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Hydraulic permeability and ageing behaviour of Dublin Port Tunnel

Perméabilité hydraulique et comportement de vieillissement du tunnel portuaire de Dublin

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ABSTRACT: Tunnel hydraulic deterioration has been widely reported and investigated in many past studies where, nevertheless, the tunnel lining permeability was assumed as constant and the time-dependent hydraulic degradation process was neglected. To investigate the hydraulic ageing behaviour of Dublin Port Tunnel, in this paper, a modified ground-lining relative permeability analytical model was derived and current deteriorated lining permeability was estimated using field monitoring water flow data. Compared with the initial watertight status of Dublin Port Tunnel, the current hydraulic state of the tunnel was found to be partially permeable after more than a decade's operation. The results from numerical simulations showed that the assumption of a constant hydraulic permeability during the lifetime of tunnel structures may not evaluate the ageing tunnel deformational performance process realistically. It is important to consider the effect of time-dependent hydraulic deterioration process on tunnel performance.

RÉSUMÉ: La détérioration hydraulique du tunnel a été largement rapportée et étudiée dans de nombreuses études antérieures où, néanmoins, la perméabilité du revêtement du tunnel a été supposée constante et le processus de dégradation hydraulique dépendant du temps a été négligé. Pour étudier le comportement de vieillissement hydraulique du tunnel du port de Dublin, dans cet article, un modèle analytique de perméabilité relative de revêtement de sol modifié a été dérivé et la perméabilité de revêtement de tunnel détériorée actuelle a été estimée à l'aide de données de surveillance sur le terrain. Par rapport à l'état d'étanchéité initial du tunnel du port de Dublin, l'état hydraulique actuel du tunnel s'est révélé partiellement perméable après plus d'une décennie d'exploitation. Les résultats des simulations numériques ont montré que l'hypothèse d'une perméabilité hydraulique constante pendant la durée de vie des structures de tunnel peut ne pas évaluer de manière réaliste le processus de performance de déformation du tunnel vieillissant. Il est important de prendre en compte l'effet du processus de détérioration hydraulique dépendant du temps sur les performances du tunnel.

KEYWORDS: Hydraulic deterioration; relative permeability; time-dependent performance; hydro-mechanical coupled modelling.

1 INTRODUCTION

Tunnel structures, like any other structure, are subject to a varying degree of deformation and deterioration of either structural or hydraulic properties during their lifetime interaction with the surrounding underground environment. Deterioration can take the form of water leakage, crack propagation, lining deformation, etc., and, if not subjected to appropriate control and management, may cause interruption to tunnel operation, pose a threat to tunnel serviceability, or even endanger tunnel safety.

Hydraulic deterioration of tunnel linings, the most commonly observed tunnel deterioration, normally exists in two forms: 1) the blockage of water drainage system which results in a decrease in drainage capacity (Kim et al. 2020); 2) the increase of water leakage into tunnels induced by factors such as lining crack propagation (Li et al. 2020). The former type of deterioration, which could stem from fine particle migration and accumulation into drainage routes, features the build-up of pore water pressure behind tunnel linings inducing detrimental effects on tunnel linings (Shin et al. 2005). However, the latter one, associated with concrete crack or construction joint propagation which expands water flow route, characterises excessive water infiltration into the tunnel (Shin et al. 2005). Both deteriorations lead to a change in tunnel lining hydraulic status and permeability and a further change in tunnel mechanical and deformational performance, as noted by many previous investigations (Shin et al. 2012; Yoo 2016; Li et al. 2020; Kim et al. 2020).

For drainage systems, Yoo (2016) concluded on the basis of numerically simulating the hydraulic deterioration of geo-synthetic filter that the decrease of drainage system capacity led to a reduction in lining permeability and this type of hydraulic deterioration induced additional loads on tunnel linings, resulting in the increase in tension stress at the tunnel crown and invert and compression stress at the spring-line. Likewise, another study by Kim et al. (2020) also examined the same effect of hydraulic deterioration of geotextile filter in tunnel drainage system on lining structural performance but differentiated from Yoo (2016) in incorporating both the deterioration of mechanical and hydraulic behaviour of the geotextile filter instead of considering its hydraulic deteriorating behaviour only.

For concrete cracking, Picandet et al. (2009) revealed by laboratory experiments that the global permeability of deteriorated concrete increased with the propagation of existing cracks (i.e. crack width) and initiation of further cracks (i.e. crack density) but also admitted that the crack width and density are highly uncertain and time-dependent. Yi et al. (2011) noted the time-dependent growth of crack width and density in concrete enabled the interconnection of water flow routes and contributed to permeability increase, further instigating water or aggressive chemical ions penetrating the concrete and accelerating deterioration.

Therefore, it can be seen that investigations on both forms of deterioration have been widely covered in previous research. However, research on the determination of the post-operation (deteriorated) lining concrete hydraulic permeability has been scarce due to the difficulty in obtaining extensive field measurements. To determine the current deteriorated hydraulic permeability of tunnel linings, Laver et al. (2013) conducted permeability tests on several intact but degraded grout samples extracted from sites around London Underground tunnels and found that the intact freshly-hardened grout can behave as an impermeable barrier while with progressive deterioration it could be transformed to a flow path instead. However, such intrusive grout/lining sampling is rarely acceptable to tunnel asset owners in practice. Bagnoli et al. (2015) established a computational model, where the total water flow into the tunnel was regarded as a function of concrete hydraulic conductivity, by best-fitting the water flow-conductivity data obtained from varying the hydraulic conductivity values in numerical simulations. The conductivity value, for which the total numerical water inflow corresponds to the field-measured water flow in a real tunnel, can subsequently be considered as the estimated value of current hydraulic conductivity. This inverse analysis-based estimate can then be compared with the initial value to assess and evaluate the hydraulic deterioration state of the tunnel lining. Li et al. (2020) managed to derive the lining permeability using the permeability model of jointed rocks with modifications and tested the model by implementing numerical simulations. Instead of calculating tunnel hydraulic permeability directly, Wongsaroj et al. (2013) proposed relative ground-lining permeability *RP* to assess the hydraulic status of a tunnel excavated in London Clay. It was obtained by equating the water flow through London Clay and tunnel lining on the assumption of one-dimensional water flow in post-construction ground consolidation around tunnels. To better address the tunnel hydraulic state, Laver et al. (2016) updated the one-dimensional *RP* definition by Wongsaroj et al. (2013) with a two-dimensional evaluation method of applying a radial water flow pattern around the same London Clay tunnel.

In this study, the deteriorated tunnel lining permeability of Dublin Port Tunnel (DPT) was investigated analytically. An equivalent *RP* model was proposed by assuming ground consolidation in all ground layers. The current degradation state of Dublin Port Tunnel was assessed and evaluated. Furthermore, a series of hydro-mechanical coupled numerical simulations encompassing the time-dependent hydraulic deterioration was conducted to evaluate the ageing performance of this tunnel.

2 HYDRAULIC PROFILE OF DUBLIN PORT TUNNEL

2.1 *Hydraulic conditions*

As a submerged urban road tunnel, Dublin Port Tunnel was equipped with a full set of drainage system that can be categorised into two subsystems: surface water drainage system and ground water drainage system. Of the three distinctive sections of this tunnel, the drainage system for the two cut and cover sections is cut off at the section end that connects the bored section, with the bored tunnel of 2,630m length contributing to drainage flowing towards the main drainage sump of the tunnel buried underneath the invert of each bore at Vehicle Cross Passage (VCP) 16 section, as illustrated in Figure 1(*a*).

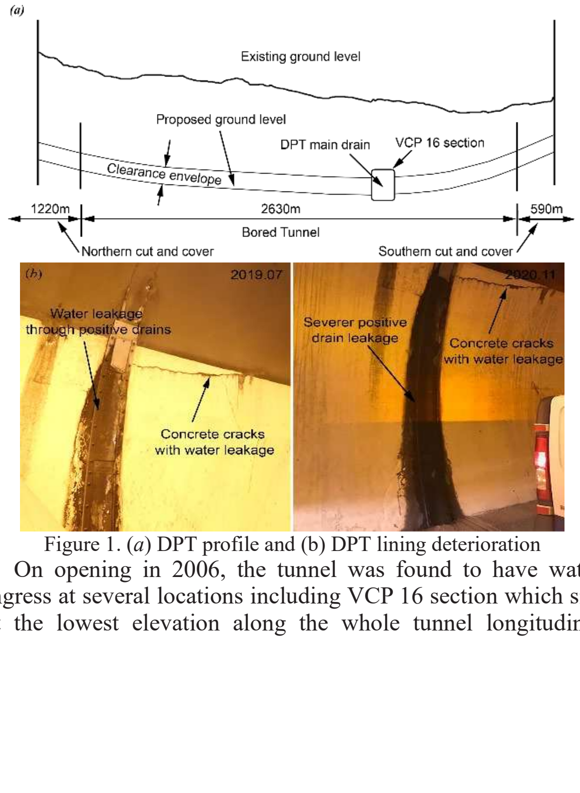


Figure 1. (*a*) DPT profile and (b) DPT lining deterioration

On opening in 2006, the tunnel was found to have water ingress at several locations including VCP 16 section which sits at the lowest elevation along the whole tunnel longitudinal alignment. Repairs comprised replacing the waterproofing membranes and concrete locally. Subsequent inspections and routine maintenance works during operation indicated that tunnel lining deteriorations (e.g. lining cracks and water infiltration) have gradually developed with time. Recently, another site inspection also noted the progressive hydraulic deterioration of tunnel linings, as illustrated in Figure 1(*b*).

According to Dublin Port Tunnel design statements, the initial permeability value for tunnel lining segments in pre-construction state is at 2.0×10^{-13}m/s. As indicated by previous studies, the permeability of tunnel lining evolves with time, implying that the actual lining permeability of Dublin Port Tunnel after decade-plus years of operation remains undetermined. In this study, a modified relative permeability model for the deteriorated VCP 16 section is proposed by referring to Laver et al. (2016), and the lining permeability is then estimated using the water flow monitoring data gathered over the past 9 years.

2.2 *Monitored water flow*

To investigate the hydraulic behaviour of Dublin Port Tunnel, the water flow into the main drainage sump has been monitored since the end of May 2011 on an hourly basis. Considering twin-tunnel interaction effect, the actual water flow rate for a single tunnel may not be simply as half of the recorded water flow rate for the twin tunnel section; in theory, the flow rate for a single tunnel should be smaller than the recorded rate for the twin tunnel (i.e. the upper bound) and greater than half of the recorded rate (i.e. the lower bound) as given in Table 1 where the water data are normalised against the length of the twin bored tunnel section (2,630m). The estimated range of water flow rate into a single tunnel will be used for analytical derivation in the following section.

Table 1. Water flow rate for Bored Tunnel of DPT

Year	Quarterly water flow rate ($\times10^{-7}$ m^3/m/s) (lower bound/upper bound)			
	Q_1	Q_2	Q_3	Q_4
2011	/	7.00/14.0	6.10/12.2	6.90/13.8
2012	7.10/14.2	3.56/7.12	6.80/13.6	2.18/4.36
2013	7.00/14.0	6.90/13.8	6.40/12.8	6.30/12.6
2014	6.60/13.2	6.20/12.4	5.80/11.6	5.30/10.6
2015	5.20/10.4	4.68/9.36	4.25/8.50	3.92/7.84
2016	4.34/8.68	4.01/8.02	4.04/8.08	3.95/7.89
2017	4.10/8.20	3.77/7.54	3.61/7.22	3.65/7.30
2018	3.96/7.92	3.54/7.08	3.21/6.42	3.42/6.84
2019	3.64/7.28	3.84/7.68	4.15/8.30	1.82/3.64
2020	3.10/6.20	3.88/7.76	3.65/7.30	/

3 GROUND-LINING RELATIVE PERMEABILITY *RP*

3.1 *Previous definitions*

3.1.1 *RP definition by Wongsaroj et al. (2013)*

To evaluate the long-term performance of a tunnel excavated in London Clay, Wongsaroj et al. (2013) derived the ground-lining relative permeability, *RP*, by applying the assumption of an unchanging water table in sandy soil layer and by using Darcy's law in an assumed one-dimensional consolidation flow scenario in London Clay, as illustrated in Figure 2.

Considering the hydraulic continuity condition of equal flow through London Clay and tunnel lining gives:

$$k_g(L_c - h_t)/C_{clay} = k_t h_t / t_t \quad (1)$$

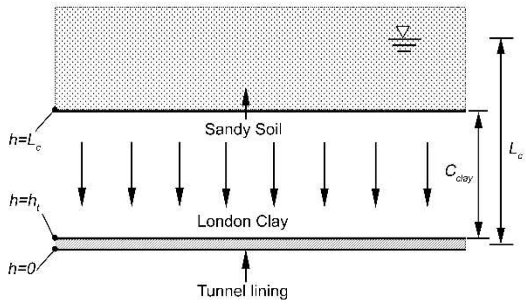


Figure 2. Mathematical model for Wongsaroj's *RP* derivation

with the *RP* definition as:

$$\frac{h_t}{L_c} = \frac{1}{1+RP} \tag{2}$$

$$RP = \frac{C_{clay} k_t}{k_g t_t} \tag{3}$$

where C_{clay} = depth of clay cover; k_t = tunnel lining permeability; k_g = equivalent permeability of surrounding ground; L_c = depth between tunnel axis and water table; t_t = thickness of tunnel lining; h = hydraulic head; and h_t = hydraulic head at lining extrados.

3.1.2 *RP definition by Laver et al. (2016)*

To consider the water flow around tunnel lining more realistically, Laver et al. (2016) extended Wongsaroj et al. (2013)'s work by adopting a two-dimensional radial inflow assumption to define a new expression for *RP*, as shown in Figure 3.

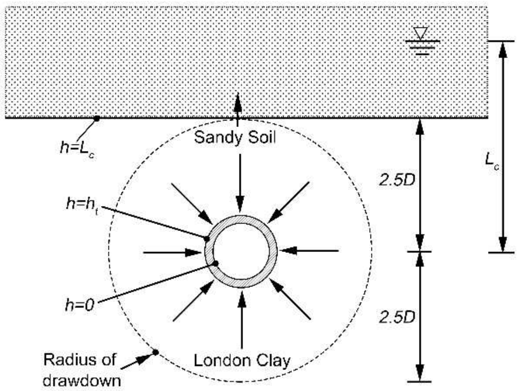


Figure 3. Mathematical model for Laver's *RP* derivation

Assuming homogeneous lining permeability and uniform radial flow into the tunnel, Laver et al. (2016) obtained the new *RP* definition on the basis of equating volumetric flow rate per unit tunnel length through tunnel lining (q_t) to that through London Clay (q_g). The volumetric flow rate through London Clay (q_g) was derived by considering an annular *dr*-thick element at radius *r* from tunnel centre, giving the q_g as follows:

$$q_g = 2\pi r v(r) = 2\pi r k_g \frac{dh}{dr} \tag{4}$$

where $v(r)$ = flow velocity through the element and dh = hydraulic head difference across the element. Integrating equation (4) by applying two boundary conditions (*a*) hydraulic head $h=h_t$ at lining extrados $r=D/2$ and (*b*) hydraulic head $h=L_c$ at a distance of $r=C_{clay}+D/2$ from the far boundary of water drawdown gives the final expression of q_g as:

$$q_g = 2\pi r k_g \frac{(L_c - h_t)}{\ln(2C_{clay}/D+1)} \tag{5}$$

where D = diameter of tunnel external boundary. Similarly, the volumetric flow rate per unit tunnel length through the lining q_t can be found from Darcy's law as:

$$q_t = \pi D k_t \frac{h_t}{t_t} \tag{6}$$

Rearranging $q_g=q_t$ in the form of equation (3) finds a new definition of *RP* as the below:

$$RP = \frac{Dk_t}{2t_t k_g}\ln(2C_{clay}/D+1) \tag{7}$$

3.2 *RP for Dublin Port Tunnel*

Previous studies by Wongsaroj et al. (2013) and Laver et al. (2016) focused on tunnels excavated in London Clay with overlying sandy layer. Both assumed that ground consolidation only occurs in the layer of London Clay. Unlike the geological conditions in their studies, Figure 4 illustrates the simplified geological cross section around VCP 16 section of Dublin Port Tunnel. The tunnel section situates in argillaceous limestone (G3), with a layer of sandy clayey gravel (G1) on top and another layer of sandy gravelly clay (G2) in-between. A quarter model of the VCP 16 section in Figure 5 shows that the layby tunnel is not circular in shape, compared with the previous two scenarios. However, for the whole bored tunnel section of a length of 2630m, the total length of the VCP section in this bored tunnel only accounts for a negligible 80m. To derive an analytical solution, some assumptions regarding tunnel shape, permeability anisotropy, water flow mode, etc. are made: (*a*) permeability anisotropy of tunnel lining and soil stratigraphy is ignored; (*b*) water table at 2.0m below surface ground remains unchanged with time; (*c*) water flows towards the tunnel in a radial pattern; (*d*) the shape of this bored section is considered as a circular tunnel with the same external diameter of 11.22m as bored tunnel.

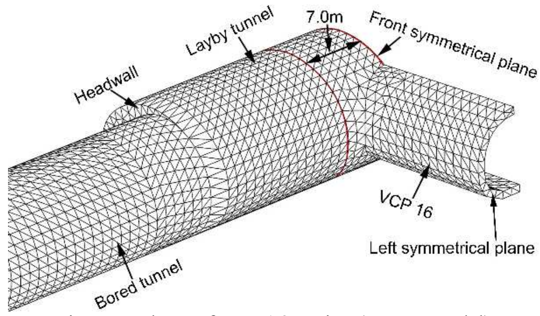


Figure 5. Shape of VCP 16 section (quarter model)

Theoretically, the volumetric flow rate per unit tunnel length through G1 (q_1), G2 (q_2), G3 (q_3) and tunnel lining (q_t) is equal: $q_1=q_2=q_3=q_t$. Assuming an annular *dr*-thick element at radius *r* from the tunnel (Laver et al. 2016), the water flow rate through the first soil layer G1 can be obtained by integrating equation (8) with i=1:

$$q = q_i = 2\pi r k_i \frac{dh}{dr}(i = 1,2,3) \tag{8}$$

The boundary conditions from Figure 4(*a*) are: (*a*) hydraulic head $h=L_c$ at water table level $r=L_c$ and (*b*) hydraulic head $h=h_1$ at the bottom of G1 layer $r= L_c+h_w-t_1$. Substituting the two

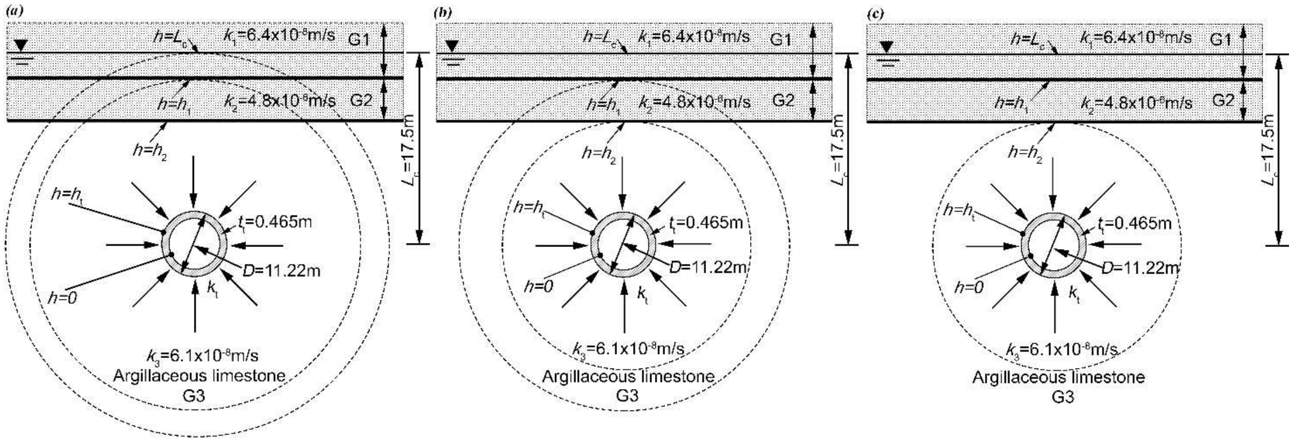


Figure 4. Mathematical model for lining permeability derivation k_t , DPT

boundary conditions into the integration of equation (8), when i=1, gives the hydraulic head at the bottom of G1 layer as:

$$h_1 = L_c + \frac{q}{2\pi k_1}\ln(\frac{L_c + h_w - t_1}{L_c}) \tag{9}$$

Similarly, through integration of equation (8) for G2 layer (i=2) with boundary conditions as (a) hydraulic head $h=h_1$ at the top of G2 layer (bottom of G1 layer) $r= L_c+h_w-t_1$ and (b) hydraulic head $h=h_2$ at the bottom of G2 layer (top of G3 layer) $r= L_c+h_w-t_1-t_2$, the final expression for the hydraulic head at the bottom of G2 layer is established as:

$$h_2 = h_1 + \frac{q}{2\pi k_2}\ln(\frac{L_c + h_w - t_1 - t_2}{L_c + h_w - t_1}) \tag{10}$$

Finally, water flows through the third layer G3 with a flow rate of q_3 as in equation (8) when i=3, and by applying the following two boundary conditions: (a) hydraulic head $h=h_2$ at the bottom of top of G3 layer (bottom of G2 layer) $r= L_c+h_w-t_1-t_2$ and (b) hydraulic head $h=h_t$ at tunnel lining extrados $r=R=D/2$, the integration of equation (8) (i=3) gives the hydraulic head at the extrados of tunnel lining as:

$$h_t = h_2 + \frac{q}{2\pi k_3}\ln(\frac{D/2}{L_c + h_w - t_1 - t_2}) \tag{11}$$

In terms of the flow rate per unit tunnel length for the tunnel lining, it can be derived from Darcy's law as the below:

$$q = q_t = \pi D k_t \frac{h_t - 0}{t_t} \tag{12}$$

The final expression of tunnel lining permeability can be derived by combining and rearranging equations (8)~(12). Moreover, the relative permeability RP between tunnel lining and argillaceous limestone can be obtained as:

$$\frac{h_t}{h_2} = \frac{1}{1+RP} \tag{13}$$

$$RP = \frac{Dk_t}{2t_t k_3} \times \ln[\frac{2(L_c + h_w - t_1 - t_2)}{D}] \tag{14}$$

3.3 *Modified RP for Dublin Port Tunnel*

As mentioned in section 3.2, the relative permeability RP proposed in Wongsaroj et al. (2013) and Laver et al. (2016) was derived by considering long-term post-construction water flow only in the low-permeability London Clay, but not in the highly-permeable sandy layer above it. However, this assumption is not applicable to the case of Dublin Port Tunnel where the long-term groundwater flow in the two ground layers overlying the argillaceous limestone should not be neglected, as the permeability of all three ground layers is of similar magnitude. Therefore, the ground permeability k_3 in equation (14) should be replaced with the equivalent permeability k_e of the three ground layers.

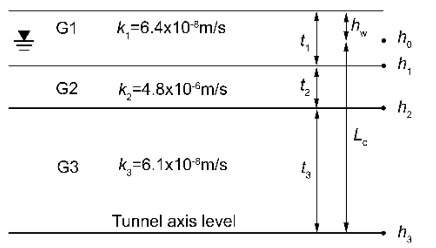


Figure 6. Ground stratigraphy around VCP 16 section

Based on Darcy's law and the mathematical model shown in Figure 6, the flow rate per unit area through each ground layer should be the same, as indicated in equation (8). The flow rate through each ground layer q_i (i=1, 2 and 3) is presented as follows:

$$q = q_1 = k_1\frac{\Delta h_1}{l_1} = k_1\frac{h_0 - h_1}{t_1 - h_w} \tag{15}$$

$$q = q_2 = k_2\frac{\Delta h_2}{l_2} = k_2\frac{h_1 - h_2}{t_2} \tag{16}$$

$$q = q_3 = k_3\frac{\Delta h_3}{l_3} = k_3\frac{h_2 - h_3}{L_c + h_w - t_1 - t_2} \tag{17}$$

Likewise, the flow rate through the equivalent ground layer can be calculated as:

$$q = q_e = k_e\frac{\Delta h_e}{l_e} = k_e\frac{h_0 - h_3}{L_c} \tag{18}$$

Combing equations (15) through (18), the equivalent ground permeability k_e is expressed as the below:

$$k_e = \frac{L_c}{\frac{t_1 - h_w}{k_1} + \frac{t_2}{k_2} + \frac{L_c + h_w - t_1 - t_2}{k_3}} \quad (19)$$

Substituting the equivalent permeability k_e in equation (19) into equation (14) gives:

$$RP = \frac{Dk_t}{2t_t k_e} \times \ln\left(\frac{2(L_c + h_w - t_1 - t_2)}{D}\right) \quad (20)$$

where t_i (i=1, 2) is the thickness of first and second ground layer; t_i (i=3) is the depth from the top of the third ground layer to the tunnel axis level; h_w is the depth from the surface ground to the underground water level; k_i (i=1, 2 and 3) is the permeability coefficient of ground layers; L_c is the depth between tunnel axis and water table; h_0 is the hydraulic head at the underground water level; h_1 is the hydraulic head at the bottom of the first soil layer; h_2 is the hydraulic head at the bottom of the second soil layer; h_3 is the hydraulic head at the tunnel axis level.

4 LINING PERMEABILITY OF DUBLIN PORT TUNNEL

4.1 *Lining permeability*

As mentioned in section 2.2, the water drainage data for the bored twin tunnel section of DPT should fall within the range between the flow rate for a single tunnel and that for two single tunnels superposed. Therefore, the current deteriorated twin tunnel lining permeability can be calculated by combining equations (8) ~ (12) with the flow rate data for a single tunnel of [1.82, 14.2]×10^{-7} m^3/m/s given in Table 1. The calculated lower and upper bound of the lining permeability values are listed in Table 2.

Table 2. Calculated tunnel lining permeability on a quarterly basis

Year	Tunnel lining permeability (×10^{-10}*m/s*) (lower bound/upper bound)			
	Q_1	Q_2	Q_3	Q_4
2011	/	6.20/12.4	5.30/10.6	6.10/12.2
2012	6.40/12.7	2.88/5.75	6.10/12.1	1.69/3.38
2013	6.20/12.4	6.20/12.3	5.60/11.2	5.50/10.9
2014	5.80/11.6	5.40/10.7	4.96/9.92	4.51/9.01
2015	4.41/8.82	3.90/7.80	3.51/7.01	3.35/6.39
2016	3.59/7.17	3.28/6.56	3.31/6.61	3.23/6.45
2017	3.36/6.72	3.06/6.12	2.92/5.84	2.96/5.91
2018	3.24/6.47	2.86/5.71	2.57/5.14	2.75/5.50
2019	2.95/5.90	3.13/6.25	3.41/6.82	1.40/2.79
2020	2.47/4.93	3.17/6.33	2.95/5.90	/

Table 2 shows that the tunnel lining deteriorated hydraulically over time, with the initial lining permeability of 2.0×10^{-13} m/s increasing to a range between [1.40×10^{-10}, 1.27×10^{-09}] m/s during 14 years of operation, in line with the previous findings by Bagnoli et al. (2015), Li et al. (2020), etc.

4.2 *Tunnel hydraulic state*

After determining the current tunnel lining permeability, the modified relative ground-lining permeability for Dublin Port Tunnel can be derived by substituting the initial permeability and deteriorated permeability of tunnel lining into equation (20). The modified *RPs* for Dublin Port Tunnel are *RP*=2.33×10^{-5} and *RP* ∈ [1.63×10^{-2}, 1.48×10^{-1}] for the initial and current deteriorated states, respectively. Based on extensive numerical simulations on long-term settlement above a single tunnel with various lining permeability, a best-fit relationship between ground-lining relative permeability *RP* and dimensionless settlement *DS* was proposed in Laver et al. (2016):

$$DS = \frac{1}{1 + 1.4RP^{-1}} \quad (21)$$

$$DS = \frac{NS_{c\max(ss)} - NS_{c\max(ssi)}}{NS_{c\max(ssp)} - NS_{c\max(ssi)}} \quad (22)$$

where $NS_{c\max(ss)}$ is the non-dimensional consolidation-induced long-term maximum surface settlement in steady-state condition; $NS_{c\max(ssp)}$ and $NS_{c\max(ssi)}$ are the same settlement for fully permeable and fully impermeable cases, respectively. *DS* falls within the range of [0, 1] where tunnel lining can be regarded as fully permeable when *DS*=1 and fully impermeable when *DS*=0, as indicated in the following *S*-curve in Figure 7.

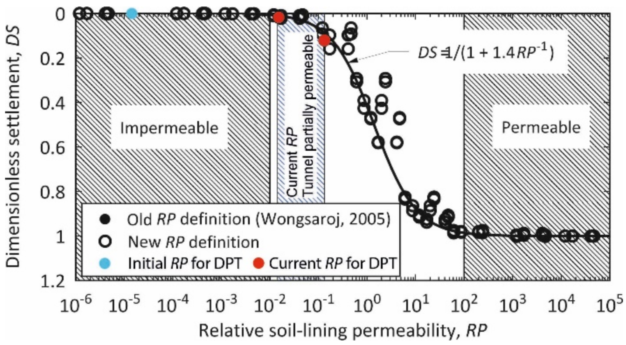


Figure 7. *RP* against *DS* (Laver et al. 2016)

The *DS* remains zero when *RP* < 10^{-2}, representing a fully impermeable lining system. As *RP* increases over 10^2 when *DS*=1, tunnel lining becomes fully permeable.

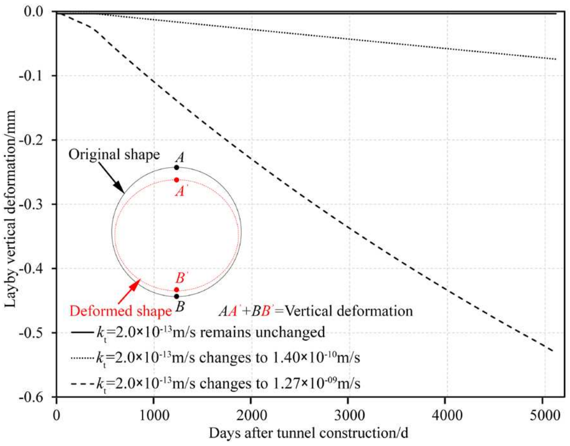
For Dublin Port Tunnel, the initial *RP* = 2.33×10^{-5} is far smaller than 10^{-2}, indicating fully impermeable tunnel lining of Dublin Port Tunnel before construction, which satisfies the design requirement of watertight tunnel. After decade-plus tunnel operation, the relative permeability gradually increased to *RP* ∈ [1.63×10^{-2}, 1.48×10^{-1}]. That is, the tunnel has started to hydraulically deteriorate as *RP* has exceeded the threshold for impermeable lining definition, and the tunnel lining is currently partially permeable compared to its initial pre-construction state, as shown in Figure 7. However, when compared with the permeability of the surrounding ground layer of 6.10×10^{-08} m/s, the tunnel section is still relatively impermeable [1.40×10^{-10}, 1.27×10^{-09}] m/s, even after 14 years of operation.

5 AGEING PERFORMANCE OF DUBLIN PORT TUNNEL

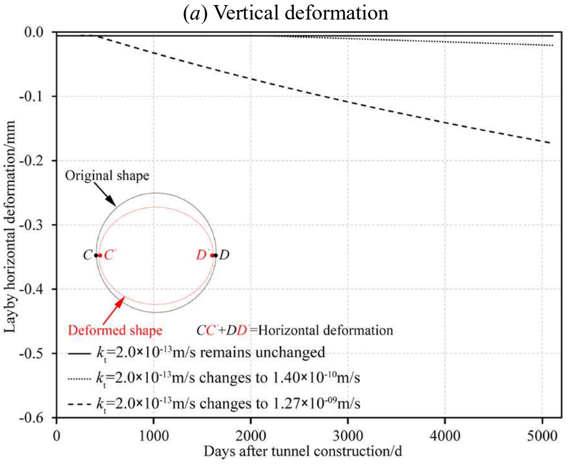
Most previous studies focused on the effect of different tunnel lining permeability on tunnel performance and assumed a constant permeability coefficient during the tunnel lifetime (Wongsaroj et al. 2013; Li et al. 2015; Laver et al. 2016). However, they failed to demonstrate the practical time-dependent hydraulic deterioration of tunnels. By applying the deteriorated lining permeability obtained in section 4, a set of hydro-mechanical coupled numerical simulations is conducted to evaluate the ageing behaviour of Dublin Port Tunnel. The tunnel lining was assumed to be watertight/fully impermeable before the long-term consolidation begins. The initial lining permeability of 2.0×10^{-13}m/s was assigned to all linings. With time, it was assumed that the lining permeability started to increase linearly during the 14-year operational period to the current deteriorated values of 1.40×10^{-10}~1.27×10^{-09} m/s at the end of the period (Li et al. 2020).

5.1 *Tunnel deformation*

Figure 8 shows the deformation of layby tunnel with time along the full ring section (7m from the front symmetrical plane, as shown in Figure 5) for three scenarios.



(*a*) Vertical deformation



(*b*) Horizontal deformation

Figure 8. Layby tunnel deformation

Figure 8 shows that if lining permeability remains unchanged at k=2.0×10^{-13}m/s throughout the 14-year period, the tunnel deformation at both vertical and horizontal directions increases to a peak value (around 0.01mm) in a short period and then stabilises thereafter. In contrast, for the other two cases incorporating tunnel hydraulic deterioration, the tunnel deforms with time linearly and its deformation sees no sign of levelling off at the end of the 14-year period. As the tunnel permeability increases to k=1.27×10^{-09}m/s, the accumulated tunnel deformations at both directions are larger than the lower bound case where k increases to 1.40×10^{-10}m/s. This is because the dissipation of pore water pressure around the tunnel depends on the tunnel hydraulic boundary conditions. The pore water pressure dissipation is barely observed when the tunnel lining remains fully impermeable. In addition, the greater the lining permeability is, the more rapid the dissipation of pore water pressure is, thus leading to the development of tunnel deformation at a faster rate.

In summary, the assumption of a constant hydraulic permeability during tunnel lifetime may underestimate the tunnel deformation. To evaluate the deformational performance of tunnels more realistically, the deterioration process (ageing process) of tunnel structures should be taken into account.

6 CONCLUSIONS

This study aims to estimate the current deteriorated hydraulic permeability of an operational Dublin Port Tunnel and reveal its ageing performance with time. The following conclusions can be drawn:

- A modified relative permeability *RP* for Dublin Port Tunnel was proposed assuming that the post-construction ground consolidation occurs in all ground layers.
- An analytical model was proposed to determine tunnel hydraulic permeability. Based upon the analytical model, the current hydraulic state for DPT was categorised as partially permeable after 14 years of operation.
- Compared with conventionally assumed constant tunnel permeability, the time-dependent tunnel permeability evolution will result in larger tunnel deformation, highlighting the significance of hydraulic deterioration on long-term tunnel structural behaviour with time.

In this study, only hydraulic deterioration was considered whilst the lining permeability was assumed to increase linearly with time. Practically, however, tunnels also deteriorate structurally, such as lining stiffness reduction, and the hydromechanical deterioration may not follow a linear relationship. The proposed method to determine lining permeability may be inaccurate because the monitored water inflow consists of all water directed towards the drainage sump (including water sourced from tunnel maintenance). Further research can be performed to fill such mentioned gaps.

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