Transportation Geotechnics 45 (2024) 101185

Contents lists available at ScienceDirect

Transportation Geotechnics

journal homepage: www.elsevier.com/locate/trgeo

Economics of geotechnical asset deterioration, maintenance and renewal

John Armstrong [a,*], Peter Helm [b], John Preston [a], Fleur Loveridge [c]

[a] Transportation Research Group, University of Southampton, Boldrewood Campus, Burgess Road, Southampton SO16 7QF, UK
[b] School of Engineering, Newcastle University, Newcastle upon Tyne NE1 7RU, UK
[c] School of Civil Engineering, University of Leeds, Leeds LS2 9JT, UK

ARTICLE INFO

Keywords:
Earthworks
Maintenance
Renewals
Cost-benefit analysis
Serviceability

ABSTRACT

Transport and other infrastructure systems are supported on, adjacent to and retained by extensive systems of earthworks of varying (and increasing) age, and of variable original construction quality. These earthworks are subject to natural deterioration, which can be accelerated and complicated by the effects of climate change. The ACHILLES research program is providing improved understanding of earthworks behavior, performance and deterioration. It is also developing methods and tools to analyze and provide decision support for the construction, maintenance and renewal of earthworks, with particular emphasis on the management of existing, deteriorating assets. The work described here aims to identify the most cost-effective timing and means of extending earthworks asset lives and maintaining their safety and serviceability. Conventional cost-benefit analysis methods, of the type used for new infrastructure projects, do not directly provide the decision support needed for the maintenance and renewal of existing earthworks assets. An alternative approach is proposed and applied to a modeled example, demonstrating the potential asset management benefits of early, pre-emptive intervention, the economic attraction of deferred intervention, and the means of identifying an intermediate whole-life cost 'sweet spot', based on a timely assessment of intervention options. The handling of the uncertainty associated with earthworks behavior, deterioration rates and times to failure is also considered, as is the extension of the single-asset approach to the management of multiple earthworks assets.

Introduction

A high proportion of transport and other infrastructure is supported on embankments or is adjacent to engineered cutting or natural slopes. This is particularly true of railways, which require extensive earthworks to provide consistent vertical alignments and comparatively shallow gradients. For example, according to the Office of Rail and Road [35], Britain's heavy rail network has a total route length of almost 16,000 km, while the total length of earthworks on the network, treating opposite sides of the tracks separately, is approximately 19,000 km [31]. Many railway earthworks were originally built in the nineteenth century, to relatively primitive standards of design and construction [5]. As the infrastructure manager (IM) of Britain's heavy rail network, Network Rail [31] estimates that a large proportion of these earthworks are over 150 years in age, and a subset are greater than 170 years old. The continuing effects on them of natural deterioration processes [6] (this issue) are now exacerbated by the increasingly extreme weather associated with climate change.

Many of these earthworks are safety–critical, supporting railways or highways, or retaining water, for example. Their failure can therefore have catastrophic consequences [32,43], while their deterioration to a condition in which they can no longer function normally and safely can result in severe and extended disruption to transport and other services, with significant socio-economic (and possibly environmental) impacts. An example of this occurred in 2019, with the near-collapse of the Toddbrook Reservoir dam near Whaley Bridge in Derbyshire, as reported by the BBC [3] and New Civil Engineer [30].

While safety and serviceability (i.e. the ability of the infrastructure to sustain a normal standard of service, without the imposition of speed or axle load restrictions, for example) are the overriding priorities for the owners and maintainers of earthworks, cost-effective and economically efficient approaches to maintenance are also important. This is especially the case for custodians of large portfolios of earthworks assets for which the available maintenance budget is limited [31]. Preventive maintenance is generally much cheaper and more cost-effective than urgent repairs (corrective maintenance) to an asset that has failed or is in

* Corresponding author.
E-mail addresses: j.armstrong@soton.ac.uk (J. Armstrong), peter.helm@newcastle.ac.uk (P. Helm), j.m.preston@soton.ac.uk (J. Preston), f.a.loveridge@leeds.ac.uk (F. Loveridge).

https://doi.org/10.1016/j.trgeo.2024.101185
Received 28 April 2023; Received in revised form 11 January 2024; Accepted 13 January 2024
Available online 19 January 2024
2214-3912/

imminent danger of doing so [42], quite apart from the safety risks involved. Planned as opposed to emergency maintenance can also be much less disruptive to users. However, the challenge of the efficient deployment of a maintenance budget is heightened by the fact that the need for maintenance may not become readily apparent until an asset's condition has deteriorated to a point where it is "actively failing [and] accelerated intervention [is required] to prevent catastrophic failure" [31], restricting the range of intervention options available and increasing the costs of those that remain.

The ACHILLES (Assessment, Costing and enHancement of long lIfe, Long Linear assEtS) research program [1] addresses these challenges faced by earthworks designers, owners and maintainers. Its vision is.

for the UK's infrastructure to deliver consistent, affordable and safe services, underpinned by intelligent design, management and maintenance

and it is enabling this by meeting three Research Challenges:

1. *Deterioration Processes*, by which materials and assets degrade over time;
2. *Asset Performance*, with and without engineering interventions; and
3. *Forecasting and Decision Support*, to forecast asset (and network) behavior with and without interventions, and to identify improved intervention strategies.

ACHILLES is thus improving the understanding and prediction of earthworks behavior, and developing decision support processes to identify improved strategies for earthworks maintenance, renewal and enhancement, and reduced whole-life asset and network costs, while maintaining required levels of safety and serviceability. This work is being delivered through four complementary themes and workstreams:

1. *Performance and Deterioration* (PaD);
2. *Monitoring and Measurement* (MaM);
3. *Simulation and Modeling* (SaM); and
4. *Design and Decisions* (DaD).

This paper describes work being undertaken in the DaD workstream, based upon inputs from SaM, to provide decision support with the aim of maintaining earthworks' serviceability and safety, while reducing their whole-life costs. It analyses the costs and benefits of earthworks maintenance and renewals, and considers how such interventions should be scheduled to reduce (and ideally minimize) overall costs, while maintaining asset safety and serviceability.

Following this introduction, the background to, and context for, the work are set out. The economics of earthworks asset maintenance and renewal are then considered and compared with the more conventional approaches used for the assessment of new infrastructure, and an overview is provided of the geotechnical modeling approaches used. The proposed analytical approach and an illustrative example calculation are then presented, including further details of the geotechnical modeling of soil nail installations. These are followed by the description of a modeled intervention example and the results, equivalent to those for the hypothetical case, obtained from it. The findings are then discussed and some conclusions are drawn.

Background and Context

Successive waves of transport and other technological and engineering developments since the first Industrial Revolution in the eighteenth century have produced an extensive legacy of canals, railways, highways, reservoirs, flood defenses and other infrastructure. Much of this infrastructure includes earthworks, with approximately "two-thirds of the UK transport infrastructure network [being] supported by or adjacent to engineered slopes" [10]. These long-lived linear assets (LLAs) deteriorate with age and are vulnerable to the extreme weather events increasingly associated with climate change [8], which in the UK is predicted to lead to wetter winters and dryer summers, with higher intensity rainfall events and more intense drying due to higher average temperatures [27]. Climate change and the associated extreme weather events are projected to occur globally [22] and these extreme events are becoming more commonly observed, with heavy rainfall and dangerous flooding of the type seen in 2021 in Belgium, Germany and the Netherlands [44] and in central China, and more recently in Pakistan, being perhaps the most obvious examples of this. However, earthworks are also vulnerable to the effects of drought and of cyclic wetting and drying, which can cause seasonal ratcheting movements and strain softening along with the development of macro-permeability features (desiccation cracks) at the near surface in plastic clays [49,47]. As such, the risk that climate change and extreme weather poses to transportation infrastructure has been formally recognized more broadly, for example in Europe [12] and the US [34]. The evidence base for these vulnerabilities is supported by recent modeling work which has shown that future climate change is likely to increase the rate of slope deterioration compared to the present [45] and that long periods of wet weather can lead to increased deterioration rates [41].

These developments are particularly challenging for railway IMs, given the extent and typical age of 'classic' railway systems, and the relatively rudimentary engineering standards and methods employed in their original construction [5]. Railway infrastructure has also been recognized as being particularly vulnerable to extreme weather events [12]. Many of the world's major highway networks, while designed and constructed more recently and to higher engineering (and documentation) standards, were built between the 1950s and 1970s, are now over fifty years old, and are also subject to natural deterioration processes, again complicated and potentially accelerated by the effects of climate change.

The maintenance, renewal and enhancement of earthworks is challenging and expensive, but much less so than dealing with earthworks failures, the related safety implications and the costs of disruption and delay to users. In the case of Britain's railways, it is estimated [14] that "emergency repairs cost 10 times more than planned works." This is greater than the general emergency to preventive cost ratio range of five to seven quoted by the Prometheus Group [42], but may be due in part to the general lack of redundancy in railway networks, and the increased resulting impacts on users. Problems with water-retaining structures can also cause disruption to and require the evacuation of downstream locations, as in the case of the Toddbrook reservoir mentioned above [3]. In the transport context, earthworks failures can, in the worst cases, cause death and injury to users and staff, and lengthy route closures, as in the case of a train derailment in August 2020 at Carmont in Scotland. This incident followed very heavy rain and the consequent washing of debris from an incorrectly installed drainage system onto the track, as reported by Network Rail [32] and Britain's Rail Accident Investigation Branch (RAIB) [43].

To improve the resilience of earthworks, recommendations have been made by the UK Department for Transport (DfT) [8] to investigate means of slope stabilization short of rebuilding. Furthermore, the UK National Infrastructure Commission (NIC) [33] states that.

predictive asset management models, could provide more detailed information on asset condition and performance, helping to prevent failures and better target maintenance interventions or renewals.

ACHILLES is addressing these twin recommendations, by undertaking a program of research work with the aims of 1) assessing rates of deterioration of earthworks as driven by weather (see, for example, [16], this issue); 2) analyzing the change in rate of deterioration and time to failure with differing tims of intervention; and 3) evaluating the relative costs and benefits of interventions at different times versus allowing a slope to fail and then rebuilding it. Elements of this work are described in the following sections.

Material and Methods

Economics of New Construction vs. Maintenance of Existing Assets

The economic assessment element of the decision-making process for proposed new infrastructure, including earthworks, is well-established and comparatively straightforward: the anticipated benefits of a scheme over a given lifespan or other time period are calculated and compared with its costs over the same period. In the case of transport infrastructure, these benefits typically take the form of journey time savings and safety improvements, in a ratio of approximately 80:20 for highway schemes. The safety benefits of rail schemes tend to be smaller, unless there is significant modal shift from road, since rail is inherently relatively safe already. Capacity increases and reduced overcrowding can be a significant additional benefit of rail schemes like London's Crossrail (now the Elizabeth Line). The corresponding costs are those associated with initial construction and ongoing operation and anticipated maintenance needs. In the case of most infrastructure schemes, benefits are accrued over extended periods, typically decades, and sometimes centuries (in the case of railway infrastructure, for example), and the same is true for operational and maintenance costs. Discounting is usually applied in such cases, reflecting individual and social time preference, i.e. the general preference of individuals and society "to receive goods and services sooner rather than later" [19]. This means that benefits obtained and costs incurred in future years are assigned a smaller present financial value than those obtained and incurred earlier. If the present value of the benefits (PVB) of a scheme exceeds the present value of its costs (PVC), it is in principle worth investing in, and the greater the difference between the PVB and PVC (its net present value, or NPV), the more economically worthwhile the scheme. When choosing between alternative, mutually exclusive options for the implementation of a given infrastructure scheme, the options can be ordered by NPV. Independent, mutually inclusive schemes (i.e. schemes which can be undertaken in combination) can be ordered by Benefit:Cost Ratio (BCR = PVB/PVC), with the options providing the highest BCRs being prioritized for implementation (all other considerations being assumed equal).

Such formal methods of cost-benefit analysis (CBA) were largely developed for transport investment decision-making purposes in the post-WW2 era, particularly to inform and facilitate the major national highway construction programs that took place across the developed and developing worlds from the 1950s onwards. These approaches can also be applied to the enhancement or upgrading/expansion of existing infrastructure, for example to the widening of highways to deliver additional capacity and (at least temporarily) relieve congestion, again providing journey time reduction benefits that can be compared with the expansion costs. As well as being amenable to conventional CBA, such new construction and upgrades can be politically attractive, providing opportunities for 'ribbon cutting' and proclamations about investment in infrastructure capacity and safety, and in reduced travel times and improved reliability.

Most of Britain's railways were built long before the development and implementation of formal CBA methods, and many of them remain in use almost two centuries after their original construction. Conventional CBA for new transport infrastructure in Great Britain is generally based upon a 60-year appraisal period, as specified in the Transport Analysis Guidance (TAG) provided by the DfT [9]. Decades-long appraisal periods may sometimes be too long to accurately forecast traffic growth, for example, and shorter timescales may be more appropriate. In other cases they may be too short, as in the case of the very long-lived 19th century railway infrastructure that continues in use today, and the TAG acknowledges that continuing maintenance and renewal of assets "effectively means that the asset life will be indefinite." The longer the appraisal period that is used, the greater the level of inherent general uncertainty involved, and 60 years is perhaps a sensible general upper limit in that regard. However, the guidance also notes that some projects have "design lives of 100 years or more before a major renewal is needed." High-speed railway (HSR) systems are a case in point, requiring careful consideration of the trade-offs, for example, between using steeper earthworks slopes to provide short-term savings in land-take and construction costs, but at the increased risk of subsequent disruptive and costly slope failures, and the longer-term benefits resulting from higher initial construction costs. The lack of redundancy and alternative routing options in HSR systems (and railways generally) makes them especially vulnerable to such asset failures. In the case of the High Speed 2 (HS2) HSR under construction in Great Britain between London and Birmingham, the infrastructure, including earthworks slopes, has a design life of 120 years [20], while London Underground [37,38] and National Highways [18] have similar requirements. These design lives are adopted on the basis that major infrastructure renewals should not be required within that timescale; however, in practice, and as noted above, the actual lifespans of these infrastructure systems are likely to be much greater, as long as they continue to be maintained and renewed as necessary. Network Rail adopts a different approach, as it is responsible for an asset portfolio where the majority of the earthworks are already greater than 150 years old, with some sections exceeding 170 years in age. As such, it specifies a serviceable life for renewals (i.e. sections of earthwork that have undergone significant repairs) of 120 years. It is also worth noting that the serviceability requirements for these different asset types vary, with tolerances for vertical and horizontal alignment change being far more stringent for HSR than for highways, for example. This is likely to have implications for the acceptable design slope angles for both new construction and the renewal of existing slopes, and also for acceptable levels of heave in the base of deep cuts in plastic clays, which should be carefully considered during the design process, along with the specification, construction and monitoring/maintenance of subgrade.

In contrast to new construction and major capacity enhancements, while routine maintenance (such as ballast cleaning and tamping and vegetation clearance) and like-for-like renewal of infrastructure are essential to provide continuing levels of serviceability and safety, they tend not to produce any noticeable improvement for users (or politicians) relative to its previous state. They nonetheless typically cause disruption to those same users, and incur (sometimes substantial) costs which may not have been included within the timescale of the original CBA for the infrastructure, and for which additional funding must be found. An example of this problem can be found in the US Interstate highway system, most of which was initially built between the 1950s and 1970s with generous federal as well as state government funding, but had ([11], p340)

> *… been deteriorating since the moment it was finished – before it was finished, actually, since old parts of the system … were already antiquated by the time the newer parts were built.*

By the mid-1970s (ibid, p337),

> *… it was becoming clear that state and other local authorities would be hard-pressed to find the revenues for [highway] upkeep on their own.*

This is supported in [36], p269, where it is observed that this

> *… decay had taken place because the federal government did not appropriate a single dollar for maintaining the Interstates until 1976. That had been left to the states; and with federal largess constantly dangling before them, the states found it easier and cheaper to build than to repair.*

The natural deterioration of infrastructure over time is worsened by higher-than-anticipated volumes and masses (in terms of axle loads) of traffic, which are then in turn subject to the delays and inconvenience caused by maintenance activities. Natural deterioration processes can be further exacerbated and complicated by the uncertain impacts of climate change, which may again require expensive interventions simply to ensure that transport and other services can (hopefully, given the uncertainty involved) be maintained, while not necessarily providing any readily apparent net additional benefit to users. Ideally, resilience-driven interventions in response to climate change should provide additional wider benefits, while, conversely, interventions for other reasons should also provide additional climate resilience – the availability of alternative routes may provide an 'amenity benefit', reducing the impacts of disruptive events.

Since such essential maintenance and renewal activities in themselves produce no additional capacity or travel time benefits (except relative to a situation where assets are allowed to deteriorate further), the PVB element of a conventional CBA is absent, with benefits instead being generated in the form of the avoided costs of failure. The economic assessment therefore depends instead upon a comparison between alternative PVCs, based, in this case, upon the assumed and/or modeled geotechnical performance of earthworks, with and without intervention.

Geotechnical Modeling Overview

As noted in the Background and Context section above, earthworks are vulnerable to various aspects of the increasingly extreme weather associated with climate change. In [6] (this issue), it is observed that these vulnerabilities include climate-related alterations to slope surfaces, including variations in vegetation cover, and changes to earthworks' pore water pressure cycles, as well as the direct impacts of increasingly intense individual rainfall and drought events. These higher-intensity rainfall events will likely lead to greater levels of runoff, placing greater demands on drainage systems, which, where present, also undergo deterioration in performance and so tend to be in varying states of repair [37,38]. Higher intensity, shorter duration rainfall events are also thought to increase the risk of shallow failure mechanisms [26] and can lead to debris slides such as that seen at Carmont in the UK [32,43]. The larger magnitude of drying may cause increased desiccation cracking [47], especially in plastic clays, affecting the porosity and permeability of the material and hence the infiltration, movement and storage of water [21]. The larger seasonal cycles of wetting and drying can also drive seasonal ratcheting movements and strain softening [41]. It is this long-term process of deterioration, rather than specific shorter-term triggers, which is captured in the geotechnical modelling described below, and subsequently used to provide context to and give recommendations around maintenance investment decisions.

These geotechnical inputs to the economic analysis of earthworks interventions and failures were obtained from detailed modeling of an earthworks cutting slope subjected to changing seasonal weather representative of the current climate of the southern UK. The modeling adopted a commercial finite difference code, FLAC (Fast Lagrangian Analysis of Continua) with Two Phase Flow [23] and made use of a meteorological surface boundary flux derived from an external soil water balance model. The geotechnical model was used to perform coupled consolidation analysis whereby deformations causing changes in material volume would drive pore fluid flow or pore pressure changes and vice versa.

The baseline model, described hereon as the 'Do Nothing' model (i.e. without any interventions) was of an 8 m high, 1V in 3.5H cut slope excavated in high plasticity overconsolidated clay (the London Clay). The model was hydrologically validated [45,41] using field data [46] and adopted a nonlocal strain softening constitutive model with pressure dependent stiffness, where the softening behavior was validated against consolidated-undrained triaxial tests (see [45]), the nonlocal behavior was calibrated by comparing the results of modeled biaxial tests with differing mesh densities (see [41]) and the stiffness was validated against oedometer test results (see [41]).

The modeled intervention took the form of soil nailing, a method adopted by Network Rail [32] for earthworks renewals, installed at alternative points in the modeled slope's lifecycle, corresponding to selected levels of slope deterioration. The coupled consolidation slope modeling background and approach is summarized in the sections of this paper covering the constitutive model, flow behavior and model geometry with initial and boundary condition, and the modeling approach adopted for the soil nailing is described in more detail in the Geotechnical Modeling Calculations section. For additional details, readers are directed to [7], [45] and [41].

Constitutive model

The model adopts a stress dependent stiffness whereby the elastic modulus, $E^{'}$, is made a function of the mean stress, $p^{'}$, as follows:

$$E^{'} = \frac{E0(p^{'}+100)}{100}$$

where E0 is the reference elastic modulus at atmospheric pressure and

$$p^{'} = \frac{1}{3}\left(s_{xx}+s_{yy}+s_{zz}\right) - p_w$$

where s_{xx}, s_{yy}, and s_{zz} are the Cartesian total stresses and p_w is the pore water pressure.

The yield criterion adopted is a Mohr-Coulomb strain softening model which makes the shear strength properties ($\phi^{'}$, $c^{'}$) a variable function of the plastic shear strain ($\Delta\varepsilon^{p}$):

$$\frac{\Delta\varepsilon^{p}}{2} = \sqrt{\frac{1}{2}(\Delta\varepsilon^{p1}-\Delta\varepsilon^{pm})^2+\frac{1}{2}(\Delta\varepsilon^{pm})^2+\frac{1}{2}(\Delta\varepsilon^{p3}-\Delta\varepsilon^{pm})^2}$$

where $\Delta\varepsilon^{p1}$ and $\Delta\varepsilon^{p3}$ are the major and minor principal plastic strains and

$$\Delta\varepsilon^{pm} = \left(\Delta\varepsilon^{p1}+\Delta\varepsilon^{p3}\right)/3$$

This allows the strength of the soil to undergo softening due to the swelling caused by stress relief following the excavation of the cut slope but also due to the seasonal cycling of pore water pressures driven by the weather boundary which acts as a flux at the model surface.

In order to reduce the influence of mesh dependency on the model results, a nonlocal regularization approach [2] was adopted which requires the inclusion of an additional softening parameter, the internal length, l_i. This approach performs averaging of the plastic shear strains in neighboring zones to derive a nonlocal plastic shear strain ε^{p^*} to calculate the magnitude of softening. ε^{p^*} is calculated as follows:

$$\varepsilon_z^{p^*} = \frac{1}{A_w}\sum_{z_n=1}^{z_t}\omega_{z_n}\varepsilon_{z_n}^{p}A_{z_n}$$

where z_n is the number of a zone neighboring z, z_t is the total number of zones within the radius of influence ($r_i \cong 3l_i$), ω_{z_n} is the weighting function calculated for z_n, A_{z_n} is the area of z_n and A_w is the sum of weighted zone areas:

$$A_w = \sum_{z_n=1}^{z_t}\omega_{z_n}A_{z_n}$$

The weighting function is derived from l_i using the approach suggested by [13] as follows:

$$\omega(r) = \frac{r}{l_i^2}^{-\left(\frac{r}{l_i}\right)^2}$$

where r is the distance between zone z, for which nonlocal softening is being derived, and neighboring zone z_n, within r_i.

Saturated and unsaturated flow behavior

The geotechnical model is capable of modelling saturated and unsaturated water and air phase flow through a porous medium. The water (q_w) and air (q_a) flow are described by Darcy's law as follows:

$$q_w = \frac{k_r^w}{g\rho_w}\frac{\partial}{\partial x_j}(p_w - \rho_w g_k x_k)$$

$$q_a = \frac{k_r^a}{g\rho_a}\frac{\partial}{\partial x_j}(p_a - \rho_a g_k x_k)$$

where k_r^w and k_r^a are the unsaturated hydraulic conductivity of the water and air phases, ρ_w and ρ_a are the water and air density and p_w and p_a are the water and air pressures.

The pore water and air pressures are linked by the capillary pressure, p_c, where $p_c = p_a - p_w$, which is calculated from the effective saturation, S_e, of the material using the van Genuchten [48] soil water retention relationship:

$$p_c = p_{vg}\left[S_e^{-1/m_{vg}} - 1\right]^{1-m_{vg}}$$

where p_{vg} is a fitting parameter which controls the suction at which desaturation begins to occur, and m_{vg} is a fitting parameter which controls the rate of desaturation with increasing suction once air entry has occurred.

In turn, the unsaturated water and air conductivity are a function of the effective saturation and the soil water retention behavior and are calculated as follows:

$$k_r^w = k^w \cdot S_e^{0.5}\left[1 - \left(1 - S_e^{1/m_{vg}}\right)^{m_{vg}}\right]^2$$

$$k_r^a = k^w \frac{\mu_w}{\mu_a} \cdot (1 - S_e)^{0.5}\left[1 - S_e^{1/m_{vg}}\right]^{2m_{vg}}$$

where k^w is the saturated hydraulic conductivity and μ_w and μ_a are the water and air dynamic viscosities respectively.

The models adopted a depth dependent function to derive k^w as follows [41]:

$$k^w = k_{ref}^w \cdot d_s^{k_s^w}$$

where k_{ref}^w is the reference hydraulic conductivity at 1 m depth, d_s is the depth below the ground surface and k_s^w is a fitting parameter that controls the rate of change of k^w with depth. The adopted material properties for the London Clay are summarized in Table 1.

Model geometry, initial and boundary conditions

The 1V in 3.5H, 8 m high cut slope model was discretized using 4-noded quadrilateral elements, with sides 0.5 m in length within the region of interest. Due to the adoption of the 0.5 m elements in the slope, the adopted softening model would require approximately 0.5 m of shear displacement for the London Clay to reach the residual strength. This is compatible with the shear displacements required to reach residual in the ring shear testing on London Clay outlined by [4].

In situ stresses were initialized assuming a coefficient of earth pressure at rest, $K_0 = 1.5$, and a phreatic surface 1 m below the pre-excavation ground surface. This K_0 value was adopted based on prior usage in modelling of natural London Clay [45,41] and was informed by *in situ* measurements [17]. The phreatic surface depth was adopted as a typical value for the UK winter condition ([50]. The initial stress distributions were then used to initialize the pressure dependent stiffness and then the depth dependent permeability function was applied.

The model boundaries at the lateral extents of the model were fixed to prevent vertical displacements and the base of the model fixed to prevent both vertical and horizontal displacements. The lateral and basal boundaries were made impermeable to fluid flow and the slope was then excavated in stages, at a rate of 1 m every 9 days to allow swelling to occur.

Once excavation was complete, a time varying weather driven surface boundary flux was applied to the model. This is described in detail in [41] and the utility of the geotechnical model in the context of weather- and climate-driven deterioration has been demonstrated in [45], [40] and [41]. The model geometry, mesh discretization, boundary conditions and the installed soil nails can be seen in Fig. 1.

These models allow the histories of various properties to be recorded, and that data can be used in the production of deterioration curves, including the change in ultimate limit state factor of safety (FoS, i.e. the ratio of shear strength to shear stress in an earthworks slope) over time as the slope deteriorates towards failure.

The FoS in this work is derived using the strength reduction method (see for example [15], using an automated algorithm where the Mohr-Coulomb shear strength parameters, c' and ϕ', are scaled by a trial value (F_t) and the model stepped. If the model returns to equilibrium, this represents an upper bound on the current FoS. If, after a prescribed number of steps, the model fails to converge to equilibrium, this is assumed to represent a lower bound/unstable FoS. These upper and lower bound values are then used to define new bounding F_t values. This process is repeated until the difference between the lower and upper bound F_t values converges below a prescribed tolerance. The resultant F_t value is adopted as the current FoS.

These FoS curves then allow the relative magnitude of deterioration at various times to be evaluated as well as the time to failure. Comparisons of these curves for different slope geometries, material properties, or, as in the case here, geotechnical interventions at different times, for the purposes of slope renewals, allow the effect on performance and deterioration to be assessed and can be used to evaluate their relative effectiveness and their relative economic costs.

Table 1
Material properties adopted for the London Clay (after [45] and [41]).

Property	Value		
Dry density, γ_d	1550 kg/m^3		
Depth, d_s	<4 m depth	≥4 m depth	
Ref. hyd. Conductivity, k_{ref}^w	1×10^{-8} m/s	1×10^{-9} m/s	
Hyd. Conductivity Exponent, k_s^w	−0.8	−1.0	
Porosity, θ_s	0.45 (-)		
Van Genuchten parameter, p_{vg}	125.0 kPa		
Van Genuchten parameter, m_{vg}	0.153 (-)		
Residual vol. water content, θ_r	0.10 (-)		
Ref. elastic modulus, $E0$	2500 kPa		
Elastic modulus, E'	$E0(p'+100)/100$ kPa		
Poisson's ratio, ν	0.2 (-)		
Nonlocal internal length, l_i	1.0 m		
Strength	**Peak**	**Av. Field Failure**	**Residual**
Friction angle, ϕ'	$\phi'_p = 21°$	$\phi'_{fs} = 13°$	$\phi'_r = 10°$
Cohesion, c'	$c'_p = 7$ kPa	$c'_{fs} = 2$ kPa	$c'_r = 0$ kPa
Nonlocal plastic shear strain, ε^{p*}	$\varepsilon_p^{p*} = 5\%$	$\varepsilon_{fs}^{p*} = 20\%$	$\varepsilon_r^{p*} = 100\%$

Theory and Calculations

Economics of Intervention

Initial earthworks deterioration and intervention modeling for ACHILLES indicated that, in general, the later in the lifecycle of an earthworks asset that a given intervention occurs, the smaller the benefits (in terms of asset life extension) that are obtained. A schematic FoS deterioration curve illustrates this (see Fig. 2), where the lifecycle of a hypothetical, illustrative earthworks asset without intervention is shown by the solid black FoS deterioration curve, and the asset life extension effects of early, intermediate and late interventions are shown respectively by the green, yellow and red dashed curves.

The discounted costs of early, intermediate and late intervention are combined with the respective discounted costs of deferred failure, and the total discounted costs are compared, to determine which

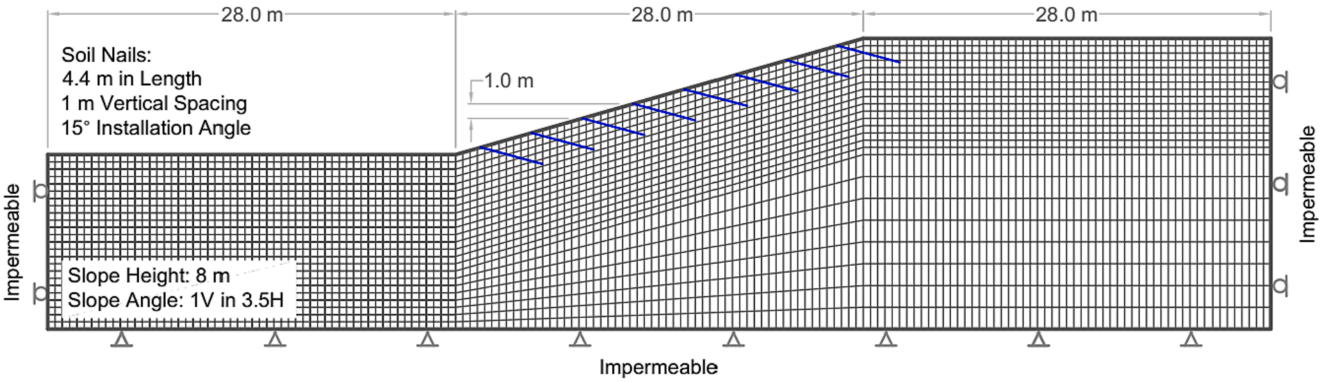

Fig. 1. Geotechnical model geometry, boundary fixities and mesh, including the location of the soil nails.

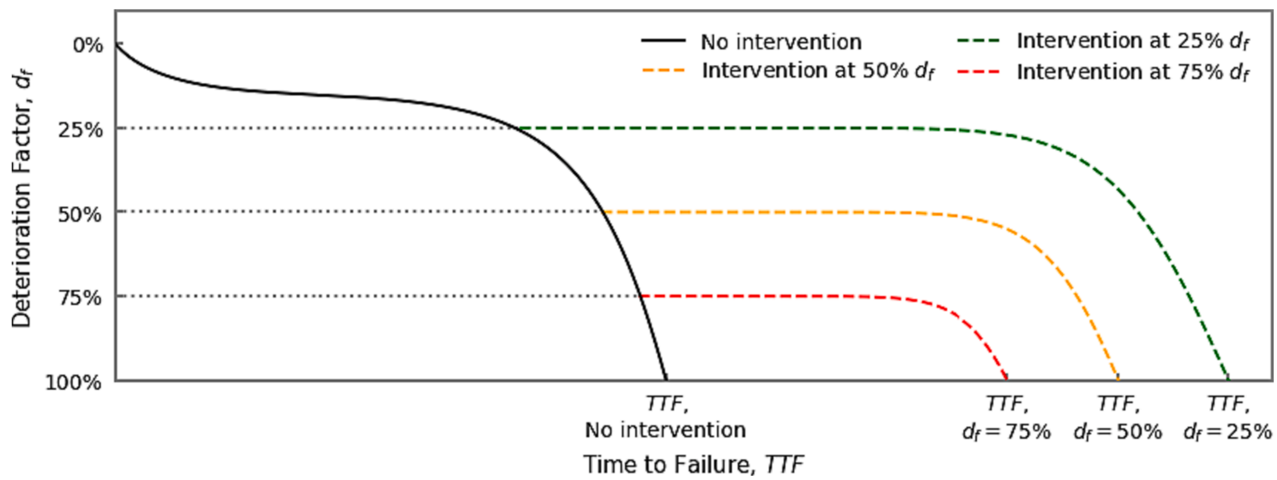

Fig. 2. Alternative Intervention Timings and Lifecycle Extensions.

intervention results in the smallest total cost. Note that it is assumed that 'planned failure' of an asset will not in fact be allowed to happen, and that further interventions will be made as necessary, beyond the time horizon of the current assessment, to maintain the 'indefinite asset life' referred to above.

The assumptions used in the development of this hypothetical example are as follows:

- Current year = 2023 (the base year for discounting purposes)
- Current asset age = 100 years
- Cost of intervention = £1,000,000 (assumed to be constant in undiscounted terms, irrespective of level of deterioration and year of intervention, i.e., at 100, 120 and 140 years)
- Cost of failure and emergency repairs = £10,000,000 (assumed to be 10 times the costs of planned, preventive intervention [14], and again assumed to be constant in undiscounted terms)
- Discount rate = 3%

Because the intervention, failure and assessment timescales vary, the resulting PVC values are not strictly directly comparable, and the PVC values are therefore multiplied by a Capital Recovery Factor (CRF) to produce annualized PVC (APVC) values, which can be directly compared with each other:

$$CRF = d(1+d)^n/((1+d)^n - 1)$$

where d = the discount rate and n = the lifetime of the investment in years.

The results of the 'Do-Minimum' (no intervention) and alternative 'Do-Something' intervention scenarios and calculations are summarized in Table 2. It can be seen that, because of the relatively long timescales for each investment and failure scenario, the CRF values tend to converge to the value of the discount rate (i.e. if n = infinity, CRF = d), and so are quite similar for each scenario.

It can be seen that the intervention at 50% deterioration, in 2043, results in the lowest PVC and APVC values. This is based on a higher discounted failure cost than intervening now, but a much lower one than intervening in 40 years' time at 75% deterioration (or not intervening at all); and on a higher discounted intervention cost than that for 40 years' time, but much lower than, almost half, that of intervening now.

The APVC values are plotted in Fig. 3 against the time to intervention, and differentiating and solving the fitted quadratic curve indicates that, in this hypothetical case, the optimal (in terms of APVC) time to intervention is approximately 17 years, as can be seen from the graph. However, considerable further analysis and validation of the results is required to justify the timing of interventions on this basis.

This hypothetical example was then extended to the detailed geotechnical modelling and economic assessment of a cutting slope, again with no intervention and with the installation of soil nails at different levels of slope deterioration. Such an approach reflects the recommendations by DfT [8] and NIC [33], noted above, to investigate means of slope stabilization and of providing more detailed information on asset condition, with a view to the improved targeting of interventions, based on both geotechnical and economic outcomes. The additional geotechnical modelling of the alternative soil nail installations is described below.

Table 2
Economic Outcomes of Alternative Intervention Timings and Lifecycle Extensions.

Scenario		No Intervention	Intervene Now	Intervene in 20 Years	Intervene in 40 Years
Deterioration at Intervention		100%	25%	50%	75%
Intervention	Year	2023	2023	2043	2063
	Asset Life Extension (years)	0	100	70	30
	Cost	£0	£1,000,000	£1,000,000	£1,000,000
	Discounted Cost	£0	£1,000,000	£553,676	£306,557
Expected Failure	Year	2073	2173	2143	2103
	Cost	£10,000,000	£10,000,000	£10,000,000	£10,000,000
	Discounted Cost	£2,281,071	£118,691	£288,093	£939,771
Present Value of Costs (PVC)		£2,281,071	£1,118,691	**£841,769**	£1,246,328
Capital Recovery Factor (CRF)		0.038865	0.030360	0.030890	0.033112
Annualized PVC (=PVC x CRF)		£88,655	£33,964	**£26,002**	£41,268

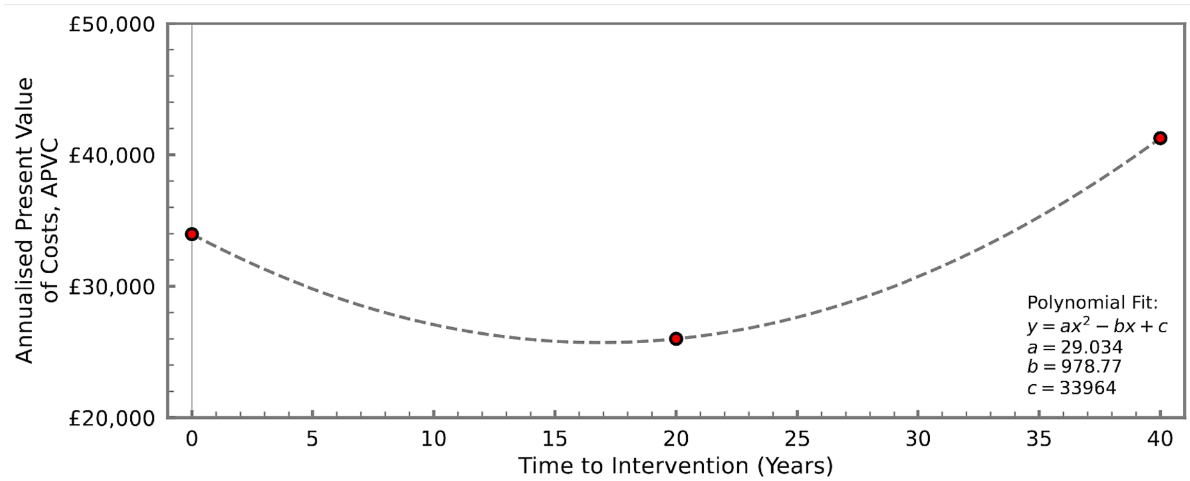

Fig. 3. Annualized present value of costs (APVC) vs. Time to Intervention.

Geotechnical Modeling Calculations

The soil nail model and parameter derivation, and their application to the slope model already described, are summarized in the following sub-sections.

Soil nail model description

Within the numerical framework adopted here, soil nails are treated as structural cables. These behave as one-dimensional axial elements that can be bonded to the model grid so that as the model grid deforms, forces can develop along the cable's length and as such are used to simulate structural support where tensile capacity is important [24].

These reinforcing cable elements representing the soil nails are slender (they have a large aspect ratio) and as such they are assumed to offer resistance along their long axis to axial tension (and compression) but no bending resistance. This tensile resistance is a function of the tensile strength of the cable element, and the shear strength of the cable-grout interface or the grout–soil interface, whichever is weakest. In the case of cable elements adopted as soil nails, this is most often the grout–soil interface shear strength. Note that deterioration of the nail (for example due to corrosion) is not considered in this work.

The cable element representing the soil nail is divided into a number of segments of equal length, L_n, with nodal calculation points, GP_n at each end.

The model describing the elastic axial deformation of the reinforcement tendon is summarized as follows [24]:

$$\Delta F^t = -\frac{E_n^{'} A_n}{L_n} \Delta u^t$$

where ΔF^t is the incremental axial force, $E_n^{'}$, is the Young's modulus of the reinforcement tendon (the soil nail), A_n is the cross-sectional area of the nail and Δu^t is the incremental axial displacement. Limiting tensile and compressive forces can also be specified, which cannot be exceeded.

The shear behavior of the soil-grout interface is modeled as a spring-slider system located at the nodal points in the cable element. The grout–soil shear stiffness (k_{bond}) influences the shear force that develops in the interface, F_s, as follows:

$$\frac{F_s}{L_n} = k_{\text{bond}}(u_n - u_s)$$

where u_n and u_s are the axial displacement of the nail and soil. k_{bond} can be estimated from the shear modulus, G, of the soil surrounding the nail as follows:

$$k_{\text{bond}} = \frac{2\pi G}{10\ln 1 + 2r_t}$$

where r_t is the radius of the nail and grout within the borehole.

The maximum shear force, F_s^{max}, that can be developed at the grout soil interface, per unit length of reinforcement, is a function of the shear strength of the interface between the grout and the soil:

$$\frac{F_s^{max}}{L_n} = s_{\text{bond}} + \sigma_c^{'} \times \tan(s_{\text{fric}}) \times p_g$$

where $\sigma_c^{'}$ is the average effective confining stress acting normally to the element, s_{fric} is the frictional strength of the grout–soil interface, p_g is the perimeter of the grout in contact with the soil and s_{bond} is the apparent cohesive interface strength, derived as follows:

$$s_{\text{bond}} = \pi \times 2r_t \times c^{'}$$

where c' is the adopted apparent cohesive strength of the soil. Note that, in this case, the soil nail strength does not change as the *in situ* soil strength changes. The behavior of the soil nails is shown schematically in Fig. 4.

Soil nail parameter derivation

Soil nail lengths are commonly defined by the ratio of nail length to nail height (the length ratio, L_r). [29] recommends a minimum trial value for L_r of 0.55 for cut slopes in all materials. The lowest values of L_r found in the literature, and adopted in practice, to stabilize slopes in London Clay are 0.625 (for temporary works) and 1.38 for long term stabilization of a 1 in 2.25, 6.7 m cut slope (see [25]). In the modeling work described here, the minimum recommended length ratio for soil nails in cuts of 0.55, as per [29], was adopted for the modeled slope as a conservative assumption. This choice was also influenced by the relatively shallow nature of the failure surface derived from the baseline model with no interventions (see [41]).

The nails were modeled at an angle of 15° below horizontal (as per [39]) and at a vertical spacing of 1.0 m and out-of-plane spacing of 1.5 m as per the recommendations in [51].

For soil nails, the geotechnical model requires the properties of the nails and the properties of the nail-grout–soil interface as discussed in the preceding sub-section. The nail properties include: a) the elastic modulus of the nail (E'_n); b) the nail tensile yield strength (σ_n^t) and c) the nail cross-sectional area. The nail-grout–soil interface properties include: d) the grout axial stiffness (k_{bond}), and e) the shear bond between the grouted nail and the soil (s_{bond}).

The nail tensile yield strength and cross-sectional area were derived from manufacturer specification sheets. The nail-soil shear bond parameter (s_{bond}) typically requires field trial data and in this case was estimated based on the published long-term soil nail pull-out test data from the London Clay cutting described in [25], which provided a direct measurement of $F_s^{\max}$ and also $F_s^{\max}$/unit nail length. As such this value was adopted as the s_{bond} value directly and the frictional component of the pull-out resistance for the nail was not activated in the model. Finally, the nail-grout axial stiffness was derived from the k_{bond} equation above. The adopted soil nail properties are summarized in Table 3.

Soil Nail Installation

The soil nails were installed into the model at the times when the slope had deteriorated by a factor, d_f, of 25%, 50%, 75% and 90%, defined as a percentage of the range from the initial Factor of Safety at time = 0 (FoS_{ini}) and FoS = 1, where the deteriorated FoS (FoS_{d}) is equal to:

$$\text{FoS}_{\text{d}} = \text{FoS}_{\text{ini}} - d_f(\text{FoS}_{\text{ini}} - 1)$$

This was done by loading a saved file from the 'Do Nothing' model at the appropriate time corresponding to a given deterioration factor and then installing the soil nails. The model was then allowed to continue running, being subjected to the same weather boundary conditions as in the 'Do Nothing' model until failure was seen to have occurred. As such,

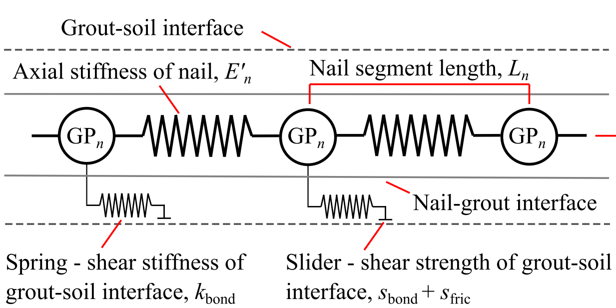

Fig. 4. Conceptual behavior of the soil nails adopted in this work (after [24]).

Table 3
Soil nail properties adopted in the geotechnical model.

Property	Value
Nail elastic modulus, E'_n	200 GPa
Nail tensile yield, σ_n^t	420 MPa
Nail diameter, $2r_n$	25 mm
Nail and grout diameter, $2r_t$	200 mm
Grout axial stiffness, k_{bond}	805 kN/m/m
Grout shear bond strength, s_{bond}	35.5 kN/m

each of the models was identical, except for the magnitude of deterioration due to strain softening of the modelled soil and the dissipation of the construction-induced negative pore pressures at the point in time when the nails were installed.

Results: Modeled Example

The results of the geotechnical modeling are illustrated in Fig. 5 a) and b), with stability expressed in terms of slope FoS. The relative effect of the interventions on serviceability is illustrated in the form of total displacement at the slope toe relative to a reference model, in this case the intervention at d_f of 25%. The relative displacement magnitude is calculated as:

$$u_{mt}, u_{mtb} = \sqrt{u_{mxx}^2 + u_{myy}^2}$$

where u_{mt} and u_{mtb} are the annual maximum total displacement for the slope model and the reference model respectively and u_{mxx} and u_{myy} are the annual maximum horizontal and vertical displacements. The relative annual maximum total displacement, u_r, is derived as follows:

$$u_r = \frac{u_{mt} - u_{mtb}}{u_{mtb}}$$

The associated economic implications are summarized in Fig. 5 c) and the economic calculations are shown in Table 4, based on the same costs and discounting assumptions used to generate the results shown in Table 2.

The modeled geotechnical results are broadly consistent with those of the hypothetical example shown in Fig. 2. In both cases, earlier interventions produce greater asset life extensions from the time of intervention, and also result in greater overall asset life, with the modelled intervention at the 25% deterioration level increasing the overall asset life by 75.5%, compared with the extension of 53.3% obtained by intervening at 90% deterioration.

The serviceability results, in the context of deformation at the slope toe, show a similar trend to those for stability. Later interventions were found to produce progressively smaller reductions in deformation relative to the baseline intervention at 25% deterioration. The intervention models at 75% and 95% are shown to produce negligible improvements in serviceability relative to the 'Do Nothing' model, where failure was allowed to occur without intervention.

The economic assessment of the modelled results, equivalent to the hypothetical example shown in Table 2, is summarized in Table 4, whose five results columns correspond to no intervention ('Do Nothing'), and to the four soil nail installation timings illustrated in Fig. 5. As in Table 2, it can be seen that the higher discounted costs of earlier interventions are offset by the lower discounted costs of later failures, and that the minimum combined PVC and APVC values (£238,362 and £7,263 respectively) correspond to a 'late intermediate' intervention in 84 years' time (vs. 120 years into the life of the hypothetical asset in Table 2), when the model predicts 75% deterioration of the asset. Differentiation and solution of the APVC curve fitted to the four intervention points indicates a minimum APVC value at approximately 81 years (vs. 117 years in the hypothetical example), when the deterioration is slightly less than at 84 years. This information supports

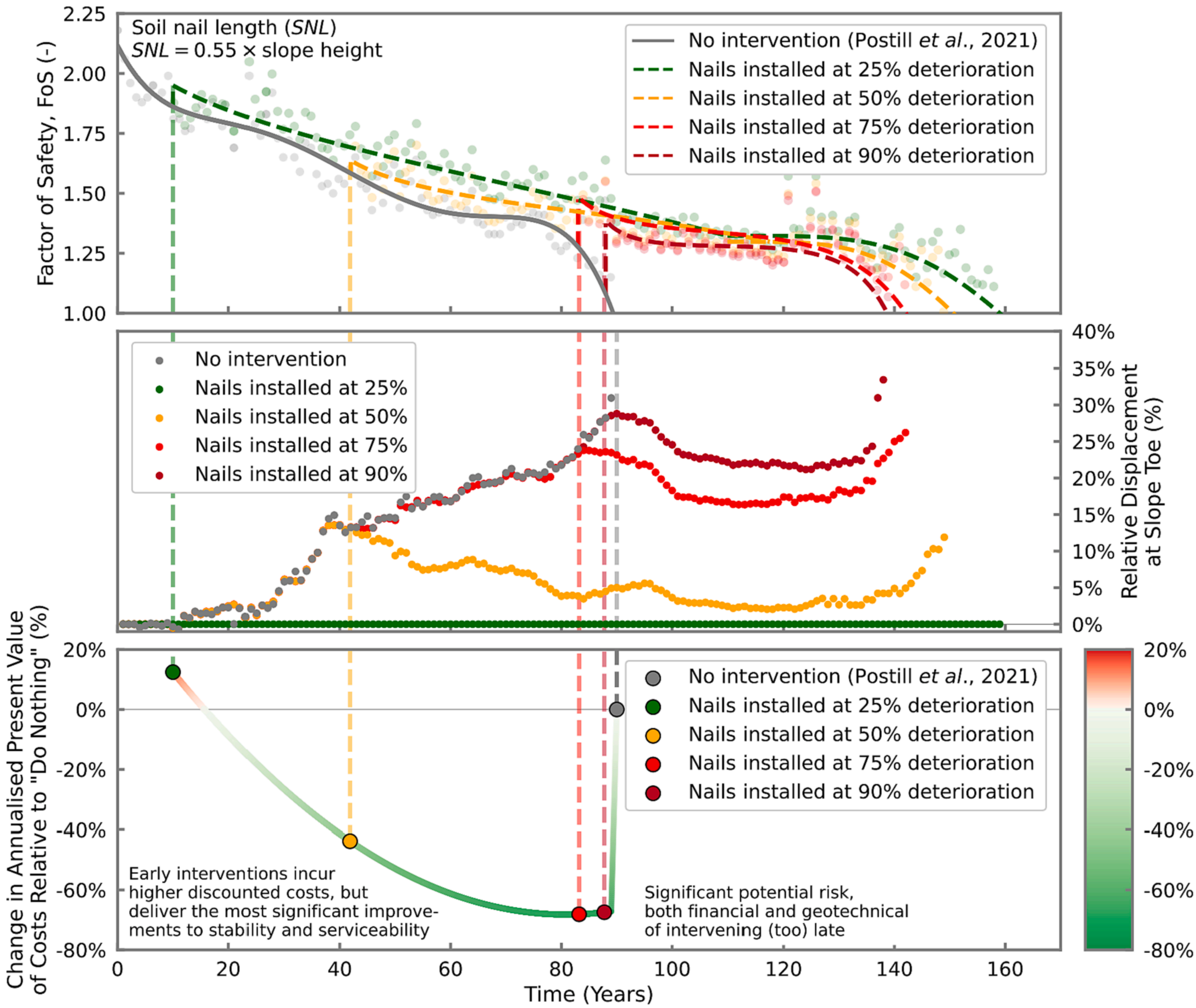

Fig. 5. a) asset life extensions with alternative soil nail intervention timings, b) the relative deformations at the toe, and c) costs relative to the 'do nothing' approach. The vertical dashed lines represent the times interventions were performed.

Table 4
Economic Outcomes of Alternative Intervention Timings and Lifecycle Extensions.

Scenario		No Intervention	Intervene in 10 Years	Intervene in 43 Years	Intervene in 84 Years	Intervene in 89 Years
Deterioration at Intervention		100%	25%	50%	75%	90%
Intervention	Year	2023	2033	2066	2107	2112
	Asset Life Extension (years)	0	68	58	51	48
	Cost	£0	£1,000,000	£1,000,000	£1,000,000	£1,000,000
	Discounted Cost	£0	£744,094	£280,543	£83,497	£72,026
Expected Failure	Year	2113	2181	2171	2164	2161
	Cost	£10,000,000	£10,000,000	£10,000,000	£10,000,000	£10,000,000
	Discounted Cost	£699,278	£93,695	£125,919	£154,864	£169,224
Present Value of Costs (PVC)		£699,278	£837,789	£406,462	**£238,362**	£241,250
Capital Recovery Factor (CRF)		0.032256	0.030284	0.030383	0.030472	0.030516
Annualised PVC (=PVC x CRF)		£22,556	£25,371	£12,349	**£7,263**	£7,362

the intervention decision-making process, but in practice, and as discussed below, asset safety and serviceability also need to be taken into consideration. Given the uncertainties inherent in asset condition assessment, the closer the FoS gets to 1, the greater the risk of failure (with its higher PVC and APVC) at any given time. This may make intervention prior to 75% deterioration desirable, assuming the level of deterioration is actually apparent.

Discussion

The results demonstrate that there are clear advantages of early intervention in terms of asset life extension from the time of intervention, overall asset life and also in terms of increased levels of maintained asset serviceability and safety. However, they also illustrate the economic and financial attractions of deferring intervention, particularly in (perhaps typical) cases of financial and resource scarcity. To ensure that asset safety and serviceability are taken into account in the decision-making process and associated decision support tools (and to avoid the impacts and much higher costs of having to repair a failed asset), such analysis and 'optioneering' would have to be constrained in practice by the need to maintain minimum acceptable levels of both safety and serviceability.

The results shown in Table 2 and Table 4 are based on deterministic values of times to failure (TTFs), defined as when the FoS value reaches 1. The exact time to failure will be sensitive to the adopted strength and strain softening properties, as well as to the rate at which construction-induced pore pressures dissipate and to the seasonal pore pressure cycle size, in turn a function of the applied weather boundary conditions. In practice, actual time to failure values, even when produced by detailed and computationally intensive modeling techniques, are subject to considerable uncertainty due to the uncertainty in the adopted model parameters. While timely interventions are essential to ensure continuing asset safety and serviceability, earlier-than-necessary intervention incurs increased discounted costs and wastes serviceable asset life and intervention resources that could be more beneficially otherwise deployed. A probabilistic approach is likely to be a useful means of dealing with these uncertainties. This is based on the increasing cumulative probability over time of asset failure without intervention, and considering and comparing the expected costs of intervention and failure over an asset's lifecycle, with the aim of intervening before the expected costs of failure exceed those of intervention, while also maintaining acceptable levels of safety and serviceability. The situation is further complicated by additional uncertainty about future weather conditions, and the need to model alternative climate scenarios if/when looking more than 40–50 years ahead.

The examples shown and the discussion above, including the handling of uncertainty, relate to single assets only. While it is relatively straightforward in principle to identify the most economically advantageous timing of interventions for individual assets, even when allowing for uncertainty, most infrastructure managers (IMs), particularly in the railway and other transport sectors, are responsible for portfolios of multiple earthworks assets. For example, Network Rail, the IM of most of Britain's railway network, is responsible for over 190,000 individual earthworks, with a total length of approximately 19,000 km, as noted above [31]. Monitoring and maintaining this collection of assets is a major organizational challenge (emphasizing the potential value of effective decision support in this area), and maximizing the extent of asset condition improvement within a given budget, as Network Rail aims to do, is still more so. This latter objective has similarities with the 'knapsack problem' in Operations Research, aiming to maximize the utility of a constrained subset of options chosen from a larger set (or accommodated within a given size of 'knapsack'). This was previously applied to railway maintenance funding by [28], and an integer programming-based approach to the problem is being considered, with constraints including the avoidance of failure and the maintenance of minimum required serviceability standards. However, such a rigorously mathematical approach to maximizing overall asset condition improvement may not meet the actual needs of Network Rail and other IMs. In many cases, rather than seeking to optimize the timings of individual asset interventions, IMs are likely to take a more pragmatic approach, taking advantage of line closures to work simultaneously on adjacent assets in a single location or area, especially if the optimal intervention timings for those individual assets are reasonably similar. Such approaches would also help to minimize both the mobilization and disruption costs for each individual intervention. These wider issues are also being addressed by ongoing and further work in the ACHILLES research program. The examples cited and shown in this article are primarily UK- and railway-focused (although the modeled slope geometry is also representative of highways slopes), but the techniques and approaches described should also be more widely and generally applicable.

Conclusions

The operation, maintenance and renewal of an extensive portfolio of earthworks assets in a cost-effective manner, while also maintaining acceptable levels of safety, serviceability and availability for use, is a significant technical and organizational challenge. This challenge is increased by the inherently uncertain behavior of ageing earthworks, complicated further by the effects of climate change. The ACHILLES research program is providing improved understanding and modeling of earthworks behavior and performance, and is developing decision support methods and tools for earthworks management.

This article sets out a proposed approach to the provision of decision support for the improved scheduling of remedial interventions for individual earthworks assets. It contrasts the economic assessment of like-for-like infrastructure renewals with the more conventional approach to the assessment of new infrastructure, and proposes an amended methodology, based on the comparison of the discounted whole-life costs of alternative intervention options.

The article presents hypothetical and modeled examples of the economic and serviceability costs and benefits associated with interventions at different stages in the lifecycle of an earthworks asset, relative to a 'Do Nothing', non-intervention approach. These analyses demonstrate the life-extension and serviceability benefits of early intervention, the potential economic attractiveness of later interventions, and the need to reconcile these conflicting priorities, minimizing the discounted total costs of intervention while maintaining acceptable standards of asset serviceability and safety.

Finally, an emerging approach to the handling of uncertainty in the assessment and decision support processes is outlined, and consideration is given to how the single-asset approach may be extended to the collective coverage of multiple earthworks on a route or network, within the constraints of an available budget.

The approaches and methods described in this article update and combine aspects of geotechnical modelling and economic assessment in a novel manner. The initial hypothesis in relation to the least-whole-life-cost timing of interventions is supported by the results of the detailed geotechnical modelling of alternative soil nail intervention timings. While the methods developed are based upon modelled outputs and a set of economic assumptions, they complement and potentially enhance the decision-support tools and systems available to Network Rail and other IMs. They thus offer the prospect of extending the asset management 'toolbox' available to IMs to address the challenge of maintaining ageing earthworks in a cost-effective manner in the face of the increasing impacts of climate change.

CRediT authorship contribution statement

John Armstrong: Formal analysis, Investigation, Methodology, Software, Writing – original draft, Writing – review & editing. **Peter Helm:** Formal analysis, Investigation, Methodology, Software, Writing – original draft, Writing – review & editing. **John Preston:** Conceptualization, Project administration, Supervision, Writing – review & editing. **Fleur Loveridge:** Conceptualization, Project administration, Supervision, 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.

Acknowledgements

Funding: The work described is funded by the United Kingdom's Engineering and Physical Sciences Research Council (EPSRC; ref. EP/R034575/1). The authors are also grateful for the input and support provided by colleagues and collaborators across the research program, notably at Network Rail and Mott MacDonald.

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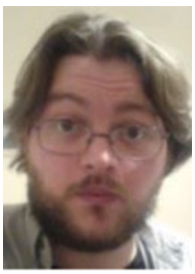
John Armstrong John Armstrong is a Senior Research Fellow at the University of Southampton's Transportation Research Group. He works primarily in railway operations planning and analysis, and in engineering economics.



Peter Helm Peter Helm is a Research Associate at Newcastle University. His background is in geotechnical engineering and modeling and his current work is focused on the modeling of geotechnical infrastructure deterioration, subject to annual weather cycles.

John Preston John Preston is Professor of Rail Transport at the University of Southampton. His work in rail is focused on demand forecasting, cost estimation and operations planning.

Fleur Loveridge Fleur Loveridge is a Professor in the School of Civil Engineering at the University of Leeds. She works in the field of sustainable and resilient geotechnical infrastructure, covering the topics of energy harvesting and climate change impacts.