessment using relatively simple methods such as empirical models for tunnelling-induced ground movements (Attewell, 1982; Mair et al., 1993) or analytical models considering soil-structure interaction (Klar et al., 2005; Franza and

Viggiani, 2021). A more sophisticated assessment, based on numerical analysis, is subsequently carried out when the initial assessment indicates that damage of the existing tunnel is likely. However, in highly overconsolidated clays, even greenfield tunnelling-induced ground movements have been proven difficult to achieve numerically (e.g., Franzius et al., 2005; Masin, 2009). By extension, it has so far remained unclear whether it is possible to predict reliably tunnelling-induced movements of existing tunnels in such soil conditions. Reliable numerical predictions can potentially decrease the need for overly conservative designs and minimise the environmental and monetary cost of new tunnels. The present work presents the first fully validated numerical study of tunnel-tunnel interaction in overconsolidated clays, thus marking the first step in delivering impact in this direction.

Previous numerical studies in three-dimensional tunnel-tunnel interaction have focused entirely on soft clays, sandy clays and sands (e.g., Chakeri et al., 2011; Ng et al., 2013; Zhang and Huang, 2014; Ng et al., 2015; Yin et al., 2018; Lin et al., 2019) and rocks (e.g., Liu et al., 2009). While they advanced the understanding on the tunnel-tunnel interaction problem, none of them considered tunnelling within an over-consolidated clay. Only Avgerinos et al. (2017) studied three-dimensional interactions between an existing and a newly constructed tunnel in an over-consolidated clay, in this case the London Clay stratum, in a hypothetical and thus unvalidated numerical study. Although the set-up of the model was inspired by the construction of the Crossrail twin tunnels beneath the Central Line twin tunnels, a direct comparison of the numerical results with field data was not possible due to the simplifications made on the problem geometry. Therefore numerical models for tunnel-tunnel interaction in over-consolidated clays are yet to be validated against field or physical modelling measurements.

This paper addresses this gap by simulating in detail the construction of the Jubilee Line Extension twin tunnels, through over-consolidated London Clay, beneath the Northern Line twin tunnels at Waterloo. The movements of the Northern Line tunnels were extensively monitored during construction of the new tunnels and after construction (Standing and Selman, 2001), rendering this case study an excellent benchmark to validate the numerical predictions. For the first time in over-consolidated clays, the numerical results are compared against field measurements, both in the short term and one and two years after construction; there is an excellent agreement between the numerical results and the field data, overcoming previous challenges associated with greenfield tunnel excavations in London clay.

Additionally, new insights are provided into the twin-tunnel interaction problem: the numerical results highlight for the first time with clarity and adequate supporting evidence similarities and differences in the deformation of existing station and running tunnels caused by new tunnel excavations in their vicinity in highly overconsolidated clays, in the short-term and after a few years of consolidation. Furthermore, the numerical results provide evidence that the longitudinal, circumferential and shear stiffness reduction factors that need to be applied to account for the segmental nature of GCI tunnel linings when their joints are not explicitly modelled are all important. So far only the circumferential stiffness reduction factor has been investigated thoroughly for GCI linings, but the longitudinal and shear stiffness reduction factors merit equal attention.

Waterloo International Terminal: interaction of Northern Line and Jubilee Line Extension Tunnels

The subject site is located in central London, south of the River Thames at Waterloo International Terminal (WIT). The case study comprises a perpendicular intersection of twin tunnel-tunnel interaction between the Northern Line (NL) and Jubilee Line Extension (JLE). The NL tunnels were constructed first in 1923/24 in a north-south direction. The JLE tunnels were constructed 71 years later (1994/95) in an east-west direction, passing at a nearly perpendicular intersection beneath the NL tunnels (Standing and Selman, 2001). The NL tunnels comprise the North Bound (NB) and South Bound (SB) tunnels, and the JLE comprise the East Bound (EB) and West Bound (WB) tunnels, as shown in plan view in Figure 1. The Bakerloo and Waterloo & City Lines, to the west and east of the intersection, respectively, are also shown or indicated in the figure. Similar to the approach followed by Standing and Selman (2001), the interaction between the JLE and the NL tunnels was considered independently from the presence of the Bakerloo and Waterloo & City Lines tunnels, which are located at a sufficient horizontal and vertical distance from the NL tunnels: there is a horizontal distance of nearly 50 m axis-to-axis between the closest NL and Bakerloo tunnels, and of 17 m between the closest NL and Waterloo & City Lines tunnels, the latter also being rather shallow with their crown at 8 m of depth.

The site comprises a typical London stratigraphy (Figure 2) of Made Ground (MG) overlying Terrace Gravels (TG). Below the Terrace Gravels the London Clay (LC) Formation is encountered overlying the Lambeth Group (LG) soils, Thanet Sands (TS) and Chalk at depth. The London Clay onsite has been demarcated into the sub-domains B, A3 and A2 in stratigraphical order based on cross referencing BGS borehole logs and descriptions given in Hight et al. (1993; 2003).

Both the NL and JLE tunnels are situated in the London Clay Formation, with the NL tunnels positioned in sub-domain A3 and the JLE tunnels positioned along the boundary of sub-domain A3 and A2. The invert levels for the NL station and running tunnels are positioned at 21.9 m and 20.55 m, respectively. For the JLE tunnels the invert levels are positioned at 32.05 m.

The internal diameters of the NL tunnels are understood to be 6.68 m and 3.57 m for the station and running tunnels, respectively. The station and running tunnels are connected via a concrete headwall assumed to be 0.5 m thickness in absence of structural information. Spacing between the NL station tunnel inverts and the JLE EB crown is around 5.3 m at the point of intersection; the vertical spacing between the NL running tunnel inverts and JLE WB crown is about 6.65 m. The axis-to-axis distance between the JLE tunnels is approximately 32 m and 33.5 m at the intersection with the NL NB and NL SB tunnels, respectively. The axis-to-axis distance between the NL station tunnels is approximately 12 m whereas that between running tunnels is around 14.5 m.

The NL tunnels were constructed with shield tunnelling methods (Craig and Muir Wood, 1978) adopting the traditional configuration of GCI segmental linings, the segments' cross-section comprising a skin and two circumferential flanges that form a U cross-section. Segments and rings are bolted together at the longitudinal and circumferential flanges, respectively. Their dimensions are typical for GCI tunnels in London (Ruiz López, 2022).

The JLE tunnels were formed with an outer diameter of approximately 4.85 m, with the linings comprising a 150 mm Sprayed Concrete Lining (SCL) and a 300 mm cast-in-place secondary lining (Standing and Selman, 2001). The dimensions of both the JLE and NL tunnels were subsequently confirmed from London Underground archive records. The excavation sequence is shown in Figure 3 and comprised the removal of a 3 m top heading forming a bench approximately in the top third of the vertical diameter. The bench and invert were excavated in the subsequent phase with a further 3 m advancement of the top heading. During the second phase, lining installation across the unsupported zone excavated in the previous phase occurred. The WB tunnel was constructed ahead of the EB tunnel at a distance of approximately 15 m.

Field measurements of NL tunnel settlements during JLE tunnel construction are detailed in Standing and Selman (2001). They were taken transverse to the JLE tunnel axis and along a 90 and 105 m length of track bed of the NB and SB lines, respectively. Ongoing yearly monitoring of long-term settlements subsequently continued for two years. Comparisons between the monitoring data and the numerical results are established in Section “Analysis Results”.

Numerical analysis

Geometry and mesh discretisation

The 3D numerical analysis of the case study was conducted with PLAXIS 3D (Bentley, 2023), using a combination of standard software features and user-defined soil models. The dimensions of the model were approximately 250 m by 159 m across the x and y-axis, perpendicular and parallel to the JLE axis, respectively. They were established by Stewart (2022) to limit the impact of near-boundary effects. The thickness of the soil domain was 68.8 m along the vertical z-axis, extending to the Chalk bedrock. The finite element mesh was composed of 10-noded tetrahedral solid elements for the soil body, with three translational degrees of freedom and a pore water pressure degree of freedom at each node. The tunnel linings were formed of 6-noded triangular shell elements with three translational and three rotational degrees of freedom at each node. In total, 241,715 soil elements and 4,078 plate elements (333,178 nodes) were generated across the entire mesh, with mesh refinement regions located around the tunnels and the zone of intersection between tunnels. The mesh and the tunnel positions within the 3D model are shown in Figure 4. Note that the geometry of the numerical model reflects accurately the alignment of the JLE tunnels (Figure 1) as well as the presence of headwalls on the NL station tunnels, this is unlike the model described in Stewart et al. (2023) where a preliminary analysis of this case study was presented. The typical duration of the analyses presented was 18 hours.

Soil constitutive models and material parameters

The IC-MAGE M01 model was employed to simulate the stress-strain behaviour of the London Clay, Terrace Gravels and Lambeth Group layers (Taborda et al., 2023a). This model combines a non-associated Mohr-Coulomb failure criterion with the nonlinear elastic model introduced by Taborda et al. (2016) which was initially designed to provide a flexible and unified platform for modelling the highly nonlinear variation of stiffness with stress and strain under both monotonic and repeated loading. The model employs an algorithm for detecting reversals in the shearing direction, which, in dynamic analyses, generates hysteresis and enables the control of energy dissipation (Taborda & Zdravkovic, 2012). It is available in PLAXIS through a user-defined model implementation (Taborda et al., 2023b). The model considers the dependency of soil stiffness on strain and stress states; it specifies the degradation of the tangent shear stiffness as a function of the deviatoric strain E_d according to the following modified hyperbolic relationship:

$$\mathrm{G_{tan}} = \mathrm{G_{ref}} \cdot \left(\frac{\mathrm{p'}}{\mathrm{p'_{ref}}}\right) \cdot \left(\mathrm{R_G} + \frac{1 - \mathrm{R_G}}{1 + \left(\frac{\mathrm{E_d}}{\mathrm{a}}\right)^{\mathrm{b}}}\right) \tag{1}$$

where p' is the effective mean stress, $p'_{ref} = 100$ kPa in the analyses presented in this paper, and G_{ref}, R_G, a and b are all fitting parameters controlling the reduction of shear stiffness with deviatoric strain. Similarly, the bulk modulus varies according to the following expression:

$$\mathrm{K_{tan}} = \mathrm{K_{ref}} \cdot \left(\frac{\mathrm{p'}}{\mathrm{p'_{ref}}}\right) \cdot \left(\mathrm{R_K} + \frac{1 - \mathrm{R_K}}{1 + \left(\frac{|\varepsilon_{\mathrm{vol}}|}{\mathrm{r}}\right)^{\mathrm{s}}}\right) \tag{2}$$

where ε_{vol} is the volumetric strain and K_{ref}, R_K, r and s are fitting parameters controlling the degradation of bulk stiffness with volumetric strain. The model has the ability to detect reversals in the direction of loading automatically and reset the stiffness to its initial value, as described in detail by Taborda et al. (2016). Furthermore, experimental evidence by Clayton and Heymann (2001) has indicated that after a rest period the soil realises, when subjected to either loading or unloading, the stiffness of first loading. For this reason, it is also possible to reinvoke soil stiffness at very small strains manually, by removing the strain history and re-engaging the maximum elastic stiffness. This action does not affect the material strength or the yield surface. The model has been previously used in numerical analysis of tunnel excavations by Gawecka et al. (2021), Ruiz López (2022), Stewart et al. (2023) and Liu et al. (2024), among others.

The nonlinear stiffness parameters were calibrated for London clay against triaxial data from the testing programme carried out by Yimsiri (2001) on high quality samples from Kennington, and for Terrace Gravel against the laboratory results reported by Hight et al. (1993). Yimsiri (2001) tested isotropically consolidated samples (obtained from various depths) in drained and undrained triaxial compression, using LDVT transducers (the dynamic testing that was also performed was deemed by Yimsiri (2001) to have yielded unreasonably high shear modulus at small strains and was, therefore, disregarded in the calibration). The data fall within the typical range of shear modulus reduction with strain for London Clay presented in Hight et al. (2003). The model's stiffness reduction curves for units B and A3-A2, together with the laboratory data, are presented in Figure 5, and the corresponding stiffness and strength parameters are listed in Table 1 and Table 2, respectively. The constitutive behaviour of the Made Ground and Thanet Sands layers was simulated with the linear elastic non-associated plastic Mohr-Coulomb model, with parameters also reported in Table 2. Details of the calibration were reported by Stewart (2022). It should be noted that although the IC-MAGE M01 model includes 47 input parameters (and 75 hardening parameters which do not require user input and are automatically updated primarily to deal with reversals), the parameters listed in Tables 1 and 2 are sufficient to define fully the strength and stiffness characteristics of the encountered soil layers. All other input model parameters were set to values that disable certain features of the model, as they control aspects of soil behaviour that are unrelated to the boundary value problem studied (e.g. energy dissipation, stiffness dependency on void ratio, strain softening).

Structural modelling and lining material parameters

The JLE tunnel linings were simulated as isotropic linear elastic, adopting axial and bending stiffnesses determined from the sum of the dimensions of the primary and secondary linings and assuming an elastic modulus of 30 GPa. Correspondingly, the axial and bending stiffnesses were taken as $13.5 \cdot 10^6$ kN/m and $22.8 \cdot 10^4$ kNm^2/m, respectively. Poisson's ratio was taken as 0.2.

An elastic modulus of 100 GPa and a Poisson's ratio of 0.26 were adopted for the GCI NL tunnel linings (Angus, 1976). The mechanical behaviour of the NL tunnels was simulated with the anisotropic elasticity model for shells available in PLAXIS (Bentley Systems, 2024) whose formulation and all the associated input parameters employed in the analyses are provided in Appendix A. Full friction was assumed at the interface between the tunnels and the soil; preliminary analyses suggested that the interface condition had a limited impact on the results.

To account for the segmental nature of the tunnels, a set of equivalent stiffness values, determined through the application of reduction factors with respect to the full stiffness magnitudes, was employed in the analysis presented in Section "Analysis results" (referred to as "Original"). The whole set of reduction factors applied is listed in Table 3 while the rationale for establishing them is explained below.

The 'global' bending stiffness reduction factor η, derived by Ruiz Lopez et al. (2022) from a series of 3D numerical analyses on a segmental GCI lining, was applied to obtain equivalent circumferential axial and bending stiffnesses, $(EA)_c^*$ and $(EI)_c^*$, respectively; η is meant to represent the stiffness reduction needed for a continuous ring to produce the same tunnel squat (i.e. horizontal tunnel distortion) as the equivalent segmental ring. A reduction factor η of 0.777 was applied in the analyses. As can be observed in Figure 6, this magnitude is consistent with a tunnel squat of 1% which is the value recommended by TfL (2017) for existing tunnels when field measurements are not available. While the reduction factor was derived based on the response of a running tunnel ring (Ruiz Lopez et al., 2022), it was applied here to both running and station tunnels, for lack of other appropriate factors in the relevant literature.

In the absence of a reduction factor specifically derived to define the equivalent longitudinal bending stiffness $(EI)_l^*$ of GCI tunnel linings, the factor proposed by Shiba et al. (1988) was adopted. According to Shiba et al.'s method the longitudinal bending stiffness reduction factor, ξ, can be defined as:

$$\xi = \frac{\cos^3 \psi}{\cos \psi + (\psi + \pi/2) \sin \psi} \tag{3}$$

where ψ is the angle, determined from the tunnel springline, defining the position of the neutral axis and can be obtained from the following expression:

$$\psi + \cot \psi = \pi \left(\frac{1}{2} + \frac{n\, E_b A_b}{E_s\, A_s} \right) \tag{4}$$

where n is the number of bolts around the circumference; E_b and A_b are the elastic modulus and cross-section area of the bolts connecting the circumferential joints, respectively; E_s and A_s are the elastic modulus of the tunnel segment material and the circumferential cross-section area of the ring, respectively. The factors ξ to be applied in the analysis were derived adopting 53 bolts, for the running tunnel, and 83 bolts, for the station tunnel, around the circumference, and an elastic modulus of 270 GPa for the bolts in accordance with the results from tensile testing presented in Tube Lines (2008). The same factors ξ were used to obtain equivalent longitudinal axial stiffness $(EA)_l^*$ of the running and station tunnels.

Shear stiffness reduction factors are yet to be established for GCI linings. For that reason, the reduction factor ζ of Wu et al. (2015), originally meant for concrete linings, was used to obtain an equivalent in-plane shear stiffness $(GA)_{lc}^*$ of the running and station tunnels. This factor can be defined as follows:

$$\zeta = \lambda \frac{l_s}{\frac{l_b \kappa_s G_s A_s}{n \kappa_b G_b A_b} + l_s - l_b} \tag{5}$$

where l_s and l_b are the length of the segments and that of the bolts connecting the circumferential joints, respectively; G_s and G_b are the shear moduli of the tunnel segment material and the bolt material, respectively; κ_s and κ_b are the Timoshenko shear correction factors for the tunnel segments and bolts, respectively; λ is a modifying factor. The latter, κ_s and κ_b were all adopted as 1. The remaining shear stiffness components were unchanged while the torsional stiffness was also reduced according to ζ.

It is worth noting that because factors ξ and ζ were not derived specifically for GCI linings, they do not consider certain geometric characteristics, such as the U-shape segment cross-section or the deep caulking groove found in the joints connecting rings (Craig, R. and Muir Wood, 1978), that are specific to GCI linings and may differentiate their bending and shear behaviour from other types of linings. The need for establishing appropriate stiffness reduction factors is highlighted in Section "Influence of existing tunnels' circumferential, longitudinal and shear stiffness reduction factors".

The station tunnels' headwalls adopted mechanical properties typical of concrete: an elastic modulus of 30 GPa and a Poisson's ratio equal to 0.2.

Initial and boundary conditions and construction sequence

The initial pore water pressure and K_0 profiles with depth adopted in the analysis were based on data presented in Hight et al. (2003) and Hight et al. (1993), respectively, and are typical for central London. They are shown in Figure 2 in association with the soil stratigraphy and tunnel elevations. It should be noted that applying a linear variation of K_0 with depth is not a standard PLAXIS capability and was implemented as a feature of the user-defined soil model IC-MAGE M01. K_0 field data in Hight et al. (2003; 1993) are presented in Figure 2, along with the profile used in the analysis. The field data originated from a variety of tests including soil bore pressuremeter, filter paper and dilatometer, carried out at the Waterloo site.

In the first analysis phase, stresses were initialised based on the input unit weight, K_0 and the underdrained pore pressure profile. In the second phase of the analysis, the pore water pressures were allowed to equilibrate over a period of 100 years with the applied hydraulic boundary conditions under the input soil permeabilities: London Clay and Lambeth Group layers were consolidating whereas the remaining layers were set to be draining; the pore water pressures at the interface between draining and consolidating layers were kept constant throughout the analysis; the vertical boundaries were impermeable. The permeabilities of the consolidating layers are summarised in Table

2 and the approximated and measured profile from Hight et al. (1993) and the underdrained profile from the analysis are compared in Figure 2. Under these conditions and soil permeabilities, the initially input pore water pressure profile remained unchanged over 100 years, which is indicative that the soil permeabilities were compatible with the underdrained profile.

The excavation and construction of the NB NL tunnel were simulated subsequently assuming undrained conditions. The excavation mimicked the standard stress reduction procedure typically followed in plane-strain analysis (Potts & Zdravkovic, 2001): the soil along the tunnel length was removed all at once while applying a radial isotropic load around the tunnel circumference to control the stress reduction surrounding the tunnel. The magnitude of this load was adjusted through trial-and-error such that the volume loss at the end of the excavation reached the typical value of 1.5%: as a result, a 90 kPa load was applied along the running tunnel sections while a 120 kPa load was prescribed along the station tunnel sections. Additionally, a 107.5 kPa load was imposed perpendicular to the soil at the section where the running and station tunnels intersect, i.e. where the headwalls would be located. Once the excavation was completed, the tunnel lining and headwall were constructed. The SB tunnel was constructed assuming undrained conditions and a construction sequence identical to that of the NB NL tunnel.

A 71-year consolidation period followed the construction of the Northern Line tunnels. During this period, the NL tunnels were fully permeable owing to their segmental nature and typical field observations (Mair, 2008), allowing seepage through the lining. At the end of this period and prior to the construction of the JLE tunnels, the maximum elastic stiffness at very small strains was reinvoked.

The JLE tunnels were excavated with a step-by-step approach (Katzenbach and Breth, 1981), adopting a 3 m unsupported length for both tunnels, as during construction. The excavation was conducted in the same manner as already described in relation to Figure 3. Just as in the field, the JLE EB tunnel was driven about 15 m behind the JLE WB tunnel. A 2-year consolidation period was simulated after the construction of the JLE tunnels.

Throughout the analysis, the displacements normal to the vertical boundaries were restrained while displacements in all directions were restrained along the base of the model.

Analysis Results

Short-term settlements

Figure 7 shows the vertical settlements caused by the excavation of the JLE tunnels along the axes of the NL SB and NB tunnels, measured in-situ by Standing and Selman (2001) at the elevation of the track, and calculated numerically at the invert of the tunnels, at the end of the excavation. Selecting to present the numerical results at the invert rather than the track elevation, where the monitoring data were obtained from, simplified the numerical analysis, as the concrete block itself, whose thickness and stiffness were not specified by Standing and Selman (2001) and are not publicly available in other LUL documents, and where the track is founded, was not included in the analysis. The good match between the monitoring and numerical data, supports that this simplification was reasonable.

Negative values of vertical displacement in Figure 7 denote settlements. The horizontal axes denote distance along the NL tunnel axes in the numerical model. The change from station to running tunnels is clearly marked, as is the location of the axes of the JLE tunnels underneath the NL tunnels. With reference to both the SB and NB NL tunnels in Figure 7 (a) and (b), respectively, there was a marked difference between the measured settlements at the axes of the WB JLE tunnel, corresponding to the NL running tunnel length, and of the EB JLE tunnel, corresponding to the NL station tunnel length, with maximum settlements occurring above the WB JLE tunnel. Qualitatively, the movements predicted by the numerical analysis were consistent with these observations as the settlements were larger along the running tunnel length, for both the SB and NB NL tunnels. Standing & Selman (2001) postulated that this could be a consequence of the greater stiffness of the NL station tunnels in comparison with the NL running tunnels and the comparison with the greenfield analysis, presented in Section “Comparison of greenfield response with existing tunnels’ response”, seems to confirm that hypothesis.

Overall, the numerical analysis reproduced the movements of the NL tunnels, in relation to the field measurements, very well. The settlement trough of the SB NL tunnels was in excellent agreement with the measurements all along the tunnel length being considered; while the analysis underpredicted the maximum settlements of the NB NL tunnels, the differences were not significant and a good match with the field measurements was attained away from the point of maximum deflection.

The settlements at the SB and NB NL tunnels computed in the analysis are compared in Figure 7 (c). The two troughs are very similar as expected considering the horizontal distance between the two JLE tunnels. Differences are limited to the settlement of the running tunnels and are attributable to the location of the WB JLE tunnel (note that this was excavated at an oblique angle in relation to the NL tunnels and therefore crossed beneath them at a different distance along the NL tunnel axes) and to the different lengths of the softer, running NL tunnels. This similarity in settlement troughs was not present in the field data. Standing and Selman (2001) did not provide an explanation for this and therefore, it is not known if it was caused by variations in the ground conditions, the construction process of the JLE tunnels or other factors that had they been documented could have been accounted for in the numerical model.

Figure 8 presents normalised vertical settlements at the invert of the NL tunnels at four locations. Points D and C refer to the invert of the SB NL tunnel directly above the EB and WB JLE tunnels, respectively. Points B and A refer to the invert of the NB NL tunnel directly above the EB and WB JLE tunnels, respectively. The four points are clearly shown at the schematic representation of the problem geometry included in Figure 8. The vertical settlements in the figure have been normalised by the maximum vertical displacement at the examined location during the excavation of the designated JLE tunnel. The horizontal axes plot the offset of the JLE excavation from the considered point, i.e. it shows the relative position of the tunnel face with respect to the point. When the horizontal axis reads 0, the JLE tunnel is being excavated directly below the considered point. Negative values indicate that the JLE excavation is approaching the examined point, whereas positive values indicate that the excavation has advanced further than the examined point. The good agreement between the numerical and field data signifies that the numerical analysis did not only capture the magnitude of settlements at the end of excavation, but also the gradual evolution of the settlements as the tunnelling process advanced underneath the NL tunnels.

Settlements due to consolidation

Standing & Selman (2001) presented settlements at the NL tunnels 308 and 705 days after the end of excavation. The same elapsed time was considered in the numerical analysis. For ease of reference the numerical data are compared to the monitored data in Figure 9 under the labels "1 year" and "2 years". Standing & Selman (2001) highlighted that settlements approximately doubled over the two-year period examined. This suggests that waterproofing of the JLE tunnels was ineffective and as no records seem to be publicly available regarding the infiltration rate of water in the tunnel section under consideration, the JLE tunnels were simulated as fully permeable. Correspondingly, the analysis yielded settlements in very good agreement with those measured in-situ, at both the SB and the NB NL tunnels. Figure 10 presents settlements normalised by the maximum settlement of each of the troughs, allowing their shape to be compared between the various results. Overall, the shape of the numerical results is similar to that observed in the field data, the normalised results at the end of the excavation being particularly close to the monitoring data. Interestingly, the field data indicates that the troughs became wider on the station tunnels' end but not so much on the running tunnels' end; The numerical troughs, on the other hand, become slightly wider on both ends.

Accurate simulation of consolidation settlements in numerical analysis requires that the soil permeability and the applied hydraulic boundary conditions are correctly chosen. Therefore, in a sense, the good agreement between the numerical and monitoring data validates the numerical model. This is of particular significance considering that values for the permeability of the various units were selected to replicate the initial pore water pressure profile on the site.

Overall, the numerical model successfully reproduced reality, as measured in the field during the excavation of the JLE tunnels (short-term settlements) and in the following years (settlements due to consolidation). The validated numerical model was used to provide further insights into the interaction of the four tunnels in the subsequent section.

Further insights

This section aims to expand the understanding of the interaction problem between twin tunnels using the numerical model previously described, and validated by the good agreement with the field data.

Firstly, the stiffening effect of the existing tunnels on the surrounding ground is evaluated by comparison with an additional analysis without the presence of the existing tunnels; secondly, the deformation mechanisms involved in the interaction problem are shown through a careful examination of the numerical results; and thirdly, the effect of each of the stiffness reduction factors applied to the existing tunnels on the vertical settlements is investigated.

Comparison of greenfield response with existing tunnels' response

To understand the interaction between the four tunnels, the excavation of the JLE tunnels was repeated in a new numerical analysis replicating the original analysis in everything but the excavation of the NL tunnels: excavation of the two JLE tunnels was conducted under greenfield conditions, i.e. the NL tunnels were entirely absent. The same excavation sequence was otherwise followed.

The settlements produced by the two analyses are presented in Figure 11. The analysis containing both the NL and JLE tunnels has been marked as "Original" and the additional one as "Greenfield". The settlements have been presented at the exact same elevation in the two analyses, which coincided with the inverts of the NL tunnels in the "Original" analysis. It should be highlighted that there is 1.35 m vertical distance between the invert of the station and the running NL tunnels, with the station tunnels lying deeper, and, therefore, having a smaller clearance from the JLE tunnels.

With reference to the end of excavation, the settlements along the location of the running tunnel invert in the "Greenfield" analysis practically coincided with the settlements at the running tunnel invert in the "Original" analysis. This was the case for both the SB and NB NL tunnels and supports field observations in historic (e.g. Standing & Selman, 2001) and more recent case studies (e.g. Alhaddad, 2016), as well as assumptions made in practice when performing initial damage assessment (e.g. Crossrail 2011a, 2011b), that the existing tunnel stiffness has a negligible effect on longitudinal settlements and that cast-iron segmental linings follow the longitudinal deformation of the ground. Alhaddad et al, (2021) noted that this observation holds when new tunnels cross underneath existing tunnels in London clay, which is the case between NL and JLE tunnels, but not when the new tunnels are parallel to the existing ones.

The two analyses produced distinct settlements both at the SB and the NB NL station tunnels, with the "Greenfield" analysis yielding larger settlement directly above the axis of the EB JLE tunnel than the "Original" analysis. This signifies that the stiffness of the station tunnels, which is approximately 8 times larger than the (bending) stiffness of the running tunnels, cannot be considered as having negligible effect on the developed settlements. Clearly, the station tunnels did not follow the settlement of the ground when the EB JLE tunnel crossed perpendicularly underneath them, whereas the running tunnels did. It is, therefore, possible that in addition to the direction of tunnelling (parallel, perpendicular or at an oblique angle in relation to the existing tunnels), the typology of the existing tunnels (station or running) would affect the expected settlements.

It should be noted that the settlement in the "Greenfield" analysis appears to be larger above the EB JLE tunnel, which was the second to be excavated, than above the WB JLE tunnel, which was excavated first, because of the difference in elevation between the two troughs, which coincided with the NL tunnel inverts in the "Original" analysis.

With reference to the consolidation-induced movements, the "Greenfield" analysis produced significantly larger settlements than the "Original" analysis, e.g. the peak settlements of the NB station tunnel are about 30% larger than those obtained in the "Original" analysis. The difference is justified by the NL tunnels acting as drains for a total period of 71 years and lowering the pore water pressure significantly prior to the excavation of the JLE tunnels in the "Original" analysis in comparison to the "Greenfield" analysis, as illustrated in Figure 12. This meant that in the "Original" analysis, the pore water pressures at the beginning of the excavation were closer to the pore water pressures expected at the end of the 2 years of consolidation, under hydraulic boundary conditions that were altered by the presence of the new tunnels, which also acted like drains (also shown in Figure 12). Consequently, the settlements in the "Original" analysis were smaller in comparison to the "Greenfield" analysis.

It can be concluded that the existing running tunnels contributed only to settlements due to consolidation and not to the short-term settlements: although their stiffness was of little relevance, their effect on the hydraulic boundary conditions was of significance. The existing station tunnels contributed to both the short-term and consolidation settlements: their stiffness as well as their effect on the hydraulic boundary conditions were of significance.

3D movements of existing tunnels during new tunnel construction

Figure 13 illustrates the evolution of the NL tunnel shapes (cross-sections in the global y and z-directions through points A to D as per schematic) with the excavation of the JLE tunnels and after 2 years of consolidation. For illustration purposes, a scaling factor of 1000 was used and, therefore, all displacements and deformations are highly exaggerated. They do, however, provide a general impression of how the cross-sectional shape of the NL tunnels displaced and deformed as the excavation of the JLE tunnels progressed and as the excess pore water pressure started dissipating.

When the new excavation was at -6 m from the cross-sections examined, in all four cases the existing tunnels demonstrated a shape that can be described as “squatting”, with the invert shifting towards the approaching excavation. As the new excavation reached the examined cross-sections and progressed past them, the existing tunnels rotated, with the invert following the direction of the new tunnel and the shape changing to what can be described as “egging”. Similar observations were made by Avgerinos et al. (2017). Further to previous findings, the current case study clearly demonstrated that the typology of the existing tunnels and the (acute) angle of excavation did not affect qualitatively this behaviour.

Two years after the excavation of the JLE tunnels, the NL tunnels deformed further in relation to the end of excavation due to the dissipation of excess pore water pressures generated during the excavation. It can be observed from Figure 13 (a) and (b) that although the SB NL tunnels (D and C, respectively) continued deforming in the same direction as during the JLE tunnel excavation, the NB NL tunnels “rotated” the other way (Figure 13 (c) and (c) for B and A, respectively). This can be explained by the magnitude of the pore water pressures. Figure 14 (a) shows the pore water pressures in a cross-section following the axis of the EB JLE tunnel and thus cutting through the two NL station tunnels, at points B (on the left) and D (on the right). The presence of small suctions at the elevation of the inverts in between the two station tunnels (positive pore water pressures) increased the effective stresses and stiffness in this area. As the tunnels settled with time, they also rotated with their inverts moving “outwards” and away from each other. This is further illustrated in Figure 14 (b), which plots vectors of displacements for this phase of consolidation. A clear downward and slightly outward overall movement can be inferred for each of the two NL tunnels. The behaviour was similar in the cross-section following the axis of the WB JLE tunnel, with the two running NL tunnels moving qualitatively in a way similar to the station tunnels. With consolidation, the shape of all the tunnels changed towards “squatting” but their “rotation” (leftwards or rightwards) depended largely on their position in relation to their twin NL tunnel, and not at all on their typology, demonstrating that they followed the ground movements, while their lining stiffness had little qualitative contribution to their overall deformation. This is the first time that such an insight has been possible from numerical results.

Influence of existing tunnels’ circumferential, longitudinal and shear stiffness reduction factors

The NL tunnels at Waterloo International Terminal are supported by GCI linings whose segmental nature is known to affect their behaviour considerably. Although research has enabled bending moment reduction factors for GCI segmental linings to be established in the circumferential direction (e.g. Ruiz López, 2022; Ruiz López et al., 2022), no such factors have been established for the longitudinal bending stiffness or the shear stiffness. As a result, it is not known with certainty how the longitudinal bending stiffness and the shear stiffness of segmental GCI tunnel linings may be affected by the presence of bolted connections. Alhaddad et al. (2021) postulated that shearing between rings could be partly responsible for the largely flexible behaviour observed in two case studies of cast iron tunnels subjected to movements from new tunnel construction in their vicinity; the authors, however, did not provide any quantitative evidence supporting that claim. In this work, it was assumed that both the longitudinal bending and the shear stiffness are affected by the presence of the bolted connections and the stiffness reduction factors of Shiba et al. (1988) and Wu et al. (2015), which were not derived specifically for GCI linings, were employed for the first time in this type of tunnel. The numerical results matched the monitoring data successfully, supporting the argument that the adopted assumptions were reasonable. Nonetheless, the matter of stiffness reduction factors is examined further in this section by comparing the settlement troughs followed by the SB NL and NB NL tunnels at the end of the JLE tunnel excavation in analyses where only one reduction factor was applied at a time.

In the analysis denoted as "E_l" in Figure 15 only the longitudinal stiffness E_l was reduced (by the same factor ξ as in the "Original" analysis) and no other reduction factor was applied. Similarly, in the analysis "E_c" the circumferential stiffness E_c was reduced by factor η while setting the remaining stiffnesses to their full value. For reference, the results from analyses including reductions in all stiffness components ("Original") and without any stiffness reductions ("Full NL stiffness") are also included. The individual effect of reduction factors ξ and η can be assessed by comparing the troughs against that from the "Full NL stiffness" analysis. It can be observed that the reduction factors have negligible influence on the peak settlement taking place on the running tunnel section (above the WB JLE tunnel) while their application causes an increase in the peak settlement happening on the station tunnel. Such an increase is particularly obvious in the SB NL tunnel. It is worth noting that reduction factor ξ is considerably smaller than η for both running and station tunnels (0.135 and 0.117, respectively, versus 0.777) and so it can be argued that, relatively speaking, the influence of η on the peak settlements is more significant. While it is generally assumed that the tunnel's circumferential stiffness is only relevant when assessing the distortion of the tunnel cross-section, these results highlight that adopting an appropriate factor η deserves consideration also when looking at longitudinal movements. Between the peak settlements, the influence of factor ξ is clearly more considerable than that of η. In fact, the trough produced by analysis E_l is largely overlapping that given by the "Original" analysis around the point of minimum deflection located between the peak settlements. Evidently, the coupled effect of reducing the various stiffness components simultaneously is different than the summed effect of reducing them individually which is explained by the nonlinearity of the soil-structure interaction system.

For clarity, the impact of the shear stiffness reduction factor ζ is investigated in a separate figure. Figure 16 shows the settlement troughs given by an analysis, referred to as "G_{lc}", where the shear stiffness was reduced by factor ζ while the other stiffness components adopted their full magnitudes and those obtained in the "Original" and "Full NL stiffness" analyses. The significant impact of factor ζ is made apparent by the differences between the results of the "G_{lc}" analysis and the "Full NL stiffness" analysis. Moreover, the peak settlements given by the "G_{lc}" analysis are nearly as large as those predicted by the "Original" analysis; by comparison with Figure 15, it is obvious that, in terms of peak settlements, ζ is the most influential stiffness reduction factor out of the three considered. On the other hand, its influence on the settlements of the tunnel length between peak settlements is comparable to that of factor η ("E_c" analysis in Figure 15).

The results presented in this section demonstrate that the circumferential, longitudinal and shear stiffness of the existing GCI tunnels all play a role, albeit of different importance, on the longitudinal movements experienced by the tunnels. Because they all contribute to the movements, different combinations of stiffness values can potentially yield the same settlement troughs, and therefore, the stiffness reduction factors applied need to be selected carefully. Here, their selection was made a priori, based on the existing literature (i.e. their value was not back-calculated to achieve good agreement with the field data). Although the value of the circumferential stiffness reduction factor has already been established through thorough previous research, such research has not been conducted for the longitudinal and shear stiffnesses of GCI segmental tunnel linings specifically. In the present investigation, it was assumed that longitudinal and shear stiffnesses were both affected by the presence of the tunnel joints; this assumption, in conjunction with the adopted reduction factors, produced satisfactory results, however, further field and numerical research is needed to establish whether the longitudinal movements of GCI tunnels are bending-dominated, shear-dominated or a combination of the two. In terms of numerical research, considering the presence of circumferential joints (those between rings) explicitly in structural analyses similar to those already conducted for longitudinal joints (those between segments) could be a step forward towards this goal, as it would allow the local response of the joints to be investigated and informed stiffness reduction factors to be produced for use in simplified geotechnical numerical analysis (i.e. where joints are not modelled explicitly).

Conclusions

The three-dimensional interaction between the NL and JLE tunnels at the Waterloo International Terminal provided an excellent case study to validate the 3D numerical model presented here. Indeed, both short-term as well as consolidation settlements were accurately reproduced in the numerical analysis. The validated numerical model was used to provide further insights into the interaction of the tunnels and their general behaviour.

Comparison with greenfield conditions (i.e. in the absence of the existing NL tunnels) revealed that the existing tunnel stiffness has a negligible effect on longitudinal settlements and that cast-iron segmental linings within London Clay follow the longitudinal deformation of the ground only for running tunnels. This observation may not stand for station tunnels which are substantially stiffer. It is, therefore, likely that in addition to the direction of tunnelling (parallel, perpendicular or at an oblique angle in relation to the existing tunnels), the typology of the existing tunnels (station or running) would affect the expected short-term settlements as a result of the relative tunnel-soil stiffness

Furthermore, it was concluded that although they did not contribute to the short-term longitudinal settlements, the existing running tunnels contributed to settlements due to consolidation, meaning that although their stiffness was of little relevance, their effect on the hydraulic boundary conditions was of significance. The existing station tunnels contributed to both the short-term and consolidation settlements in the longitudinal direction, signifying that their stiffness as well as their effect on the hydraulic boundary conditions were of significance.

With reference to the 3D deformation of the existing tunnels, the analysis confirmed observations made by Avgerinos et al. (2017) regarding the change of shape from squatting to egging as the new tunnel approached the existing one and advanced further. Additional, new observations were made in that this behaviour did not seem to be affected by the angle of the new excavation in relation to the existing tunnel, at least for the small skew angle of the present geometry, or by the tunnel typology of the existing tunnel (station or running). Furthermore, it was demonstrated that with consolidation, the shape of all the tunnels changed towards “squatting” but their “rotation” (leftwards or rightwards) depended largely on their position in relation to their twin NL tunnel, and not at all on their typology, demonstrating that they followed the ground movements, while their stiffness had little qualitative contribution to their overall consolidation-induced deformation.

Although circumferential stiffness reduction factors have been established through extensive research for GCI segmental linings, no such factors have been proposed for the longitudinal and shear stiffnesses. Nonetheless, the parametric analyses presented confirmed that that these too are of importance for both running and station tunnels. The paper provides clear evidence that detailed and careful research into the operational longitudinal and shear stiffness of segmental tunnel linings in the field is required, similar to that already conducted for the circumferential stiffness.

Data availability statement

Some models employed during the study are proprietary, or confidential in nature and may only be provided with restrictions.

Appendix

Anisotropic shell constitutive model for existing NL tunnels

Formulation

The anisotropic shell constitutive model (Bentley Systems, 2024) adopted in the analyses to simulate the behaviour of the NL tunnels is described in this section.

Firstly, the formulation is defined according to the system of local axes depicted in Figure A1.

The model considers the following relationships between axial forces (N_l, N_c) and strains (ε_l, ε_c):

$$\begin{bmatrix} N_l \\ N_c \end{bmatrix} = \begin{bmatrix} E_l d & \upsilon_{lc} E_c d \\ \upsilon_{lc} E_c d & E_c d \end{bmatrix} \cdot \begin{bmatrix} \varepsilon_l \\ \varepsilon_c \end{bmatrix} \qquad \text{(A 1)}$$

where E_l and E_c are the elastic moduli in the longitudinal and circumferential directions, respectively, ν_{lc} is the Poisson's ratio and d is the thickness of the shell. Likewise, the following relationship between shear forces (Q_{lc}, Q_{lr}, Q_{cr}) and strains (γ_{lc}, γ_{lr}, γ_{cr}) is imposed:

$$\begin{bmatrix} Q_{lc} \\ Q_{lr} \\ Q_{cr} \end{bmatrix} = \begin{bmatrix} G_{lc}d & 0 & 0 \\ 0 & kG_{lr}d & 0 \\ 0 & 0 & kG_{cr}d \end{bmatrix} \cdot \begin{bmatrix} \gamma_{lc} \\ \gamma_{lr} \\ \gamma_{cr} \end{bmatrix} \tag{A 2}$$

where G_{lc} is the in-plane shear modulus; G_{lr} and G_{cr} are the out-of-plane shear moduli; and k is the shear correction factor which is taken as 5/6. Lastly, the following relationship between bending moments (M_{ll}, M_{cc}, M_{lc}) and bending strains (κ_{ll}, κ_{cc}, κ_{lc}) is considered:

$$\begin{bmatrix} M_{ll} \\ M_{cc} \\ M_{lc} \end{bmatrix} = \begin{bmatrix} \frac{E_l d^3}{12} & \frac{\upsilon_{lc} E_c d^3}{12} & 0 \\ \frac{\upsilon_{lc} E_c d^3}{12} & \frac{E_c d^3}{12} & 0 \\ 0 & 0 & \frac{G_{lc} d^3}{12} \end{bmatrix} \cdot \begin{bmatrix} \kappa_{ll} \\ \kappa_{cc} \\ \kappa_{lc} \end{bmatrix} \tag{A 3}$$

Adopted parameters

Since the GCI segments employed to construct the NL tunnels have a U cross-section, rather than a rectangular cross-section as assumed by the shell formulation outlined above, the input material stiffnesses and thickness d were adjusted such that the resulting axial, bending and shear stiffness matched those of the NL tunnels when assuming the tunnel joints are rigid. For the “Full NL stiffness” scenario, the input parameters were determined as follows:

$$d = \sqrt{\frac{12EI}{EA}} \tag{A 4}$$

$$E_l = E_c = \frac{EA}{d} \tag{A 5}$$

$$G_{lc} = G_{lr} = G_{cr} = \frac{GA}{d} \tag{A 6}$$

$$\nu_{lc} = \nu \tag{A 7}$$

where E is the elastic modulus of the material, adopted as 100 GPa; I is the second moment of area of the segment’s cross-section, taken as $4.58 \cdot 10^{-5}$ m^4/m for the running tunnels and $2.16 \cdot 10^{-4}$ m^4/m for the station tunnels; A is the area of the segment’s cross-section, taken as $3.61 \cdot 10^{-2}$ m^2/m for the running tunnels and $5.89 \cdot 10^{-2}$ m^2/m for the station tunnels; ν is the material’s Poisson’s ratio, adopted as 0.26; and G is the material’s shear modulus, determined as $E/2(1+\nu)$.

As explained in Section 0, reduction factors were applied to the various stiffness components in order to account for the segmental nature of the NL tunnels in analysis “Original” and other analyses described in the paper. First, equivalent longitudinal axial $(EA)_l^*$ and bending $(EI)_l^*$ stiffnesses were established by considering reduction factor ξ so that $(EA)_l^* = \xi(EA)$ and $(EI)_l^* = \xi(EI)$. This is included in the model by adopting $E_l = \xi EA/d$. It is worth noting that such a reduction affects not only the “local” stiffnesses of the shell but also, by the same factor, the tunnels’ cross-sectional axial and bending stiffnesses, which are computed using the area and second moment of area of the cross-section of a tube and are relevant when interpreting the behaviour of a tunnel as an equivalent beam. For reference only, the calculation of such cross-sectional stiffnesses is illustrated in Figure A2. Similar to the above, equivalent circumferential axial $(EA)_c^*$ and bending $(EI)_c^*$ stiffnesses were obtained through application of the circumferential reduction factor η such that $(EA)_c^* = \eta(EA)$ and $(EI)_c^* = \eta(EI)$. This was accomplished by adopting $E_c = \eta EA/d$ in the model. Lastly, an equivalent in-plane shear stiffness $(GA)_{lc}^*$ was computed by applying reduction factor ζ such that $(GA)_{lc}^* = \zeta(GA)$. This was realised in the model by assigning $G_{lc} = \zeta GA/d$. Note that the torsional stiffness (the bottom right term in the stiffness matrix of Equation (A 3)) is also reduced by ζ.

The full set of model parameters for analyses “Full NL stiffness” and “Original” are listed in Table A 1.

Supplementary materials

The results of an additional analysis where the NL tunnels were simulated as impermeable can be found online in the ASCE Library

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Layer	G_{ref} (kPa)	a_0	b	R_{Gmin}	G_{min} (kPa)	K_{ref} (kPa)	r_0	s	R_{Kmin}	K_{min} (kPa)
TG	52.4E3	2.10E-4	1.250	0.100	2000	50 E3	1.3E-4	1.34	0.10	5000
LC B	37.5E3	1.90E-4	0.925	0.100	2667	75 E3	2.5E-5	0.75	0.12	5000
LC A3-A2	25.0E3	5.85E-4	0.900	0.075	2667	48 E3	2.5E-5	0.80	0.14	5000
LG 1-2	25.0E3	5.85E-4	0.900	0.075	2667	48 E3	2.5E-5	0.80	0.14	5000

Table 1 Shear and bulk stiffness parameters of IC-MAGE M01 model

Layer	E (kPa)	μ	c′ (kPa)	φ′ (°)	ψ′ (°)	k (m/s)
Made Ground	10E4	0.2	1	30	12.5	Drained
Terrace Gravels	IC-MAGE M01 model		0	35	17.5	Drained
London Clay B	IC-MAGE M01 model		5	23	11.5	1.08E-8
London Clay A3	IC-MAGE M01 model		10	25	11.5	5.21E-10
London Clay A2	IC-MAGE M01 model		15	25	12.5	2.6E-10
Lambeth Group 1	IC-MAGE M01 model		11	27	11	1.74E-10
Lambeth Group 2	IC-MAGE M01 model		11	27	11	1.33E-10
Thanet Sands	30E4	0.2	0	32	16	Drained

Table 2 Stiffness, strength and permeability parameters adopted in the analyses

Stiffness reduction factor	Running tunnels	Station tunnels
η (circumferential)	0.777	0.777
ξ (longitudinal)	0.135	0.117
ζ (shear)	0.319	0.229

Table 3 Set of reduction factors adopted in analysis "Original"

Model parameter	Full NL stiffness		Original	
	Running tunnels	Station tunnels	Running tunnels	Station tunnels
$\boldsymbol{E_l}$	$29.36 \cdot 10^6$ kPa	$28.05 \cdot 10^6$ kPa	$3.97 \cdot 10^6$ kPa	$3.29 \cdot 10^6$ kPa
$\boldsymbol{E_c}$	$29.36 \cdot 10^6$ kPa	$28.05 \cdot 10^6$ kPa	$22.81 \cdot 10^6$ kPa	$21.80 \cdot 10^6$ kPa
$\boldsymbol{G_{lc}}$	$11.65 \cdot 10^6$ kPa	$11.13 \cdot 10^6$ kPa	$3.72 \cdot 10^6$ kPa	$2.55 \cdot 10^6$ kPa
$\boldsymbol{G_{lr}}$	$11.65 \cdot 10^6$ kPa	$11.13 \cdot 10^6$ kPa	$11.65 \cdot 10^6$ kPa	$11.13 \cdot 10^6$ kPa

$\boldsymbol{G_{cr}}$	$11.65\cdot10^6$ kPa	$11.13\cdot10^6$ kPa	$11.65\cdot10^6$ kPa	$11.13\cdot10^6$ kPa
$\boldsymbol{d}$	0.1232 m	0.2099 m	0.1232 m	0.2099 m
$\boldsymbol{\upsilon_{lc}}$	0.26	0.26	0.26	0.26

Table A 1 Summary of model parameters adopted by the NL tunnels in the "Full NL stiffness" and "Original" analyses