

The influence of tunnel joints on the present-day condition of a grey cast iron tunnel

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ABSTRACT

The present-day condition of a grey cast iron (GCI) tunnel was investigated numerically by means of a series of plane-strain geotechnical analyses that focused on establishing the influence of the modelling approach adopted to simulate the behaviour of longitudinal tunnel joints. The GCI tunnel lining was simulated in three different ways: (i) as a continuous lining; (ii) as a jointed lining adopting a simple joint model that neglects the presence of the joint bolts and (iii) as a jointed lining adopting a more sophisticated model that considers both the nonlinear response of the joint after opening and the contribution of the bolts to the ultimate capacity. The numerical results demonstrate that accounting for the tunnel joints has a considerable impact on the tunnel response. Significant differences are also observed in the results obtained with the two joint models, hence demonstrating the practical significance of employing an advanced model able to reproduce accurately the rotational behaviour of the joint. The drainage behaviour of the joints was also investigated and the numerical results indicate that drainage even through only the knee joints causes significantly more soil consolidation than that from a watertight tunnel and consequently, larger tunnel deformations and forces.

1. Introduction

Segmental grey cast iron (GCI) tunnels were historically used during the early development (19th and early 20th century) of metropolitan underground railway systems, e.g., New York, Hong Kong, Beijing (Yu, 2014). In London, they constitute the largest fraction of the deep tube lines. Although GCI has nowadays been replaced by concrete segments for new tunnel construction, hundreds of kilometres of GCI linings remain operational, and are inevitably impacted on by all new construction works in their vicinity. On such occasions, the potential detrimental effects on the structural integrity of the GCI tunnel must be carefully assessed before proceeding with the works. The structural assessment must provide confidence that the tunnel deformations and stresses will remain within the limits prescribed by the tunnel owner as well as establishing whether mitigation measures need to be implemented. The evaluation of the present-day condition of the existing tunnel is crucial for an accurate structural assessment, as it largely determines the available capacity for additional movements and stresses.

It is well understood that GCI tunnel linings in London Clay have generally squatted over time. Squatting refers to the deformation mode where the horizontal diameter enlarges while the vertical diameter

shortens (the opposite mode is denoted as egging). Wright (2013) reported an average tunnel squat (horizontal diametric strain) between 0.5 and 1% obtained from circularity surveys conducted along running tunnels of the Northern, Piccadilly and Jubilee Lines, which are mostly lined with GCI tunnel linings. In fact, TfL (2017) recommends adopting a tunnel squat of 1% in tunnel assessments where the present-day ovalisation of the tunnel in question has not been measured.

There seems to be a relationship between tunnel squatting and drainage of water into the tunnel, as the circularity surveys mentioned above revealed that tunnel sections subjected to special grouting measures, and so were relatively watertight, remained closer to circular, or even had gone into egging, compared with other tunnels. Field measurements of tunnel ovalisation over time confirm that GCI tunnels indeed squat over time, presumably as a result of soil consolidation (Ward & Thomas, 1965; Thomas, 1976).

While measuring the present-day ovalisation of any given tunnel is relatively straightforward, the lining stresses can rarely be measured as that would require segments to be removed from the tunnel ring. The few measurements available on GCI tunnel linings show significant variability of the stress magnitudes compared to the corresponding value calculated assuming the full overburden acting on the tunnel.

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Ward & Chaplin (1957) determined the (circumferential) axial stresses obtained from pairs of old GCI tunnel linings (more than 50 years old) from four different sites in London. Most of the stresses were lower than the value corresponding to the full overburden although some of the reported values were up to four times larger. Overall, the stresses tended to be larger at the springline than at the crown and invert. More recently, Yu et al. (2015) took measurements of axial strains before and after dismantling a number of segments from a station tunnel at Tottenham Court Road. The corresponding axial forces ranged between 30% above and 63% below that calculated assuming the full overburden value. Additionally, the measurements indicated negative bending (tension at extrados) at the springline which is consistent with the bending mode expected for a squatting tunnel at that location.

Since the present-day stresses of any given tunnel cannot generally be measured, recourse must be made to calculation. While structural models such as the elastic continuum model (Duddeck & Erdmann, 1985) can be useful in providing preliminary estimates of the internal forces, their consideration of the interaction between the lining and the ground is too simplistic. Geotechnical finite element (FE) models (where the ground is explicitly included) are more appropriate for assessing the tunnel present-day condition as they can realistically simulate the soil-tunnel interaction incorporating aspects such as complex soil behaviour and coupled consolidation. As per CIRIA 791 (O'Brien & Higgins, 2020), it is good practice to first perform 2-dimensional (2D) FE analyses before conducting any 3-dimensional (3D) analyses should these be required. Therefore, developing numerical strategies on how to accurately incorporate the effect of joints in segmental GCI tunnel linings in 2D geotechnical FE analysis is of both relevance and significance.

The numerical simulation of segmental GCI tunnel linings involves the additional difficulty of considering appropriately the effect of the joints on the structural behaviour. While this has been attained in 3D structural models (Li et al., 2014; Tsiamposi et al., 2017; Ruiz López et al., 2022) it is yet to be incorporated within geotechnical models. Avgerinos (2014) conducted plane-strain analyses of the long-term behaviour of the Central Line GCI tunnels beneath Bayswater Road simulating the presence of the joints with a constitutive model introduced by Potts and Zdravkovic (2001) specifically for accounting for the presence of joints. Nonetheless, the bending moments generated did not seem large enough (the maximum squatting only reached about 0.2%) for the nonlinear component of the model to be invoked.

Li et al. (2015) performed 3D geotechnical FE analyses of the long-term behaviour of a cross-passage between two GCI tunnels. The tunnel joints were not considered in the geotechnical analysis, but only included in a subsequent structural analysis where the soil stresses obtained from the geotechnical model were applied, neglecting the potential influence of the joints on them. The tunnel lining was considered fully permeable and the soil permeability was isotropic. The resulting tunnel squatting was limited to around 0.04%, possibly because of the assumption of isotropic permeability.

Mair (2008), discussing the results from a series of plane-strain geotechnical analyses with different soil and tunnel permeabilities, showed that the analyses with higher permeability linings and larger soil permeability anisotropy ratios produced greater tunnel squatting. The maximum squatting was around 0.4%, slightly below the expected range (0.5–1%) for tunnels in London.

While these numerical studies adopted a uniform condition all around the lining, it is more likely that drainage into the tunnel occurs along preferential paths through the joints (Mair, 2008) and so it is important to consider the effect of localised drainage on the tunnel response, in addition to modelling adequately the segmental nature of the lining and the behaviour of the joints.

Both aspects are considered in the series of plane-strain finite element (FE) analyses presented here, which were conducted with the Imperial College Finite Element Program – ICPEP (Potts and Zdravkovic, 1999) and focused on the present-day condition of a GCI tunnel lining. The novelty of the paper lies in comparing different numerical modelling

strategies to account for the segmental nature of GCI tunnel linings in 2D geotechnical FE analysis: adopting a nonlinear joint model which accounts both for the stiffness degradation resulting from joint opening and for the presence of bolts (Ruiz López, 2022), or adopting a joint model which neglects them both (Potts & Zdravkovic, 2001). The two approaches are compared between themselves and with the case where the presence of joints is entirely disregarded. The analyses results demonstrate that accounting for the joints and the model adopted to simulate their behaviour can be crucial in the assessment of the tunnel present-day bending moments.

Furthermore, the long-term behaviour of the lining is studied under different drainage conditions: the more realistic scenario of drainage through the joints is compared to the most commonly adopted, but less realistic, scenarios where the lining is either fully permeable or of finite permeability. The results show that drainage through the joints causes greater soil consolidation around the tunnel compared to the finite permeability lining, albeit less than the fully permeable case, leading to substantially different present-day tunnel deformations and forces.

2. Description of the numerical model

2.1. Soil stratigraphy and model geometry

The stratigraphy employed in the analyses is illustrated in Fig. 1. It follows that adopted by Avgerinos (2014) in his analyses of the Central Line tunnels and can be considered typical of a Central London site. The stratigraphy includes London Clay units B2, A3 and A2 that sum a total of 57 m. These are overlain by 3 m of superficial deposits. 6 m of Upper Lambeth Group and Lower Lambeth Group are present below the London Clay A2 unit. A 3.81-m external diameter tunnel with its crown at 20 m depth was considered. The geometry and dimensions of the longitudinal joint and tunnel segments cross-section are depicted in Fig. 2. The tunnel lining geometry is based on the Central Line running tunnels beneath Bayswater Road and can be considered standard for GCI running tunnels of the London Underground (LU) network. Each tunnel ring is formed with six segments and one smaller key segment located at the crown. The section of the Central Line tunnels beneath Bayswater Road was constructed with the longitudinal joints adopting a continuous configuration (Yu, 2014), i.e. joints aligned between subsequent rings, hence allowing plane-strain conditions to be applicable, at least in regard to the mechanical response of the tunnel. While the continuous configuration was initially standard, tunnels adopting a staggered configuration (i.e. where the longitudinal joints were rotated relative to each other) became more common by the early 1900s (Craig and Muir Wood, 1978). The simulation of the latter configuration would require a

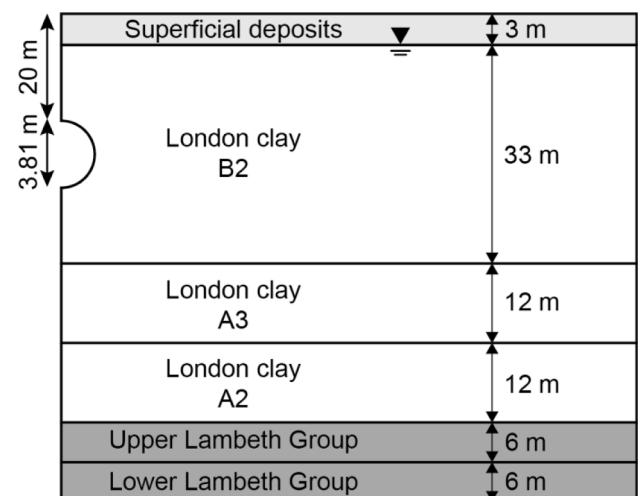


Fig. 1. Stratigraphy adopted in the numerical model.

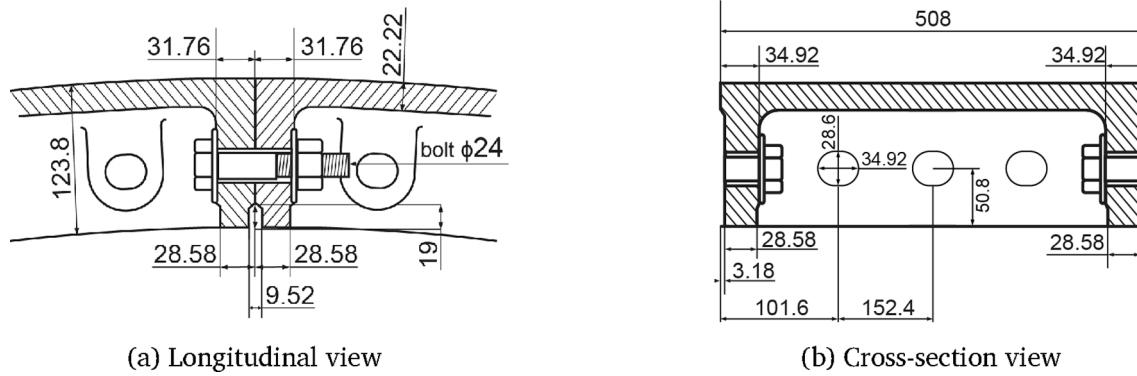


Fig. 2. Geometry and dimensions of the joints and segments cross-section corresponding to the 3.81-m external diameter tunnel. All dimensions in mm.

3D model including a succession of several rings, appropriately connected, along the tunnel axis. The longitudinal joints were placed (with 0° set at the crown and increasing clockwise) at 5° , 59° , 120° and 180° and were simulated with 2° arc length beams. The whole FE mesh used in the analyses is shown in Fig. 3 along with a detailed view of the discretisation around the tunnel cavity. The width of the model was set equal to 100 tunnel diameters which was deemed sufficiently large for boundary effects not to alter the response. 8-noded and 4-noded quadrilateral solid elements were adopted for the mechanical and seepage fields, respectively. Additionally, 3-noded beam elements were employed to simulate the mechanical response of the tunnel lining. A full bond, i.e., no relative movement, was assumed between the soil and the lining. According to the numerical analyses conducted by Avgerinos (2014), the interface condition between the soil and the lining does not affect the deformation mode experienced by the tunnel although having adopted a tangential slip condition (i.e. no tangential stresses between the soil and the lining) might have resulted in slightly larger long-term deformations.

2.2. Soil constitutive modelling

The behaviour of London Clay was simulated with a nonlinear transverse isotropic stiffness model along with the non-associated perfectly-plastic Mohr-Coulomb failure criterion. The transverse isotropic stiffness model was first utilised by Zdravković et al. (2021) for the analysis of laterally-loaded monopiles in London Clay; the model is an extension of that proposed by Taborda et al. (2016) incorporating the three-parameter transverse isotropic formulation of Graham & Houlsby (1983), in similar manner to the earlier transverse isotropic model of Franzius et al. (2005).

The elastic stiffness of a transverse isotropic material is defined by

seven elastic parameters: the elastic moduli in the horizontal and vertical directions, E_h and E_v , respectively; the shear moduli in the horizontal and vertical planes, G_{hh} and G_{vh} , respectively; Poisson's ratios of horizontal strains due to vertical and horizontal strains, μ_{hv} and μ_{hh} , respectively, and Poisson's ratio of vertical strains due to horizontal strains μ_{vh} . Since the horizontal plane is isotropic the following relationship applies:

$$G_{hh} = \frac{E_h}{2(1 + \mu_{hh})} \quad (1)$$

Moreover, the following equality holds due to thermodynamic considerations:

$$\frac{\mu_{hv}}{E_h} = \frac{\mu_{vh}}{E_v} \quad (2)$$

Furthermore, Graham & Houlsby's (1983) transverse isotropic formulation establishes the following interdependence between elastic parameters through the stiffness anisotropy ratio α :

$$\alpha = \sqrt{\frac{E_h}{E_v}} = \frac{\mu_{hh}}{\mu_{vh}} = \frac{G_{hh}}{G_{vh}} \quad (3)$$

so that, the stiffness behaviour at small strains can effectively be described with only three elastic parameters. In this case, the stiffness anisotropy ratio α , the shear modulus in the vertical plane G_{vh} and the Poisson's ratio of horizontal strains due to horizontal strains μ_{hh} were prescribed while the remaining elastic parameters (at small strains) were determined from Eqs. (1)–(3).

The three London Clay units were simulated as a single unit and so adopted the same model parameters, which were calibrated on tests reported by Gasparre (2005), Gasparre et al. (2007) and Hight et al.

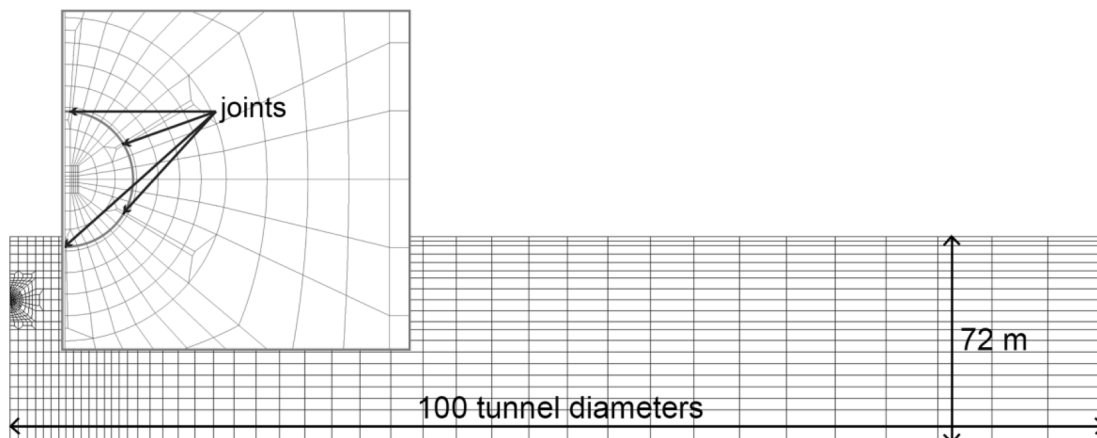


Fig. 3. Mesh discretisation adopted in the numerical model.

(2007). The respective model parameters are summarised in Table 1 and details of the calibration can be found in Ruiz López (2022).

Fig. 4 depicts the values for the elastic parameters, at small strains, that were adopted here along with laboratory data corresponding to samples from different depths. The laboratory data of anisotropy ratio α (Fig. 4a) reveal that the ratio $\sqrt{E_h/E_v}$ is smaller than G_{hh}/G_{hv} with depth; additionally, the ratio $\sqrt{E_h/E_v}$ tends to increase with depth whereas no clear trend can be established from the G_{hh}/G_{hv} data. The model anisotropy ratio α was adopted as 2 which provided a better match for the shear moduli ratio than the elastic moduli ratio. Preference was given to the former since its determination is more reliable (Mašin & Rott, 2014). It is worth noting that Hight et al. (2007) also reported a value of 2 for the anisotropy ratio G_{hh}/G_{hv} determined from shear wave velocity measurements taken at the T5 site. Values of 200 and 400 were adopted for G_{vh}/p' and G_{hh}/p' . As shown in Fig. 4c, significant scatter can be observed in the laboratory data of the three Poisson's ratios (μ_{hv} , μ_{vh} and μ_{hh}) which reflects the challenges involved in determining consistently Poisson's ratio over the small strain range. The value of μ_{hh} was set equal to 0.1 so that according to Equations (2) and (3) the values for μ_{hv} and μ_{vh} were determined as 0.2 and 0.05, respectively.

The decay of G_{vh} with strain is presented in Fig. 5 along with the laboratory results and the corresponding curve of G_{hh} . The cohesion c' was taken as 5 kPa whereas the angle of shearing resistance φ' was adopted as 25° ; a zero angle of dilation $\psi = 0^\circ$ was adopted in order to prevent the soil from dilating indefinitely after yielding. Fig. 6 shows the p' - q stress paths of five undrained triaxial compression tests pre-consolidated to different states. Unlike models adopting isotropic stiffness, the analysis captures the initial inclination of the p' - q paths towards the left observed in the tests and the subsequent reduction in inclination upon shearing, the latter is realised by adopting a varying anisotropy ratio α which reduced from its value at small strains $\alpha_{ss} = 2$, to that at large strains $\alpha_{ls} = 1$.

Fig. 7 presents the analysis results of three oedometer tests compared to the laboratory results reported by Hight et al. (2007). The numerical simulation yields a similar compressibility to the laboratory data for tests O7 (C unit) and O35 (A3 unit), whereas it exhibits a softer response at early loading than that observed in test O28 (B2 unit). Overall, a reasonable simulation of the laboratory results in the range of effective stresses (up to 4000 kPa) considered in the analyses was achieved.

The constitutive modelling of the superficial deposits and Lambeth Group layers broadly followed that of Avgerinos (2014). The superficial deposits were simulated as linear elastic in conjunction with a non-associated perfectly-plastic Mohr-Coulomb model. An elastic modulus of 10 MPa and a Poisson's ratio of 0.2 were employed, the Mohr-Coulomb model adopted a cohesion of $c' = 0$ kPa, angle of shearing resistance $\varphi' = 25^\circ$ and a dilation angle of $\psi = 0^\circ$. The upper and lower Lambeth Group layers were simulated with the isotropic nonlinear elastic model of Taborda et al. (2016) along with the Mohr-Coulomb model. The input parameters of the Taborda et al. (2016) model were derived from those employed by Avgerinos (2014) who used an earlier nonlinear elastic model (Jardine et al., 1986). The parameters of this earlier model were converted to those of the Taborda et al. (2016) model following the procedure outlined by Measham et al. (2014), the resulting parameters are summarised in Table 2 and Table 3. For the Upper Lambeth Group cohesion of $c' = 10$ kPa, angle of shearing resistance $\varphi' = 28^\circ$ and dilation angle $\psi = 0^\circ$ were adopted while the Lower Lambeth Group was given a cohesion of $c' = 0$ kPa, angle of shearing resistance $\varphi' = 36^\circ$ and dilation angle $\psi = 18^\circ$.

Table 1
Transverse isotropic model parameters for London Clay.

$G_{0,vh}(\text{kPa})$	$p'_{ref}(\text{kPa})$	m_G	$R_{G,min}$	$G_{min}(\text{kPa})$	a	b	α_{ss}	α_{ls}	a_α	b_α
200	1.0	1.0	0.14	2667	5×10^{-4}	1.5	2.0	1.0	0.002	2.0

2.3. Tunnel lining modelling

The GCI tunnel segments were simulated with linear elastic material behaviour. The geometric and material parameters assigned to the segments are summarised in Table 4. The response of the tunnel joints was simulated in three different ways: adopting linear elastic behaviour with the same parameters as the tunnel segments so that a continuous lining was effectively simulated; using the joint model, referred to subsequently as non-bolted joint (NBJ) model, first introduced in Potts & Zdravkovic (2001); and finally, adopting the joint model recently introduced by Ruiz López (2022) and referred to as bolted joint (BJ) model. A summary of the formulations of the two joint models is included in Appendix A.

The NBJ model is an elastoplastic model to be assigned to a small beam element simulating the joint. The bending moment is bounded by the ultimate bending moment which depends on the axial force magnitude and is computed assuming compressive failure. The contribution of the bolts is not considered in the calculation of the ultimate bending moment. Before reaching the ultimate capacity, the joint behaves according to the bending stiffness of the beam and does not incorporate the stiffness decay after opening of the joint. Consequently, a bilinear bending moment-rotation (M - θ) relationship is realised for any given axial force, as shown in Fig. 8.

The BJ model is also meant to be adopted by a small beam element. It incorporates the most relevant features of the behaviour exhibited by GCI longitudinal joints, which have been identified by Ruiz López et al. (2023a), namely the nonlinear M - θ response of the joint from opening to the ultimate capacity and the influence of the bolts on the ultimate capacity. The model also considers the dependency of the response on the axial force magnitude. This is also shown in Fig. 8.

Fig. 8 demonstrates the superiority of the BJ over the NBJ model: the M - θ responses obtained with the two models for positive and negative bending under varying axial forces are benchmarked against the M - θ curves obtained by detailed 3D analyses by Ruiz López et al. (2023a). The three axial force levels shown correspond to tunnel depths of 6 m (118 kN), 12 m (236 kN) and 24 m (472 kN) when assuming that the full overburden stress acts all around the tunnel. The BJ model predicted the 3D joint response very well from opening to the end of the capacity for both positive and negative bending. Conversely, the NBJ model failed to reproduce the expected behaviour: it overestimated the bending moments at low rotations (particularly under larger axial forces) and largely underestimated the ultimate capacity of the joint. These results clearly demonstrate the large differences between the two joint models in terms of the local M - θ response at the joint locations. The analyses in Section 3, demonstrate the significant impact of these differences on the global tunnel response in a geotechnical boundary value problem.

In addition to the elastic parameters included in Table 4, the NBJ model requires the definition of the ultimate compressive strength f_c of the joint material, which was taken as 560 MPa, and the input of the angle of friction between the tunnel segments, which was adopted as 40° and which defines the shear force envelope. Table 5 lists the input parameters of the BJ model; note that the rotational stiffnesses assigned to the 180° joint were doubled to account for the symmetry condition at that location.

2.4. Initial conditions and soil permeability

The profile of the coefficient of earth pressure at rest K_0 employed in the analyses is presented in Fig. 9 where the soil stratigraphy and the

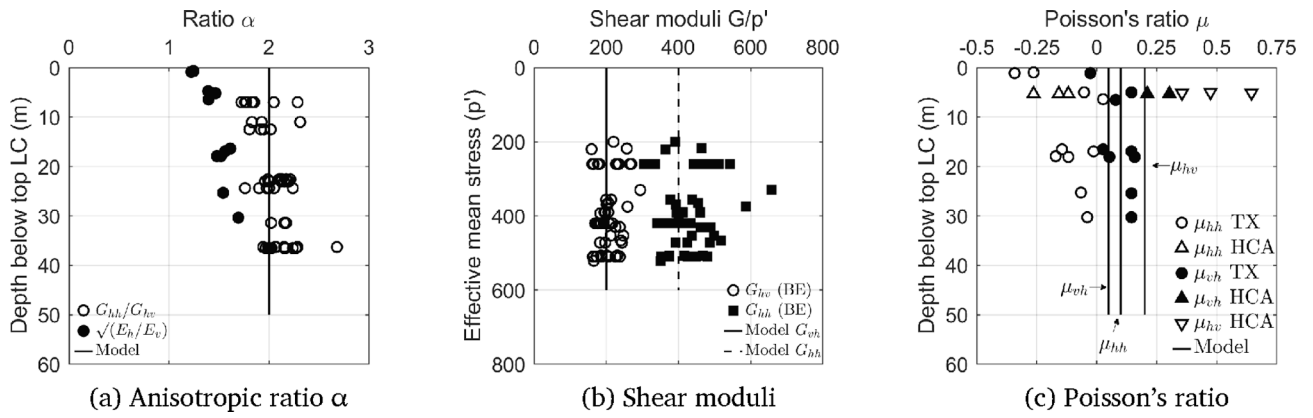


Fig. 4. London Clay elastic properties in the small strain range with depth, experimental data from Gasparre (2005) and Gasparre et al. (2007).

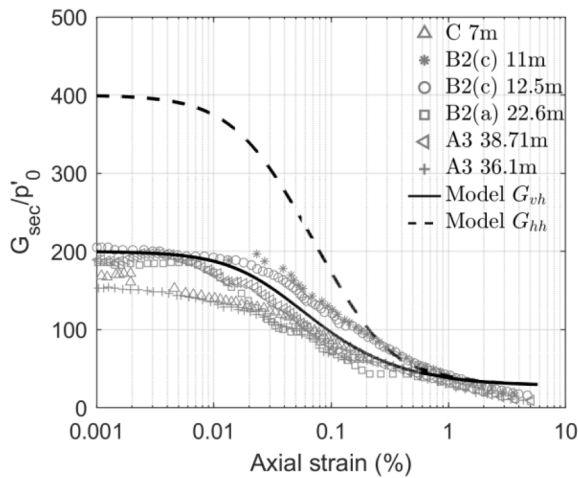


Fig. 5. Shear stiffness degradation with strain, laboratory data from Hight et al. (2007).

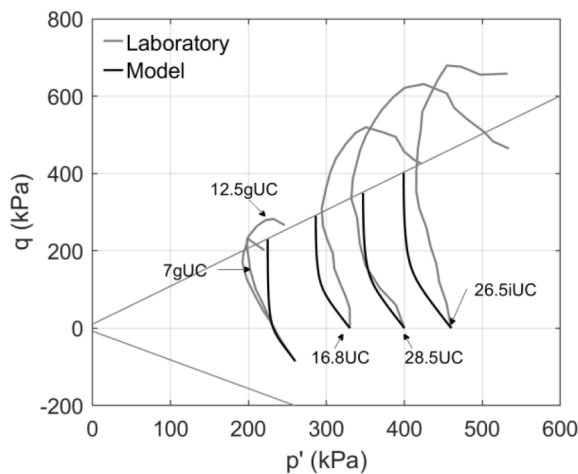


Fig. 6. Effective stress paths of undrained triaxial compression tests, laboratory data from Gasparre (2005).

depth of the tunnel are indicated for reference. The value of K_0 reduces linearly from 1.5 at a depth of 17.4 m to a value of 1.0 at a depth of 36 m. Correspondingly K_0 is equal to 1.38 at the depth of the tunnel springline. Initial hydrostatic conditions were assumed in the analyses with the phreatic level set at a depth of 3 m. Fig. 10 presents field data of horizontal permeability k_h values from several sites across London collected

by Hight et al. (2007). The field data exhibit significant scatter with values varying up to three orders of magnitude, even at the same depth. For this reason, it is not obvious which permeability values and permeability anisotropy ratio k_h/k_v should be adopted in numerical analysis despite several studies having examined the long-term movements due to tunnelling in London Clay. In the analyses considering permeability anisotropy, Mair (2008) adopted an anisotropy ratio k_h/k_v of 4, while Wongsaraj et al. (2013) considered ratios k_h/k_v of 2 and 10. Avgerinos et al. (2016), in their modelling of the St. James's Park case study, adopted $k_h/k_v = 2$ for the whole depth except for the top of the London Clay A3 unit where values of 2, 25 and 100 were considered. In this investigation, an anisotropic permeability model that enables permeability to vary logarithmically with depth was adopted along with k_h/k_v equal to 8. The model defines the variation of permeability with the following expression:

$$k = k_0 \cdot 10^{G_y \cdot (y - y_0)} \quad (4)$$

where k_0 is the permeability of the reference vertical coordinate y_0 , G_y is the parameter governing the variation of permeability with vertical coordinate y . The profiles of k_v and k_h with depth adopted in the analyses are shown in Fig. 10 alongside the field data. The reference vertical coordinate in Equation (4) was set at the top of London Clay, k_0 and G_y for the vertical permeability profile were adopted as $5 \cdot 10^{-10}$ m/s and 0.025 m^{-1} , respectively, whereas they were taken as $40 \cdot 10^{-10}$ m/s and 0.025 m^{-1} for the horizontal permeability profile. These parameters resulted in the permeability profiles decreasing with depth, as shown in Fig. 10.

2.5. Boundary conditions

Regarding the mechanical boundary conditions, the horizontal displacements of the vertical boundaries of the model were restrained whereas vertical and horizontal displacements were fixed along the bottom of the model. Additionally, the rotation degrees of freedom of the tunnel lining nodes located at the symmetry plane were restricted. The tunnel excavation was performed reducing the nodal forces (corresponding to the initial total stresses) incrementally acting all around the boundary of the tunnel cavity while an outward radial isotropic pressure applied on the same boundary was progressively ramped up so that by the end of the excavation only the radial pressure was active. The construction of the tunnel lining was then carried out by activating the beam elements representing the tunnel lining (deactivated prior to the start of the analysis) and releasing the radial pressure such that the tunnel lining became loaded. This modelling procedure was suggested by Potts & Zdravkovic (2001) for the analysis of tunnelling in soft clays and is also similar to the grout pressure method proposed by Möller and Vermeer (2008). The magnitude of the radial pressure was adjusted through trial-and-error until a volume loss of about 1.5% of the tunnel

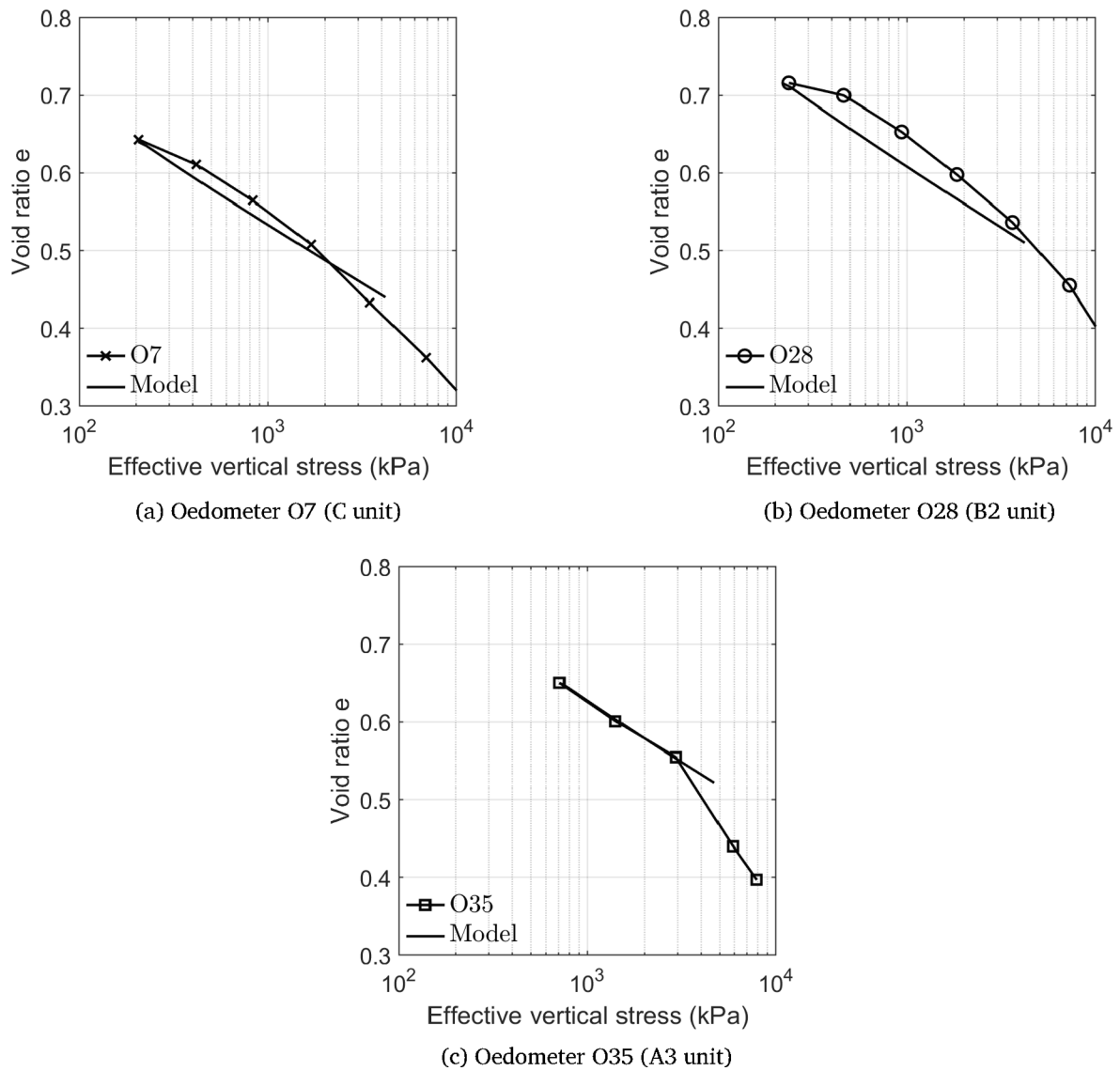


Fig. 7. Simulation of oedometer tests from Hight et al. (2007).

Table 2

Shear stiffness parameters of the Taborda et al. (2016) model for the Lambeth Group layers.

Material	G_0 (kPa)	p'_{ref} (kPa)	m_G	$R_{G,min}$	a	b	G_{min} (kPa)
Upper Lambeth Group	334.5	1.0	1.0	0.067	1.12×10^{-4}	1.2	2000
Lower Lambeth Group	377.0	1.0	1.0	0.09	9.5×10^{-5}	1.04	2000

Table 3

Bulk stiffness parameters of the Taborda et al. (2016) model for Lambeth Group layers.

Material	K_0 (kPa)	p'_{ref} (kPa)	m_K	$R_{K,min}$	r	s	K_{min} (kPa)
Upper Lambeth Group	300.0	1.0	1.0	0.096	6.5×10^{-5}	1.1	2500
Lower Lambeth Group	449.7	1.0	1.0	0.085	2.4×10^{-5}	1.1	2500

cross-section area was obtained. The applied radial pressure can be representative of a grouting pressure (GCI tunnel linings were usually grouted as discussed by Craig & Muir Wood (1978)). Moreover, the

Table 4

Geometric and material parameters adopted by the GCI tunnel segments.

Parameter	Symbol	Value
Unit weight	γ	69.26 kN/m ³
Elastic modulus	E	$100 \cdot 10^6$ kPa
Poisson's ratio	μ	0.26
Cross-section area	A	0.01838 m ²
Second moment of area	I	$2.3267 \cdot 10^{-5}$ m ⁴
Shear correction factor	κ_s	0.8

application of this method is advantageous for the simulation of the tunnel deformations and forces since it leads to a post-construction uniform contraction of the tunnel lining rather than the egging deformation mode that would be attained with the standard stress release

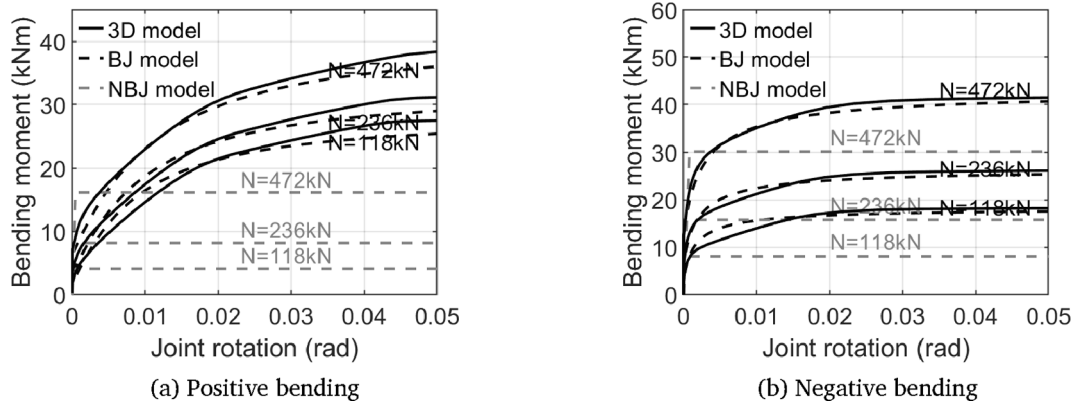


Fig. 8. $M-\theta$ response of the BJ and NBJ models along with that predicted by the 3D analyses of Ruiz López et al. (2023a) for a range of axial force levels.

Table 5

Summary of parameters of the BJ model.

Parameter description	Symbol	Value
Elastic modulus	E	100 GPa
Shear modulus	G	39.68 GPa
Cohesion for maximum shear force	c	137.98 MPa
Angle of friction for maximum shear force	δ	40°
Uniaxial tensile strength of bolts	f_{ub}	239 MPa
Rotational stiffness closed joint in positive bending (per ring)	K_o^+	35,748 kNm/rad
Rotational stiffness open joint in positive bending (per ring)	K_i^+	2520 kNm/rad
Shape parameter for positive bending	n^+	1.25
Rotational stiffness closed joint in negative bending (per ring)	K_o^-	35,748 kNm/rad
Rotational stiffness open joint in negative bending (per ring)	K_i^-	8909 kNm/rad
Shape parameter for negative bending	n^-	0.87
Width section dimension (per ring)	b_1	0.5080 m
Height section dimension	h_1	0.0222 m
Width section dimension (per ring)	b_2	0.0698 m
Height section dimension	h_2	0.1016 m
Depth of caulking groove	h_U	0.0190 m
Diameter of bolt thread	ϕ_b	0.0240 m
Number of bolts row 1 (per ring)	n_{b1}	3
Distance from intrados to bolt row 1 centreline	h_{G1}	0.0508 m

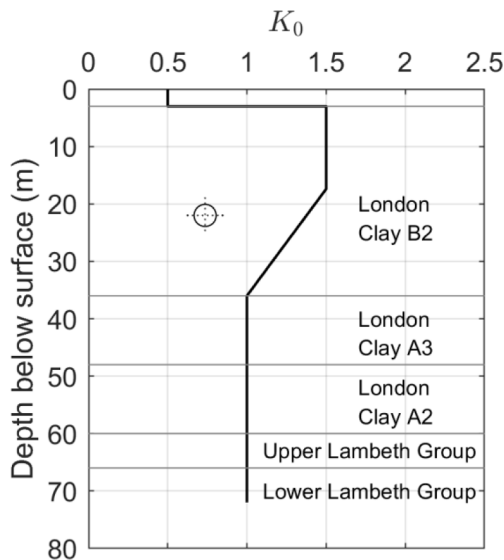


Fig. 9. Initial profile of coefficient of earth pressure at rest K_0 with depth.

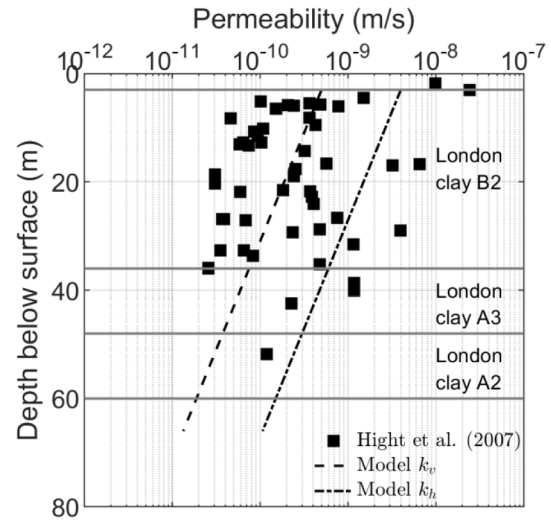


Fig. 10. Permeability profile with depth adopted in the numerical model along with the field data collected by Hight et al. (2007).

procedure (Potts & Zdravkovic, 2001) under the stress conditions assumed in the analyses (i.e., K_0 greater than 1). As explained above, egging is usually not observed in the tunnels of the LU network (Wright, 2013).

The superficial deposits and the Lower Lambeth Group layers were assumed to be drained and therefore a zero pore pressure change was prescribed in the interfaces between those layers and the consolidating layers, i.e. London Clay and Upper Lambeth Group. A zero pore pressure change was also prescribed in the far-field vertical boundary and a non-flow condition was applied along the symmetry plane vertical boundary. A dual hydraulic boundary condition, the so-called precipitation boundary condition Potts & Zdravkovic (1999), was applied along the tunnel boundary. It was adopted in such a way that either a zero pore pressure or a zero flow boundary were applied at the nodes located at the tunnel boundary. At the start of every increment, the program checks the pore pressure magnitudes: if the magnitude at any given node is compressive then it applies a zero pore pressure condition whereas a non-flow boundary condition is applied at that node if the magnitude is tensile. A non-flow pore pressure condition may apply shortly after the tunnel excavation in some areas around the cavity where suctions are developed, hence preventing the unrealistic simulation of water flowing from the tunnel into the soil. The zero pore pressure condition should prevail in the long-term response as a consequence of the dissipation of excess pore pressures taking place during consolidation.

A fully permeable tunnel lining, where the lining permeability was

neglected, was adopted as the base condition for this numerical investigation. Nonetheless, the sensitivity of the tunnel response to different tunnel drainage conditions was examined as part of a sensitivity study (presented in Section 4). In all analyses other than those adopting a fully permeable lining, the lining permeability was simulated following the approach proposed by Shin et al. (2002), where a certain finite permeability value, representative of the structure, was assigned to the first layer of solid elements surrounding the tunnel boundary, see Fig. 11. The magnitude of the tunnel segment permeability was taken as 1% of $k_{soil} = \sqrt{k_v \cdot k_h}$ at the depth of the tunnel springline. Likewise, the solid elements next to the joint locations adopted a permeability equal to 1% of k_{soil} when simulating a tunnel lining with finite permeability. The effect of preferential drainage paths through the joints was simulated by assigning the soil permeability to the solid elements adjacent to the joints which effectively meant a fully permeable condition was applied at the joint locations. It is worth noting that since the analyses assumed plane-strain conditions, the potentially significant effects of drainage through the circumferential joints on the flow regime were not considered. Accounting for such effects could be the subject of future research with 3D modelling. The analyses also did not account for potential preferred flow along the interface between the ground and the tunnel extrados (or grout layer); while this might influence the tunnel response, it would be difficult to quantify and hence to simulate accurately.

2.6. Simulation cases and analysis procedures

The numerical investigation presented in subsequent sections comprised a total of seven analyses. These included three analyses adopting a fully permeable lining and simulating the tunnel lining as either continuous or jointed adopting the BJ and the NBJ models from which the impact of the approach taken to simulate the segmental lining could be evaluated (Section 3). Four additional analyses were undertaken adopting the BJ model to simulate the behaviour of the joints and the following tunnel drainage conditions: tunnel lining with finite permeability, tunnel segments adopting finite permeability and tunnel joints as fully permeable, 120° joint (knee joint) as fully permeable and the rest of the lining adopting finite permeability and lastly, 59° (shoulder joint) and 120° joints as fully permeable and the rest of the lining adopting finite permeability. The responses from these analyses were also compared to that obtained with the fully permeable tunnel lining condition (Section 4).

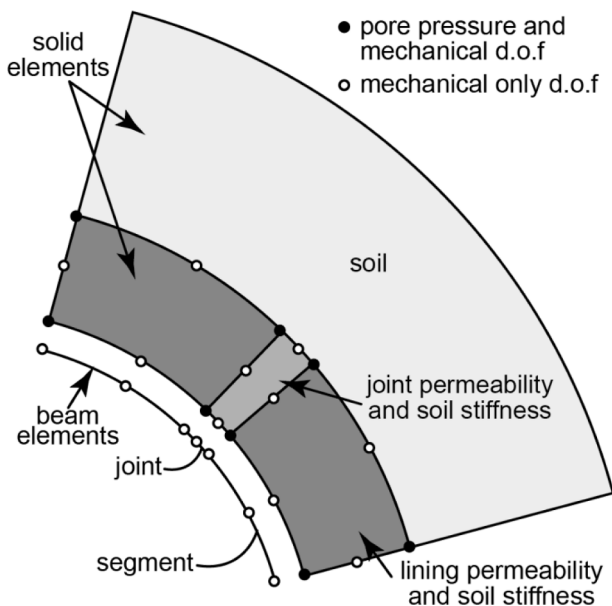


Fig. 11. Model set-up adopted to simulate the tunnel lining finite permeability.

Coupled consolidation was incorporated throughout the analyses. The tunnel construction time was defined as 36 h which was approximately the time span over which changes in the surface ground movements, as reported by Nyren (1998), could be observed at the monitoring section during the construction of the Jubilee Line tunnel at St. James's Park and which was adopted by Ruiz López et al. (2021) when studying the short-term ground response to tunnelling. A total consolidation period of 130 years, meant to extend from 1892 to 2022, was simulated after tunnel construction.

3. Effect of the segmental lining modelling approaches

3.1. Pore pressure response

The long-term tunnel response is largely governed by the difference in the pore pressure distribution between the end of tunnel construction and present day, both of which are discussed as follows. The pore pressure response is not affected by the modelling approach adopted for the segmental lining and so these results are applicable to the three analyses considered. Fig. 12a depicts the vertical pore pressure profile from surface to the tunnel crown (left figure) and the horizontal pore pressure with distance from the tunnel springline (right figure) corresponding to the end of construction. Above the crown, the pore pressures gradually deviate with depth from the initial hydrostatic profile from a depth of about 10 m and then reduce abruptly to zero at the tunnel crown. To the side of the tunnel, the pore pressures are zero at the tunnel lining but quickly become slightly larger than hydrostatic and remain so for about 20 m from the springline. Ruiz López et al. (2023b) showed, by comparing the results with those obtained with an analogous isotropic stiffness model, that such an increase with respect to the initial hydrostatic conditions can be mostly attributed to the effect of stiffness anisotropy on the soil response; the soil adjacent to the springline is subjected to stress conditions similar to triaxial compression along which, as shown in Fig. 6, compressive pore pressure changes are generated when $\alpha > 1$. Fig. 12b presents the pore pressure profiles at present day. The pore pressure distribution at present day is governed by the hydraulic boundary conditions, lining drainage condition and the soil permeability profile. The consolidation process is primarily driven by the tunnel acting as a drain and results in substantial soil consolidation around the tunnel from the end of construction to present day. The distribution of pore pressure changes between the two time points is presented in Fig. 13 where it can be observed that significantly more soil consolidation takes place on the side of the tunnel than above and below it. Consequently, soil contraction is more prominent next to the tunnel springline which is the cause for the tunnel going into squatting as described below.

3.2. Structural response

Fig. 14 shows the distribution of internal forces and deformations around the tunnel at present day. As mentioned previously, the tunnel deforms into the squatting mode for the three tunnel lining models (Fig. 14a) and consequently positive bending moments (tension at the intrados) occur around the crown and invert while negative bending moments (tension at the extrados) develop around the springline, see Fig. 14b. As a result of the opening of the 5° and 180° joints, which are the only joints opening (at the intrados), the two analyses considering joints exhibit smaller bending moment magnitudes than the continuous lining around the crown and invert. The differences between the three analyses are marked: when the BJ model is employed the bending moments are about 172% and 223% at the crown and invert of those predicted with the NBJ model, whereas the continuous lining analysis yields bending moments around 186% and 157% of those given by the analysis adopting the BJ model. The effect of joint opening can be seen in the bending moment distributions from 0° to about 45° (and 135°–180°), whereas further away from the joints, from around 45° to 135°,

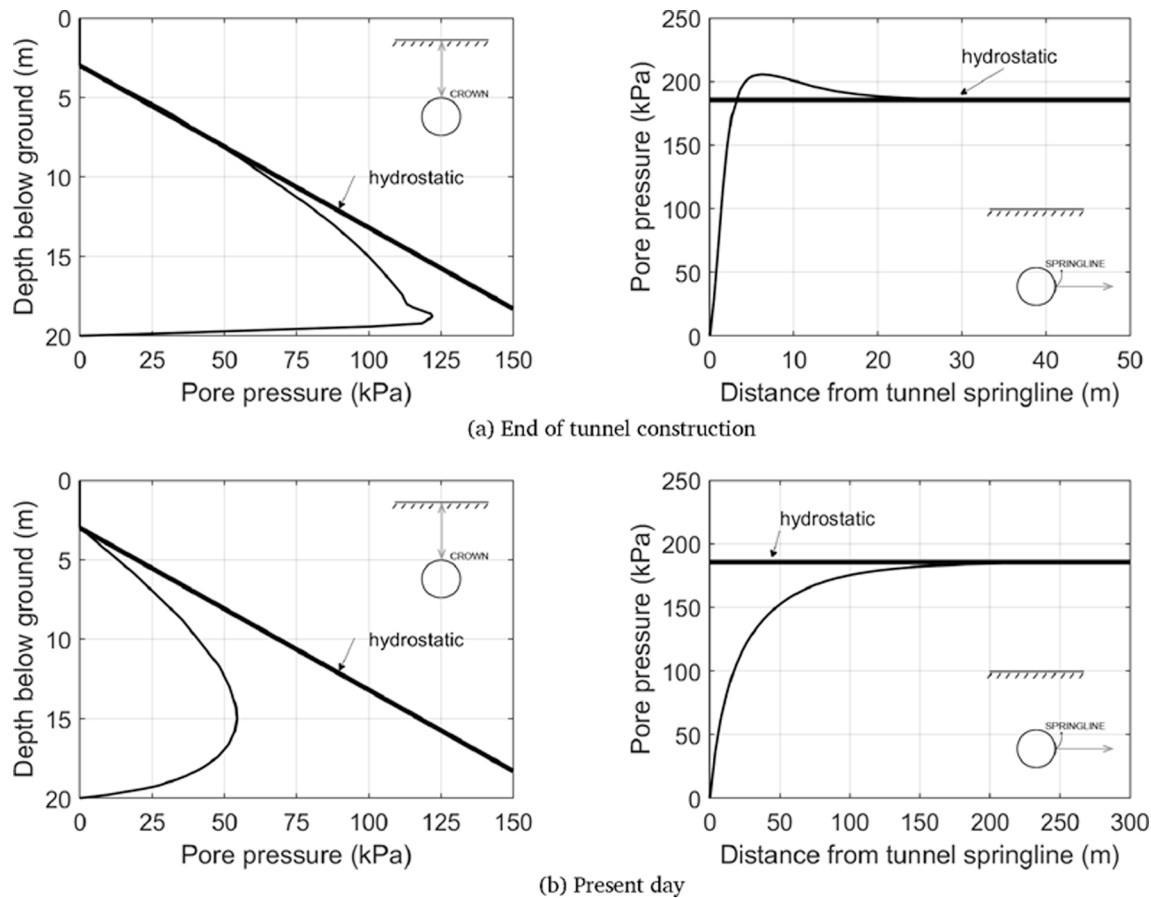


Fig. 12. Pore pressure profiles above the tunnel crown (left) and with distance from the tunnel axis (right).

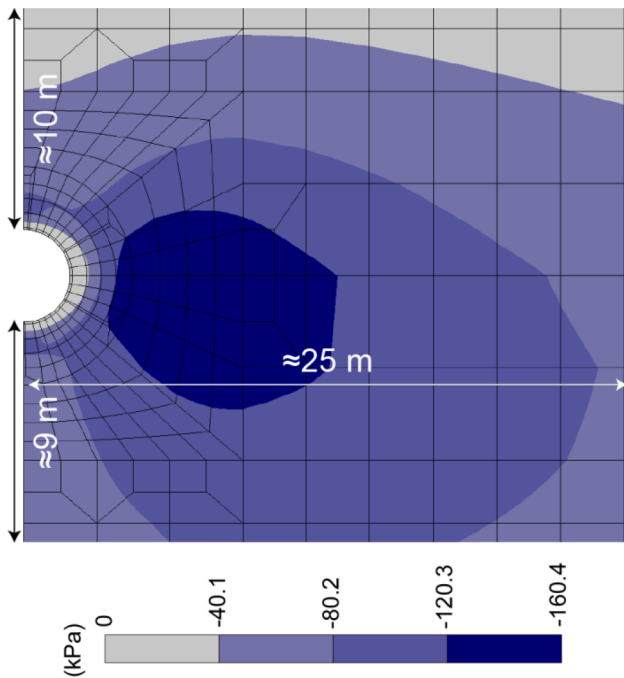


Fig. 13. Contour plot of the change in pore pressure (kPa) around the tunnel from the end of tunnel construction to present day (compression is positive).

the bending moments broadly match those obtained with the continuous lining. This feature of joint opening influencing the bending moment distribution only locally was also observed in the experimental tests conducted by Afshan et al. (2017) on a half-scale GCI ring. The tunnel squatting magnitudes range between 0.52 and 0.55% and so fall towards the lower end of those usually observed in the field (0.5–1%). The analysis adopting the NBJ model results in the largest tunnel distortions followed by the BJ model and the continuous lining, consistent with the bending moment distributions, i.e. larger bending moments are associated with smaller displacements. Nevertheless, the differences in displacements are more modest than those observed in the bending moments, particularly in terms of tunnel squatting: the analysis adopting the NBJ model produces a tunnel squat only about 6% larger than that of the continuous lining (% with respect to the latter magnitude). This agrees with the observations of Ruiz López et al. (2022) who showed that joint opening affects substantially more the bending moments locally around the joint than the tunnel squat. More significant differences are observed in the vertical distortion as the analysis adopting the NBJ model yields a vertical diametric distortion almost 30% larger than that given by the analysis adopting a continuous lining (% with respect to the latter magnitude). As shown in Fig. 14c, negligible differences are found in the axial force distribution around the lining for the three analyses. This is reasonable since the axial force did not reach the maximum tensile or compressive magnitude considered by the joint models and so the three models mobilised the same axial stiffness all around the lining.

The evolution of the diametric distortion with time is depicted in Fig. 15a. The final tunnel distortions are largely reached around 10 years after construction although full steady-state conditions are not attained until several decades later. The vertical distortion from the analysis adopting the jointed models starts deviating from the continuous lining

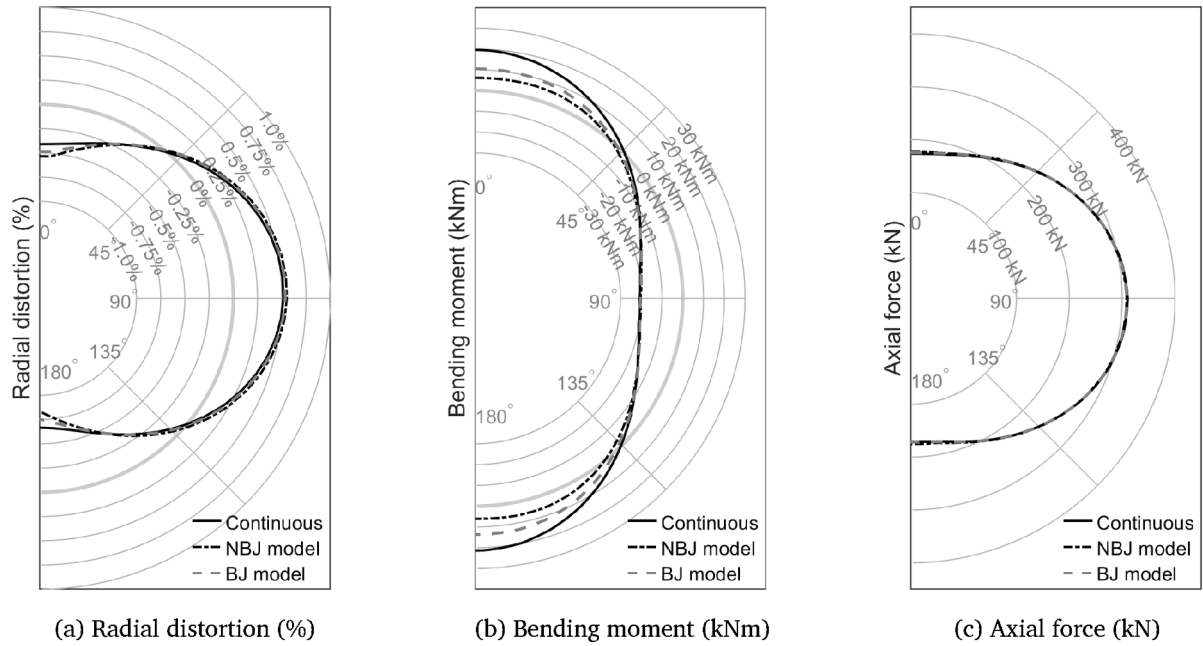


Fig. 14. Distribution of tunnel deformation and internal forces of the tunnel at present day with different segmental lining modelling.

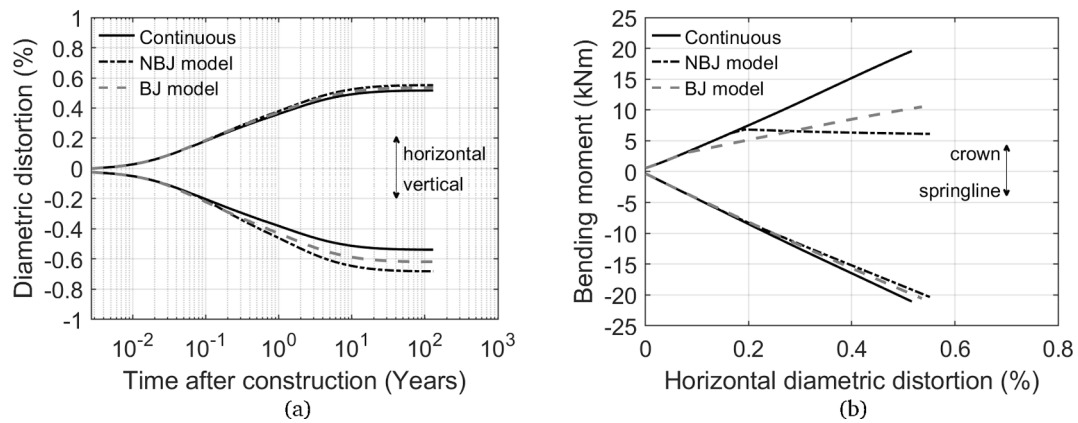


Fig. 15. (a) Diametric distortion (% diameter) with time and (b) Bending moment (kNm) at crown and springline with horizontal diametric distortion (% diameter) with different segmental lining modelling.

result at around 36 days (0.1 years) after construction when the vertical and horizontal distortion are about 0.2%. The distinct response of the two joint models is clearly observed in Fig. 15b where the bending

moment with tunnel squat is depicted. Positive bending moments were observed at the crown and negative at the springline. When the BJ model was employed, and as a result of joint opening, the crown exhibited a

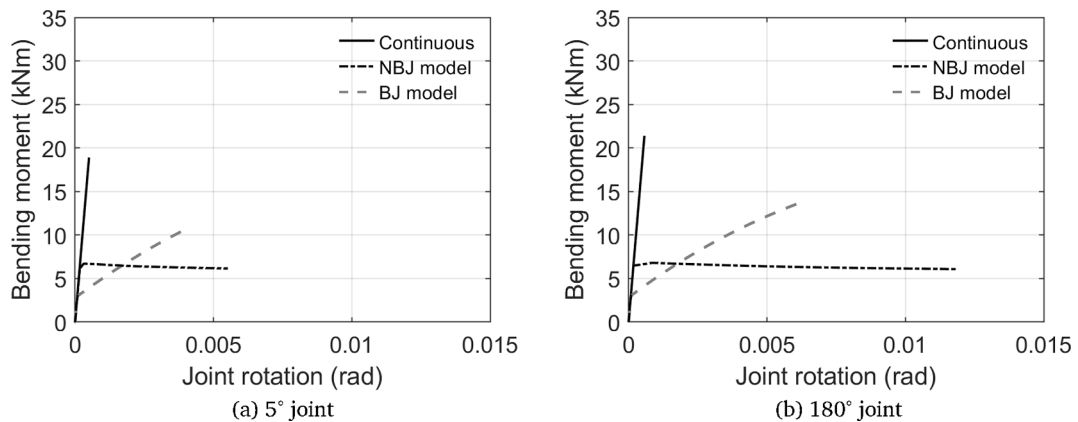


Fig. 16. Bending moment-rotation response of the 5° and 180° joints with different joint modelling.

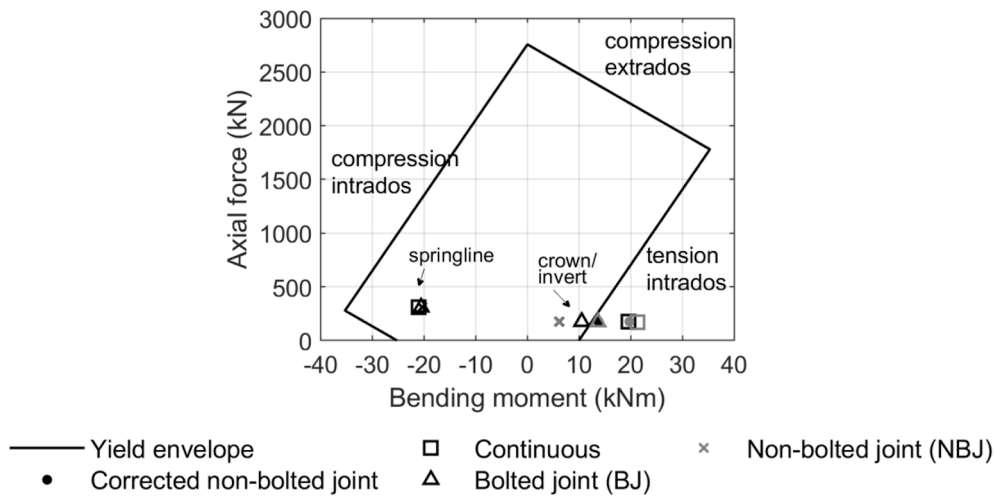


Fig. 17. Bending moment-axial force (M - N) interaction diagram corresponding to the present-day condition (invert data points in grey).

smaller stiffness with respect to the continuous lining from a bending moment of approximately 3 kNm and a tunnel squat of 0.08%. By the end of the analysis, at a tunnel squat of 0.54%, the slope of the line corresponding to the BJ model suggests that the bending stiffness at the crown is far from exhausted. Conversely, the result from the analysis adopting the NBj model indicates that the crown section does not attract any additional bending moments after the joint fails at around 7 kNm. The curves corresponding to the springline indicate only a slightly softer response from the jointed models in comparison to the continuous lining. As the springline is away from the opening joints (5° and 180°), its stiffness is hardly affected and the small differences from the continuous lining are due to the overall reduction of the global stiffness of the tunnel due to joint opening.

Fig. 16 depicts the bending moment-rotation response of the 5° and 180° joint locations for the continuous lining and the two jointed models. The rotation was determined as the relative rotation between the ends of the beam elements representing the joints; that of the 180° joint was doubled to account for symmetry. Since the 180° location (Fig. 16b) tends to attract greater bending moments than the 5° cross-section (Fig. 16a), the former develops larger rotations, particularly with the jointed models, and so the differences between the three models are more apparent there. In line with the previous results, the analysis adopting the NBj model exhibits a much larger joint rotation for the present-day condition than that obtained with the BJ model while the continuous lining shows a much stiffer response than the two jointed models. It is useful to consider the opening magnitudes associated with the rotations shown in Fig. 16. The opening at the intrados can be estimated assuming rotation of the joint about the point at the extrados such that $\text{opening} = 2 \cdot h \cdot \tan(\theta/2)$ where h is the total height of the joint section (0.1238 m). Based on this, the openings taking place at the 180° joint at present day are approximately 0.070 mm, 1.462 mm and 0.760 mm for the continuous lining, NBj model and BJ model, respectively.

3.3. Evaluation of internal forces

The lining stress state produced with the different lining models is assessed here with respect to a bending moment-axial force (M - N) yield envelope presented in Fig. 17. The M - N yield envelope was derived adopting the tensile and compressive yield strengths of GCI as 38 MPa and 150 MPa (TFL, 2017), respectively, and assuming that material behaviour was linear elastic prior to yield. The stress states from the crown, springline and invert (in grey) are included in Fig. 17. These cross-sections are subjected to the largest bending moments and so are potentially most sensitive to yielding.

The dataset referred to as “Corrected Non-bolted joint” follows a

similar methodology as that applied in GCG (2002), where the NBj model was used and the bolts’ contribution was obtained from the M - θ relationship obtained by Thomas (1977) from a laboratory test on two GCI segments bolted together and subjected to positive bending under zero axial force. Instead of using the Thomas (1977) relationship here, the bolts’ contribution was determined from the M - θ relationship shown in Fig. 18, which was obtained from a 3D analysis on the joint geometry of Fig. 2 adopting the same set-up as Ruiz López et al. (2023a). Similar to Thomas (1977), the joint response was considered under positive bending and zero axial force. The bolts’ contributions at crown and invert were obtained as the bending moments from Fig. 18 corresponding to the rotations (after yielding) of the 5° and 180° joints, respectively, from the analysis using the NBj model (Fig. 16). Those bending moments were directly added to the bending moments of the two cross-sections given by the analysis. Note that the bolts’ contribution is considerably larger at the invert (13.7 kNm) than at the crown (7.6 kNm) in correspondence with the rotation magnitudes predicted in the analysis at the 5° and 180° joints, see Fig. 16.

Regarding the results shown in Fig. 17, the stress state from the springline section is well within the yield envelope for the three analyses. Conversely, the results indicate that the crown and invert sections may be prone to yielding in tension at the intrados. Moreover, the outcome of the assessment (whether the stress state lies within the envelope or not) depends on the lining model adopted in the analysis. When the continuous lining is employed, the crown and invert data points are clearly outside the yield envelope. As discussed previously,

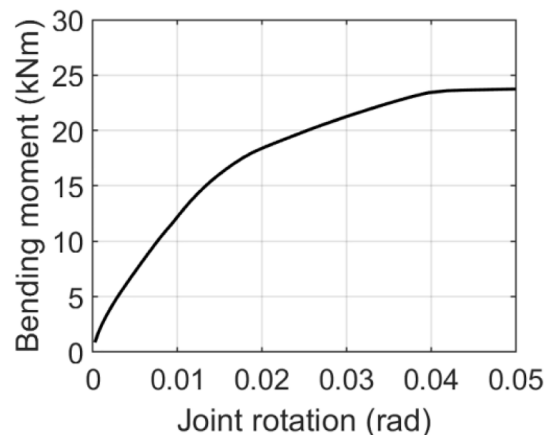


Fig. 18. Bending moment (kNm) contribution from the bolts for a joint rotation under positive bending as obtained in 3D analysis.

the use of the BJ model results in significantly smaller bending moments at crown and invert than those from the continuous lining: the stress state from the crown section remains inside the envelope while that from the invert is just on the outside when the BJ model is adopted. The stresses from the crown and invert given by the analysis employing the NBJ model (crosses) lie on top of each other and fall well within the yield envelope. On the contrary, when the contribution from the bolts is added to the bending moments obtained in the analysis of the NBJ model, the data points from the crown and invert lie outside the envelope. The latter are greater than the corresponding data points from the BJ model mostly because the bolts' contribution is determined from joint rotations that are significantly larger than those obtained from the analysis using the BJ model, see Fig. 16. Furthermore, when the contribution from the bolts is compounded a posteriori to the results of the NBJ model, the bending moment at the invert section is similar in magnitude to that of the continuous lining which illustrates that the application of this approach can end up neglecting the influence of the joints on the bending moment magnitudes.

The results discussed above demonstrate that the modelling approach adopted to simulate the GCI tunnel lining can be critical for the outcome of structural assessments in practice. While adopting a continuous lining may give the upper limit of the bending moment, the effect of the joints is usually included when the resulting structural assessment fails (TFL, 2017). Nonetheless, none of the previously available methodologies for doing so seem ideal: the bending moment magnitudes given directly by the NBJ model can be regarded as non-conservative, in terms of internal forces, whereas those considering the bolts' contribution a posteriori can be overly conservative. It is therefore proposed here that the BJ model is employed to account adequately for the influence of the tunnel joints on the bending moment magnitudes.

4. Effect of different tunnel drainage conditions

4.1. Pore pressure response

Fig. 19 depicts the vertical and horizontal pore pressure profiles at present day for the analyses considered. The displayed profiles start at the extrados of the tunnel lining which corresponds to the tunnel boundary when a fully permeable condition is adopted and to the edge opposite to the tunnel of the first soil element when finite lining permeability is adopted. The magnitude of the pore pressure at the tunnel extrados at the crown and springline largely controls the overall magnitude of the vertical and horizontal profiles, respectively. The analyses adopting a fully permeable lining (Fully perm.) and a lining of

finite permeability (Finite perm.) gave the lowest and highest pore pressures, respectively. The impact of having some or all the joints as fully permeable as compared to these two extreme cases is very significant. By having just the 120° joint as fully permeable (120° joint perm.) there is a very clear drop in pore pressure with respect to the finite permeability case both above and on the side of the tunnel; when both the 59° and the 120° joints are fully permeable (59°-120° joint perm.) a further significant reduction in pore pressure can be observed. Some further differences are found above the tunnel between the latter case and the case where all the joints were fully permeable (All joints perm.) while they are almost negligible on the side of the tunnel. The overall small differences observed between these two analyses can be attributed to the drainage through the 59° and the 120° joints being dominant over the crown and invert joints, in the analysis with all joints permeable, as a result of the permeability anisotropy ratio k_h/k_v being larger than 1. It can be concluded that, according to the analyses, the impact of the tunnel drainage condition on the pore pressure distribution is very significant.

4.2. Structural response

Regarding the tunnel structural response, Fig. 20 shows the tunnel deformations and internal forces at present day for different tunnel drainage conditions. The analysis with a fully permeable lining yields the largest tunnel distortion with a tunnel squat of about 0.54% (Fig. 20a). It is followed by the analyses with all the joints and with the 59° and the 120° joints modelled as fully permeable, which yield virtually identical distortions around the tunnel circumference, and a tunnel squat of about 0.4%. A smaller tunnel squat (0.31%) is obtained in the analysis where only the 120° joint is fully permeable and lastly, the analysis adopting a lining with a low, finite permeability gives the smallest tunnel squat (0.19%). The impact of drainage through the joints on the tunnel deformation is quite remarkable: for instance, drainage only through the 120° joint produces a squatting 63% larger than that of the lining with finite permeability (% with respect to the latter quantity). It is also worth noting the apparent relationship between the horizontal profile of pore pressures (Fig. 19b) and the tunnel squat, i.e. smaller pore pressure relating to larger squatting.

The magnitudes of the bending moment distributions arising from different drainage conditions (Fig. 21b) reduce from one analysis to the next in the same order as described for the tunnel distortions. While differences in the bending moment magnitudes between analyses are evident at the springline, they are less clear around the crown and invert due to the stiffness degradation arising from the opening of the 5° and 180° joints.

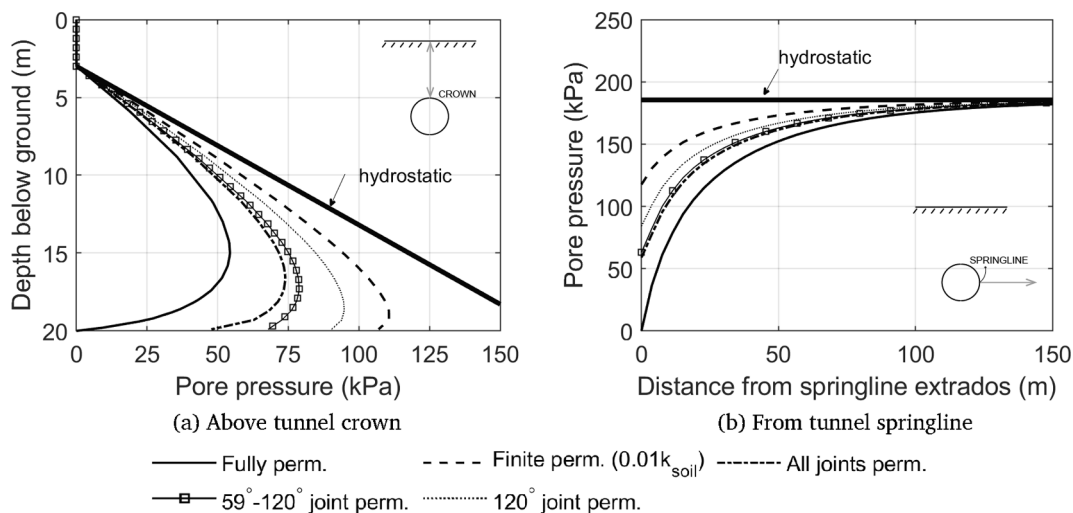


Fig. 19. Pore pressure profiles at present day for different tunnel drainage conditions.

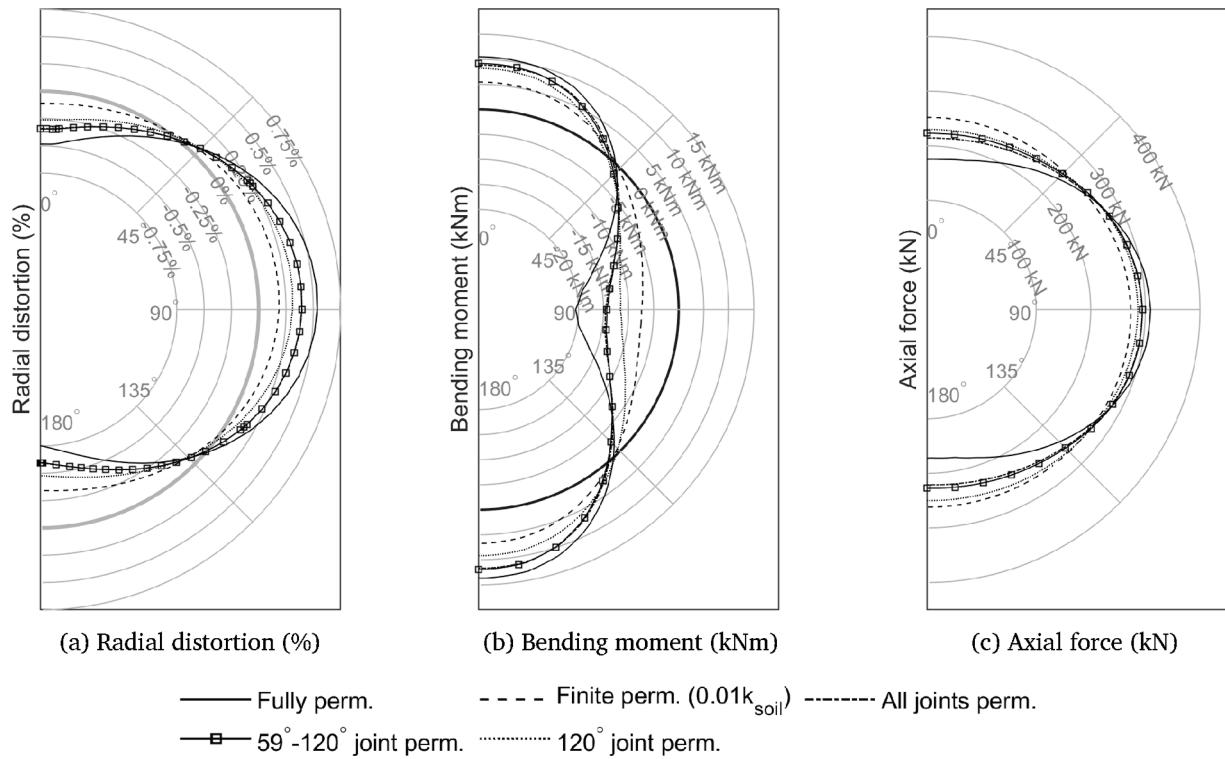


Fig. 20. Tunnel deformation and internal forces around the tunnel at present day for different tunnel drainage conditions.

The analyses adopting all the joints and the 59° and the 120° joints as fully permeable produce very similar axial forces all around the lining (Fig. 21c), which along with previous observations on the tunnel distortion and bending moments demonstrate that the tunnel response is essentially equivalent for the two tunnel drainage conditions. The analysis with the 120° joint as fully permeable yields a similar axial force distribution to the latter two analyses apart from a slight deviation around the invert. At the crown/invert, the finite permeability lining case yields the largest axial forces followed by the cases where the 120° joint is fully permeable, the 59° and the 120° joints are fully permeable, all the joints are fully permeable and lastly, the case where the lining is fully permeable. This order is reversed at the springline. Differences in magnitudes are significantly more prominent at the crown and invert than at the springline. The predicted trends suggest that the magnitude

of the axial force at the springline is primarily dominated by the bending experienced by the tunnel lining (i.e. the greater the squatting the larger the forces). Conversely, while the axial force magnitude at the crown and invert is also influenced by bending, it is mostly governed by the drainage condition (i.e. the less permeable the lining the larger the forces).

The evolution of the axial force with time at the crown and springline locations is presented in Fig. 21 in terms of absolute values and as a percentage of the axial force corresponding to the full overburden stress σ_v at the depth of the springline (i.e. $N = \sigma_v \cdot R \cdot b$, R =tunnel radius, b =ring width). Immediately after construction, the axial force at the crown has a value of about 220 kN, which constitutes 53% of the overburden value for all tunnel drainage conditions except for the fully permeable lining where it is slightly smaller (it reduces shortly after

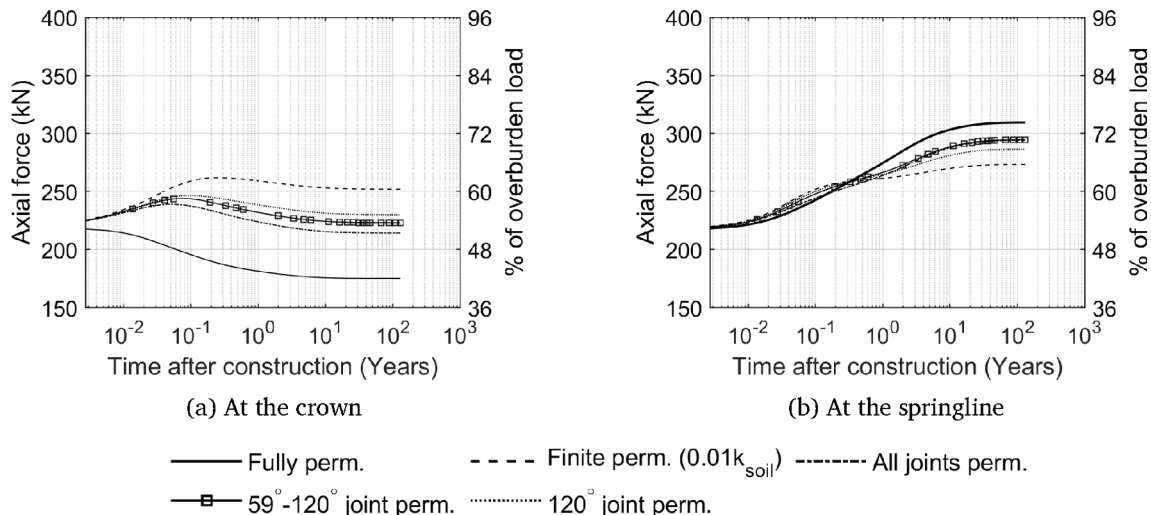


Fig. 21. Evolution of the axial force with time for different tunnel drainage conditions.

construction). In the latter case, the axial force continues its decay until about 10 years after construction to a final value of around 42% of the overburden value. In the remaining cases, the axial force at the crown increases over the first 2.5 months after construction and then reduces gently over time. Out of these, only the finite permeability lining case shows an apparent increase from the force magnitude at the end of construction to that at present day (about 60% of the overburden value), whereas the other cases exhibit forces at present day of similar magnitude as immediately after construction. The results indicate that the axial force at the springline (Fig. 21b) increases significantly during soil consolidation in all the cases, with the magnitude at present day ranging from 65% to 74% of the overburden value. The overall increase in the axial forces around the tunnel agrees qualitatively with field measurements taken on GCI tunnel linings (Ward & Thomas, 1965; Thomas, 1976). Moreover, the numerical model agrees with the observations of Ward & Chaplin (1957) who reported larger axial forces at the springline than at the crown.

The analysis results demonstrate that considering non-homogeneous tunnel drainage conditions has a major impact on the GCI tunnel structural response. In particular, the analyses reveal that local drainage through some or all of the joints significantly increases the tunnel deformations and affects the distribution of internal forces. Since tunnel joints constitute potential leakage paths in practice, the results presented here suggest that analyses of the long-term behaviour of GCI tunnels (and possibly other segmental tunnels as well) in clay should consider the potential effects of drainage through the joints in addition to the usual drainage conditions.

5. Conclusions

This paper investigates numerically the influence of the approach adopted for simulating the mechanical behaviour of tunnel joints and of considering drainage through the joints on the structural response of segmental GCI linings from their construction to present day. In relation to the former, three different approaches were considered for the simulation of the GCI segmental lining: a continuous lining (i.e. no joints) and adopting the NBJ model and the BJ model to simulate the behaviour of the joints.

The numerical results indicate that considering explicitly the joints has a very significant influence on the tunnel structural response, particularly on the bending moment distribution around the tunnel. When the joints were included, the bending moment magnitudes around the crown and invert were considerably lower than those obtained with the continuous lining.

Moreover, the bending moments obtained with the BJ model were evidently larger than those produced by the NBJ model, demonstrating that considering the influence of the bolts along with the stiffness decay after joint opening is crucial to the analysis of the present-day condition

of GCI tunnel linings. The simulation of the joints noticeably affected the vertical distortion of the tunnel, whereas the effect on the horizontal distortion was not as pronounced. The importance of the simulation approach adopted for the segmental tunnel lining was shown to be critical in the assessment of the tunnel internal forces against the M - N yield envelope specified by TfL (2017).

It can be argued that the approach adopting the BJ model provided the most reasonable assessment of material yielding since the remaining approaches do not consider adequately the stiffness of the joints and so appear as either excessively conservative or non-conservative. This has important practical implications because the calculated present-day condition of the tunnels largely determines their available capacity for additional solicitations and it can therefore influence the decision-making on any new construction activities taking place in the proximity of the tunnel.

The investigation into different tunnel drainage conditions demonstrated that significant soil consolidation around the tunnel can be induced by drainage into all or some of the joints. Such consolidation has an important effect on both tunnel deformations and internal forces. Overall, the results presented in this paper suggest that both the mechanical response and the drainage of tunnel joints should be accounted for in the evaluation of the present-day condition of segmental GCI tunnels.

CRedit authorship contribution statement

Agustín Ruiz López: Methodology, Investigation, Formal analysis, Writing – original draft. **Aikaterini Tsiampousi:** Supervision, Writing – review & editing, Funding acquisition. **Jamie R. Standing:** Supervision, Writing – review & editing. **David M. Potts:** Software, Writing – review & editing.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data availability

Data will be made available on request.

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Appendix A. Formulation of the tunnel joint models

A1. Non-bolted joint (NBJ) model (Potts & Zdravkovic, 2001)

The NBJ model, available in ICFEP and introduced conceptually in Potts & Zdravkovic (2001), is an elastoplastic model meant to be adopted by a small beam element representing the joint. The bending moment acting at the joint is limited by the ultimate bending moment M_u which is determined assuming that a portion of the section fails under compression. Furthermore, the presence of joint bolts is not considered and so the model assumes that the joint cross-section does not develop any tension. Prior to the bending moment reaching the ultimate bending moment M_u , the joint response is governed by the bending stiffness. The model inputs are the ultimate compressive strength f_c and the dimensions of the joint cross-section: b_1 , h_1 , b_2 , h_2 , b_3 and h_3 which, as shown in Fig. A1, allow for a general shape cross-section to be defined. The program determines, from the input dimensions, the distance from the centroid to the extrados and to the intrados, y_o and y_i , respectively. The expressions adopted to compute the ultimate bending moment under positive and negative bending are presented below. Note that different expressions apply depending on the compressive failure area in relation to that of the three geometry blocks shown in Fig. A1.

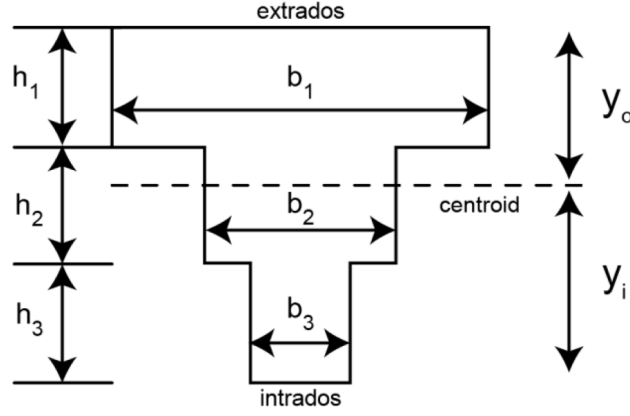


Fig. A1. Geometry of the joint cross-section as input to the NBJ model.

Note that in the expressions below compression is assumed to be negative and that $F_c = -N$.

Under positive bending, the following expressions apply:

$$M_u^+ = \begin{cases} F_c \left(y_o - \frac{F_c}{2b_1f_c} \right) & F_c \leq b_1h_1f_c \\ b_1h_1f_c \left(y_o - \frac{h_1}{2} \right) + (F_c - b_1h_1f_c) \cdot \left(y_o - \frac{h_1}{2} - \frac{F_c - (b_1 - b_2)h_1f_c}{2b_2f_c} \right) & b_1h_1f_c < F_c \leq b_1h_1f_c + b_2h_2f_c \\ b_1h_1f_c \left(y_o - \frac{h_1}{2} \right) + b_2h_2f_c \left(y_o - h_1 - \frac{h_2}{2} \right) + (F_c - b_1h_1f_c - b_2h_2f_c) \cdot \left(y_o - \frac{h_1}{2} - \frac{h_2}{2} - \frac{F_c - (b_1h_1 + b_2h_2)f_c + b_3(h_1 + h_2)f_c}{2b_3f_c} \right) & F_c > b_1h_1f_c + b_2h_2f_c \end{cases} \quad (A1.1)$$

Similarly, under negative bending:

$$M_u^- = \begin{cases} F_c \left(y_i - \frac{F_c}{2b_3f_c} \right) & F_c \leq b_3h_3f_c \\ b_3h_3f_c \left(y_i - \frac{h_3}{2} \right) + (F_c - b_3h_3f_c) \cdot \left(y_i - \frac{h_3}{2} - \frac{F_c - (b_3 - b_2)h_3f_c}{2b_2f_c} \right) & b_3h_3f_c < F_c \leq b_3h_3f_c + b_2h_2f_c \\ b_3h_3f_c \left(y_i - \frac{h_3}{2} \right) + b_2h_2f_c \left(y_i - h_3 - \frac{h_2}{2} \right) + (F_c - b_3h_3f_c - b_2h_2f_c) \cdot \left(y_i - \frac{h_3}{2} - \frac{h_2}{2} - \frac{F_c - (b_3h_3 + b_2h_2)f_c + b_1(h_3 + h_2)f_c}{2b_1f_c} \right) & F_c > b_3h_3f_c + b_2h_2f_c \end{cases} \quad (A1.2)$$

Additionally, the model prevents the joint from developing tensile axial forces and sets the most compressive axial force to be $N = -f_c A$. The ultimate shear force S_u is defined in the model as $S_u = -N \tan(\delta)$, where δ is the angle of friction between the tunnel segments.

A2. Bolted joint (BJ) model (Ruiz López, 2022)

The formulation of the BJ model, introduced by Ruiz López (2022), is summarised here. The BJ model, as implemented in ICFEP, is meant for plane-strain beam elements. As the NBJ model described above, the BJ model is devised to be assigned to a small beam element representing the joint so that the joint rotation θ is realised as a relative rotation between the ends of that small beam element. The equations introduced below are written assuming that compression is negative; superscripts + and - indicate positive (tension at intrados) and negative bending (tension at extrados), respectively.

To begin with, the model assumes that the joint behaviour is linear prior to opening. The bending moment of opening M_o corresponds to that at which the edge of the joint goes into tension, the assumed stress distribution at this point is shown in Fig. A2. Under positive bending, it is calculated as:

$$M_o^+ = -\frac{N \cdot I}{y_i \cdot A} \quad (A2.1)$$

where N is the axial force, I is the second moment of area of the segment cross-section and A is the segment cross-section area. I and A are determined from the input dimensions of the joint geometry. Under negative bending, it is computed as:

$$M_o^- = -\frac{N \cdot I}{y_o \cdot A} \quad (A2.2)$$

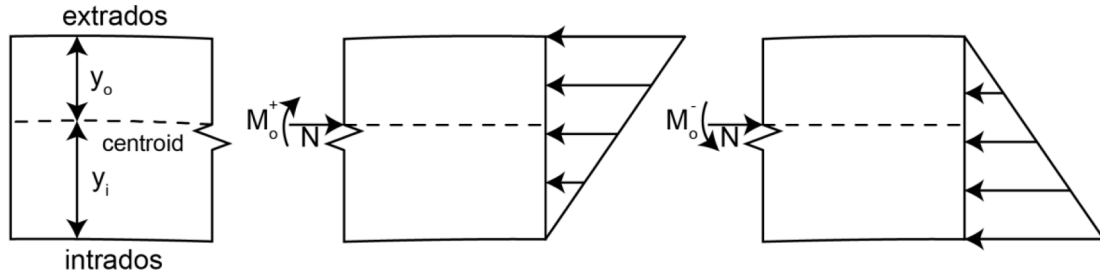


Fig. A2. Stress state at the onset of opening assumed by the joint model.

Before opening, the M - θ response is governed by the rotational stiffness K_o so that the rotation at the onset of opening θ_o can be determined as:

$$\theta_o^+ = \frac{M_o^+}{K_o^+}, \theta_o^- = \frac{M_o^-}{K_o^-} \quad (\text{A2.3})$$

The M - θ behaviour after opening is nonlinear and defined by a hyperbolic expression so that the bending moment after opening can be expressed as:

$$M = M_o + \frac{K_i \cdot (|\theta| - \theta_o)}{\left[1 + \left(\frac{K_i \cdot (|\theta| - \theta_o)}{M_u - M_o}\right)^n\right]^{\frac{1}{n}}} \quad (\text{A2.4})$$

where θ is the joint rotation, K_i is the initial rotational stiffness after opening and n is the shape parameter of the hyperbolic function. M_u is the ultimate bending moment which has a tension component $M_{u,b}$ from the action of the bolts and a compression component $M_{u,c}$ from the contact force. Correspondingly, under positive bending:

$$M_u^+ = M_{u,b} + M_{u,c}^+ \quad (\text{A2.5})$$

The tension component $M_{u,b}$ enables two rows of bolts at different heights to be considered assuming the same diameter and strength for all the bolts. Moreover, the model assumes that all the bolts reach their ultimate tensile strength and so $M_{u,b}$ is calculated as follows:

$$M_{u,b} = [n_{b1} \cdot (y_i - h_{G1}) + n_{b2} \cdot (y_i - h_{G2})] \cdot F_{tu,b} \quad (\text{A2.6})$$

where n_{b1} and n_{b2} are the number of bolts for the first and second rows of bolts, respectively, h_{G1} and h_{G2} is the distance from the intrados to the bolt centrelines of the first and second rows, respectively, and $F_{tu,b}$ is the ultimate tensile force of the bolt:

$$F_{tu,b} = \frac{\pi \cdot \phi_b^2}{4} \cdot f_{tu,b} \quad (\text{A2.7})$$

where ϕ_b is the bolt thread diameter and $f_{tu,b}$ is the uniaxial tensile strength of the bolt material. The compression component $M_{u,c}$ can be calculated either assuming that failure of the area under compression occurs or neglecting that possibility which implies that the contact force reaches the maximum joint eccentricity. The analyses presented here adopted the latter option based on the behaviour observed in 3D analyses of the GCI tunnel joint. The compression component under positive bending is then calculated as:

$$M_{u,c}^+ = F_c \cdot y_o = [-N + (n_{b1} + n_{b2}) \cdot F_{tu,b}] \cdot y_o \quad (\text{A2.8})$$

where F_c is the contact force between segments, expanded on the right-hand side. Fig. A3a depicts the stress state assumed by the model at the ultimate bending moment M_u^+ . Similarly to the above, the ultimate bending moment under negative bending M_u^- is determined as follows:

$$M_u^- = -(M_{u,b} + M_{u,c}^-) \quad (\text{A2.9})$$

and the compression component under negative bending $M_{u,c}^-$ is given by the following expression:

$$M_{u,c}^- = -F_c \cdot (y_i - h_U) \quad (\text{A2.10})$$

where h_U is the depth of the caulking groove which, as shown in Fig. A3b, is assumed not to be effective in transmitting compressive stresses.

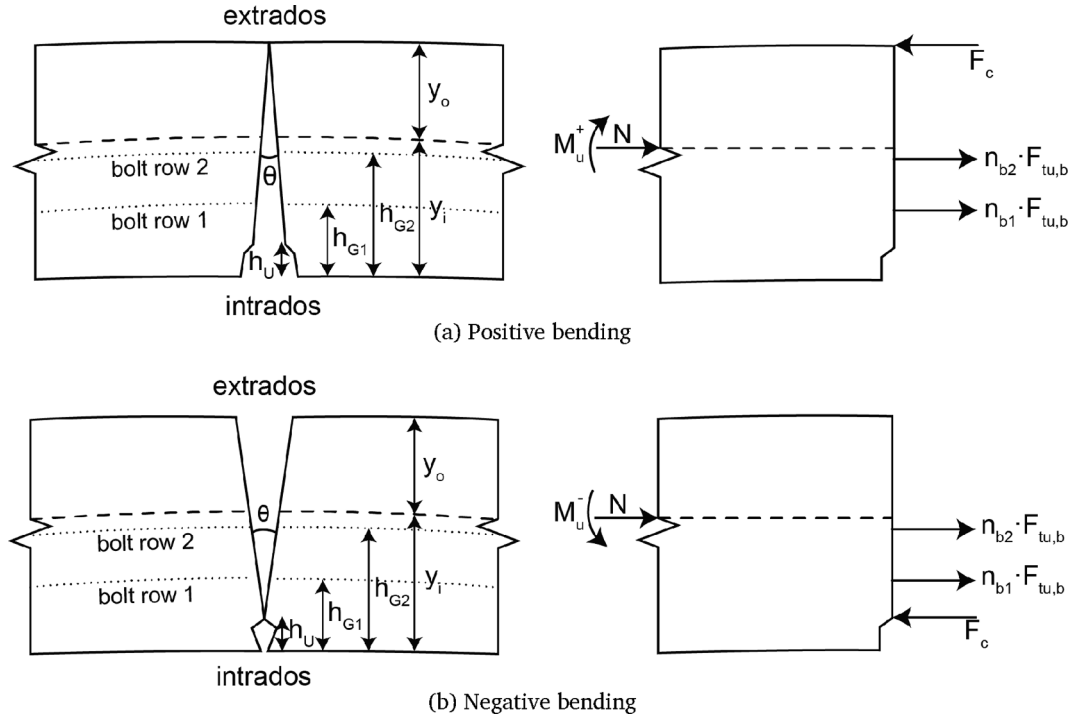


Fig. A3. Stress state corresponding to the ultimate bending moment M_u .

After opening, the model incorporates the axial force dependency of the joint response through the presence of the bending moment of opening M_o and the ultimate bending moment M_u in the denominator of the hyperbolic expression in Expression (A 4). Fig. A4 depicts schematically the M - θ response realised by the model for different axial forces. Although not considered in the analyses conducted for this paper, the model also includes the option of defining a bending moment cut-off M_f through the definition of a rotation θ_f beyond which the bending moment does not further increase for any given axial force. For convenience, Expression (A 4) is written in terms of rotational stiffnesses, however, appropriate conversions to bending stiffnesses take place in the program for the model to be compatible with beam elements (Ruiz López, 2022).

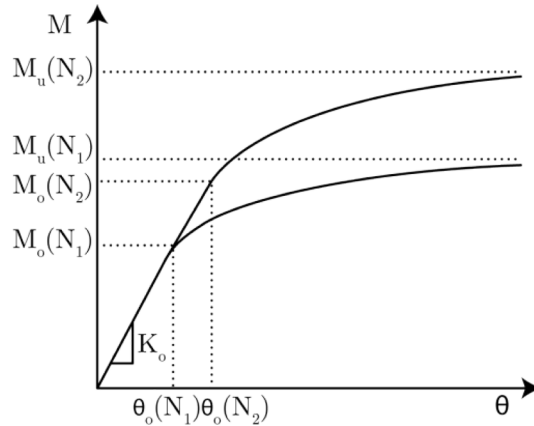


Fig. A4. M - θ response defined by the model for varying axial forces.

Regarding the behaviour of the axial force, the model imposes that the ultimate tensile axial force is equal to:

$$N_{tu} = (n_{b1} + n_{b2}) \cdot F_{tu,b} \quad (A2.11)$$

Lastly, the ultimate shear force S_u is determined with the following expressions:

$$\begin{aligned} S_u &= c^* - N \cdot \tan(\delta) \quad \text{if } N \leq 0 \\ S_u &= c^* \quad \text{if } N > 0 \end{aligned} \quad (A2.12)$$

where the cohesion term c^* is defined as follows:

$$c^* = c \frac{\pi \phi_b^2}{4} (n_{b1} + n_{b2}) \quad (A2.13)$$

where c corresponds to the maximum shear stress the bolts can mobilise. Expression (A2.12) applies to both positive and negative shear forces.

The model parameters relating to the out-of-plane direction are input in terms of the width of a tunnel ring, denoted subsequently with a hat. These correspond to the width dimensions of the section ($\bar{b}_1$, $\bar{b}_2$ and $\bar{b}_3$), number of bolts ($\bar{n}_{b1}$ and $\bar{n}_{b2}$) and the rotational stiffness ($\bar{K}_o$, $\bar{K}_i$). The conversion factor b_{max} is the maximum width of the segment ring $b_{max} = \max\{\bar{b}_1, \bar{b}_2, \bar{b}_3\}$, such that the conversion to plane-strain dimensions is computed as follows:

$$K_o = \bar{K}_o / b_{max}, K_i = \bar{K}_i / b_{max} \quad (\text{A2.14})$$

$$n_{b1} = \bar{n}_{b1} / b_{max}, n_{b2} = \bar{n}_{b2} / b_{max} \quad (\text{A2.15})$$

$$b_1 = \bar{b}_1 / b_{max}, b_2 = \bar{b}_2 / b_{max}, b_3 = \bar{b}_3 / b_{max} \quad (\text{A2.16})$$

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