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Multiple hazard fragility analysis for granular highway embankments: moisture ingress and scour

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Abstract

Fragility functions express the probability that an asset exceeds some serviceability or limit state for a given level of environmental perturbation or other loadings, to which the asset is subjected. They are important components in the quantitative risk analysis of infrastructure exposed to natural hazards and they have typically been derived for structural assets. It is relatively difficult to derive fragility functions for geotechnical assets, such as highway or railway slopes and embankments, due to their inherent heterogeneity. In this paper, a generic granular highway embankment is modelled using the finite element method, considering various groundwater profiles and scour depths at the toe to quantify the deformation of the road surface. A probabilistic assessment of the magnitude of deformation and the groundwater level and scour depth is undertaken to derive fragility functions for the prediction of damage to assets exposed to these multiple hazards. The process of fragility function derivation is explained, uncertainty values are derived, and various regression methods are undertaken. This study is a first attempt to provide a basis for the prediction of slope deformation, and hence of damage, due to moisture ingress and scour, which can be aggravated by climate change. This can be used for the assessment of existing assets, and the design of new ones in the pursuit of more resilient transport networks, as well as for other assets such as levees, dams and other similar earthworks, with some limitations.

Keywords: transport infrastructure; embankments; fragility and vulnerability functions; moisture ingress; scouring; multiple hazards

Introduction

Extreme weather events are becoming increasingly frequent and severe. In the UK, the occurrence of high-intensity rainfall events and flooding is increasing (Environment Agency, 2018). Severe flooding in the winter of 2013-14 resulted in costs of £1.3bn (Department for Environment Food and Rural Affairs, 2016), which was exceeded after the 2015-16 flooding events (Marsh et al. 2016). During the 2013-14 floods, extensive damage to rail lines across the southwest was observed (Thorne 2014), causing severance of access on these routes. Large scale flooding across Bosnia and Herzegovina in 2014, damaged 1262km of roads with a repair cost of about €257m. This included natural landslide and damage to slopes and embankments, the washing away of carriageways, damage to bridge structures, tunnels and drainage systems, and the destruction of traffic signs and equipment, as well as private properties (Reeves et al. 2019a; Winter et al. 2020). Transport networks are heavily dependent on road and rail, both including linear assets consisting of embankments and cuttings as well as bridges, retaining walls and tunnels. The increased severity of rainfall and the occurrence of flooding due to climate change exacerbates the risk of instability and failure of transport assets. The age and dispersed nature of the transport infrastructure mean that many railways and roads were built before adoption of modern design standards or the development of geotechnical slope stability theory. Even where recent standards have been adopted, climate predictions and the environment within which the slopes exist is changing and this has commonly not been taken into account during the initial design. Therefore, it is of paramount importance to consider the environmental, weather and climate change effects, not only in the design and construction of new assets, but also for the maintenance and preservation of the existing road and rail networks to improve their resilience (FHWA 2012, Reeves et al. 2019b). In this respect, insurance companies, infrastructure owners and operators need advanced approaches to reliably and rapidly assess the performance and damageability of the different assets, providing a robust basis for the prioritisation of mitigation actions and asset management. Hence, the quantification of transport asset resilience is critical in maintaining the continuity of services and safety for the millions of users per day, considering the finite funding and practical timeframes (Argyroudis et al. 2020).

Risk-based management approaches are widely applied by transport infrastructure stakeholders to prioritise the assets that require mitigation measures based on Quantitative Risk Analysis (QRA) (FHWA 2013). QRA assists in estimating the potential economic, functional and social losses in a specific period, determined probabilistically, as a function of hazard, exposure and vulnerability (Winter and Wong, 2020). Vulnerability refers to the propensity of the transport

infrastructure assets to suffer adverse effects when impacted by hazard events and external factors (UNISDR 2009). Fragility functions provide a mechanism to quantify vulnerability and losses (see Figure 1), specifically defining the probability of exceeding pre-determined damage states or performance criteria, e.g. settlement of the road surface, for a given intensity measure, e.g. rainfall intensity, water depth for hydrological hazards or peak ground acceleration for earthquakes (Argyroudis et al. 2019). The usefulness of fragility functions is dependent on the ease of definition of the intensity measure and the generalness of the derived fragility functions for use in different situations. The main methods to assess vulnerability are classified as (i) judgemental/heuristic, (ii) data-driven, using data from past events, and (iii) analytical, using numerical models (Corominas et al. 2014).

The derivation of fragility functions has typically been focussed on structural assets, such as bridges, tunnels and retaining walls subjected mainly to seismic shaking effects and in some cases to combined hazards (Tsiaousi et al. 2018, Argyroudis et al. 2019). The vast majority of the literature has not investigated the soil-structure interaction (SSI) in depth. For bridges, the soil is typically modelled through equivalent springs at the abutment and pier foundations (Stefanidou and Kappos 2017), however, the SSI effects can be significant at the critical damage states (Stefanidou et al. 2017). For tunnels or retaining walls, the SSI effects are taken into account by numerical models that include both the structure and the surrounding soil using more generic soil models (Huang et al. 2020). Considering this aim of the broad applicability of fragility functions and the site-specific variability of soil materials, unsurprisingly, soils have been greatly generalised in the assessments undertaken.

Landslides triggered by rainfall or earthquakes, including sliding, debris flow, rockfall or mudflow can induce severe damage to transport assets such as embankments, slopes or tunnels (Winter 2014) and therefore, comprehensive vulnerability and risk assessment of these assets is an urgent need in the transport sector. Lee and Jones (2014) provide an overall introduction to landslide risk assessment, while Corominas et al. (2014) give an overview of quantitative risk analysis for landslides, providing a summary of the characteristics that control instability and also a discussion on vulnerability quantification. Although more centric to landslides, debris flows and rockfall, the characteristics are relevant for all slope stability assessments (Krahn, 2012). Winter et al. (2014) provided fragility functions for roads subjected to debris flows based on an expert judgement approach. Generic fragility functions for rainfall-triggered landslides were proposed by Martinović et al. (2016) for a range of slope angles. Maruyama et al. (2010)

provided fragility curves for expressway embankments, based on seismic damage data in Japan, considering cracks and vertical separation in the roadway. Fragility functions for railway embankment and track ballast scour were developed by Tsubaki et al. (2016) using damage observations in Japan and simulated overtopping water depth. Deck and Verdel (2012) presented a methodology for deriving fragility models, where previous events or statistical data are not available, using Monte Carlo simulations. Damage is assessed for a set of theoretical assets representative of a particular type, taking into account the variability in the type and the associated uncertainties. Argyroudis and Kaynia (2015) proposed fragility functions for embankments, cuttings and bridge abutments subjected to seismic loading, based on numerical simulations of representative typologies. The damage was classified as minor, moderate and extensive/complete, based on the range of vertical displacements for highways and railways.

Recent trends in the literature show a preference for the assessment of multiple hazards in the derivation of fragility functions (Yilmaz et al. 2016, Gidaris et al. 2017, Mitoulis et al. 2019). A multiple hazard or multi-criteria risk analysis is advantageous as it is more detailed and allows the criteria to be weighted to quantify their relative importance (Deck and Verdel, 2012, Clarke and O'Brien 2016). In this respect, Argyroudis et al. (2019) presented a 'fragility surface' modelling for critical transport assets exposed to multiple hazards to allow the representation of two hazard intensity measures, for example, flood and earthquake, along with damage exceedance probability. Also, Gehl and D'Ayala (2016) applied Bayesian Networks for the multi-hazard fragility assessment of bridge systems.

In numerical models, where soils have been directly assessed, typically they have been modelled with total stresses and undrained parameters (Argyroudis et al. 2019). This is appropriate where rapid loading, e.g. seismic loads, occurs or cohesive soils are known to exist. To account for flooding, an undrained shear strength profile dependent on vertical effective stress (σ'_v) has been used by Argyroudis et al. (2018). To the authors' knowledge drained parameters, i.e. effective stress assessment, have not been used in the derivation of fragility functions. The adoption of effective stress explicitly controls the strength profile based on the saturation of the soil materials (Terzaghi 1925). Furthermore, to date, no numerical fragility assessment of slopes/embankments that do not contain structural elements and exposed to flood effects has been undertaken considering soil deformations. This could be due to the preference for limit equilibrium assessment of slope stability within geotechnical practitioners, consistent with the methodologies presented by Janbu (1954), Bishop (1955) and Morgenstern and Price (1965). Limit equilibrium

assessment is a binary approach, i.e. stable/unstable, and it is useful for deriving geotechnical parameters of existing slopes or ensuring new slopes meet the minimum requirements as per current design standards. However, limit equilibrium is a strength-based assessment and does not predict deformations for comparison with performance criteria, and therefore, cannot be used to quantify slope damage and derive fragility functions.

This study aims to derive fragility functions that can be used to predict the displacement, and hence the damage, of a carriageway on a granular embankment that is subjected to flooding. The fragility assessment is based on numerical simulations considering drained parameters of the soil materials, for a gradually increased level of the water table, followed by scour hole development at the toe of the embankment slope. The study has focussed on road embankments but could be readily adapted for rail by changing the damage thresholds. This represents, to the authors' knowledge, the first attempt to derive fragility models for granular soils with drained parameters subjected to multiple hazard effects. It is envisaged that the fragility functions derived can be used within a risk quantification framework for existing assets to prioritise remedial works and improve the resilience of independent and interdependent assets and systems of assets to impending climate changes; specifically, increased frequency of high-intensity rainfall and flooding events. The fragility functions can also be part of the design process as they provide information for the performance of the asset under diverse hazards considering different design levels of hazard magnitudes and frequencies.

Methodology

Fragility functions or curves are usually described through a cumulative lognormal distribution (Φ) for each of a number of damage states, described by a median value of the hazard intensity measure (IM_{mi}) that is required to exceed the particular damage state (DS_i), and a logarithmic standard deviation (β_{tot}) (Argyroudis et al. 2019). They provide the probability of exceeding each DS_i for a given intensity measure, IM as follows:

$$P_f(DS \geq DS_i | IM) = \Phi\left[\frac{1}{\beta_{tot}} \ln\left(\frac{IM}{IM_{mi}}\right)\right] \quad (1)$$

Figure 1 shows how for a given IM (step 1), we can calculate the probability of reaching or exceeding a damage state (step 2) and the probability of being in a damage state (step 3) through fragility curves. The damage states are commonly related with repair ratios, i.e. ratio of cost of repair to the cost of replacement (step 4). The loss ratio for the given IM is computed as the sum

of the products of the probability of being in each damage state and the corresponding repair ratios (step 5). The direct loss for the given IM is then calculated by multiplying the loss ratio with the construction or replacement cost of the asset (step 6). In this example, indicative fragility curves for minor, moderate and extensive damage are presented, along with representative repair ratios and replacement cost for a highway embankment. If we calculate the loss ratios for all the IMs, then the corresponding vulnerability curve can be plotted, which correlates the loss ratio (vertical axis), with the IM (horizontal axis). In this way, it is possible to quantify, in monetary terms, the direct losses for different hazard scenarios.

Figure 1. Example of using fragility functions for a given intensity measure (IM) (step 1) to calculate damage probabilities (steps 2 and 3) and direct losses (steps 5 to 6) for representative repair ratios (step 4) for minor, moderate and extensive damage.

This study initially produces fragility functions for granular highway embankments subjected to flooding or significant moisture ingress. Subsequently, a multiple hazard assessment was undertaken to assess the influence of scour as well as the varying groundwater level. The Intensity Measure (IM) describing the flood hazard effect was defined as the proportion of the embankment height that would be considered saturated (floodwater IM or WIM, see equation 2). The IM describing scour was defined as the depth of that scour (scour IM). When assessing the multiple hazard effects, the two IMs were combined, i.e. floodwater and scour. It is noted that the use of proportion for the embankment saturation and depth for the scour should be repeated when adopting the fragility curves derived. The main steps to derive fragility functions based on numerical modelling are the following I to V:

- I. Derive a model including the slope/embankment and foundation soil and the imposed hazard effects, i.e. water table and scour due to flood, and perform numerical analysis for increased hazard intensities (*see below, Numerical model*).
- II. Define damage states based on the range of damage measure values (*see below, Damage states*).
- III. Plot the intensity measure, e.g. height of flood above the base of the embankment or scour depth, versus corresponding damage measure, e.g. ground deformation, and derive best-fit regression. Use best-fit regression to derive median threshold intensity measure (IM_{mi}) at each damage state (*see below, Relationship between IM and DM & Numerical modelling challenges*).
- IV. Calculate component uncertainty values and combine to calculate total uncertainty (β_{tot}) values (*see Uncertainties*).
- V. Calculate the lognormal cumulative distribution function (equation 1) using the median values of step III and the standard deviation (β_{tot} in step IV) to derive the fragility curves (*see below, Fragility functions for moisture ingress & Fragility functions for combined moisture ingress and scour effects*).

Numerical model

A two-dimensional finite element model was built in the proprietary software Plaxis 2D (Plaxis, 2019). A plane strain model with 15-noded elements was used allowing a stiffness-based assessment to be undertaken. A 2D assessment in-plane strain was considered acceptable considering the linear nature of the assets to be assessed. The default iteration settings were used and a fine mesh with additional mesh refinement was generated at the toes of the embankment slopes (Figure 2). Plaxis runs a staged assessment, where progressing to the subsequent stage is contingent on completion of the previous stage. When modelling displacement, the software recognises that soils can yield or ‘collapse’. If this occurs the stage is aborted, and subsequent stages cannot be computed.

A representative embankment with a simplified geometry was selected, having a symmetrical 10m high profile, and a crest width of 13m, including two traffic lanes of 3.5m each, with 20kPa uniformly distributed load and 3m exclusion zone from the crest to the road on each side (Figure 2a). A typically adopted and prevalent slope angle 1(V):2(H) with no benches was considered,

which is generally assumed to be 'safe' in the long term for permanent works design. A single soil unit for the embankment and a single soil unit for the foundation stratum was adopted to make the findings of the assessment suitably generic that interfaces between different foundation materials would not affect the assessment. The lateral boundaries of the model were at 30m from the embankment toe; therefore, the width of the model was 113m. The basal boundary was twice the height of the embankment below the toe. Fixities were modelled at the boundaries as per the default conditions in Plaxis. The adopted geotechnical parameters are shown in Table 1. The granular characteristic parameters, i.e. effective angle of friction, ϕ' , effective cohesion, c' , were adopted from BS8002:2015 (2015), while a typical unit weight, γ , for a granular material was adopted from Bowles (1979). Stiffness parameters were assumed for normally consolidated granular soils based on the authors' experience and accepted practices from industry. The Mohr-Coulomb criterion was adopted for both soil units to account for soil non-linearities. It is noted that, as per common practice, during the analyses, displacements were reset to zero following the initial phases, i.e. after building the embankment, applying gravity load and the surcharges, while resets were not undertaken for subsequent stages, i.e. raising of the groundwater to the assessed saturation level and scouring steps.

A pure granular soil, as modelled in the embankment soil, is not common for embankment construction. It is more probable that a mixture of cohesive and granular components will form part of the fill material; this will be through a combination of availability, cost, the practicality of construction and issues surrounding sustainability (Gomes Correia et al. 2016). The closest material to a pure granular material would be a rock fill. This would have much higher stiffness than that modelled, and the formation of soil behaviour would probably control the behaviour of the embankment during any flooding event. The assessment has necessarily been generic and two homogenous soil blocks have been modelled. The variability within the soils (especially any cohesion) and compaction will change the performance in the saturated condition (Tatsuoka and Correia 2018), and this could lead to piping, dissolution, or other void formers that would create localised high displacement areas.

(a)

(b)

(c)

Figure 2. (a) Generated mesh for the 2D model in Plaxis. (b) Groundwater depths investigated from 1m depth to 7m depth in 1m increments (greyed outlines). The groundwater profiles show that the width of saturation is the full crest width (0m offset from the crest). (c) Close up of the groundwater profiles showing that the width of saturation is laterally offset from the crest by 3m (1m grid squares shown for scale).

Table 1. Geotechnical parameters used in the assessment.

soil unit	ϕ' [°]	c' [kPa]	γ [kN/m ³]	E' [MPa]
embankment fill	30	0	18	10
foundation soil	32	1.5	18	32

The model was first set up with a gravity loading stage with a low groundwater table to simulate the history and stress changes due to the construction of the embankment. Subsequently, the surcharge was applied. These initial stages formed the starting condition for all subsequent assessments. Groundwater behaviour was set as seepage on the left and closed at the base and the right. A symmetric groundwater profile was modelled in all assessments. The groundwater profile was investigated at seven different depths, i.e. 1m to 7m from the top of the embankment and with different widths of saturation within the embankment (Figure 2c). The width of saturation was symmetric and defined as the lateral offset from the crest for the maximum groundwater level; in Figure 2b there is no offset from the crest (13m saturated width), in Figure 2c there is a 3m offset from the crest, i.e. 7m symmetric saturated width. A total of seven different offsets (0m to 6m) were modelled equating from 13m to 1m saturated crest width. The width of saturation was used to provide seven data points for each groundwater profile depth. The latter was used to derive the proportion of the embankment that was saturated (WIM), i.e. normalised water table, as per equation 2.

$$WIM = \frac{h_{emb} - h_w}{h_{emb}} \quad (2)$$

where WIM is the Water Intensity Measure, h_{emb} is the height of embankment, and h_w is the depth to the groundwater table. The scour assessment was undertaken by adding a final stage where a volume of material was removed at the toe of the slope representing the consequence of a flash flood or river flow at the toe of the slope, whilst the raised groundwater level was maintained. A consequence of this staged approach is that if the preceding stage fails then further stages cannot be run. This means that if the soil model collapsed with no scour then scour could not be assessed for this soil model. The geometry of the scour was based on a 45° cut into the foundation soil at the toe of the embankment, a 3m base width and a 45° cut into the foundation on the opposite side. The depth of scour was modelled from 0m to 3.0m depth in 0.5m intervals (Figure 2b). It is noted that the numerical model adopted is 2D and in-plane strain, therefore the scour is assumed to persist along the full length of the embankment. This is a limitation of the numerical model and is unlikely to be accurate but provides a conservative estimate (see also the section ‘Discussion’).

Damage states

Damage states should correspond to defined performance criteria for an asset. Once a damage state is reached there are specific outcomes or expectations, described with either qualitative and/or quantitative criteria, as to what should follow concerning future performance and/or restoration measures. Current practices in assessing slope risk to embankments include ‘vulnerability ratings’, which are considered synonymous with damage states for embankment failure (e.g. Stewart et al. 2002). These ratings are generally qualitative or semi-quantitative and based on the risk to life, dependent on the vehicle speed. To derive fragility functions, it is necessary to have fully defined quantitative damage states, based on a variable or Damage Measure (DM) that is measurable in the numerical model. The DM in this study was defined as the maximum vertical displacement (u_y) at the road surface, and the damage states used in the assessment were adopted from Argyroudis and Kaynia (2015) and replicated in Table 2. In particular, for the assessment of the DM excluding scour, the maximum u_y occurs at the same location due to the symmetry of the model. However, the scour modelling was asymmetric and as a consequence, the location of the maximum u_y is located at a different position along the road surface, and nearer the scour hole

Table 2. Damage states adopted in the assessment (after Argyroudis and Kaynia, 2015).

damage state	u_y [m]			qualitative descriptor
	minimum	maximum	mean	
minor	0.02	0.08	0.05	open, reduced speeds or partially closed during repair
moderate	0.08	0.22	0.15	closed or partially closed during repair works
extensive/complete	0.22	0.58	0.40	closed during repair works

Relationship between IM and DM

The numerical model was used to derive the vertical displacement at the road surface, which represents the DM, for each WIM at each saturation width. The natural logarithm (Ln) of the WIM and DM (u_y) were calculated to smooth the data and increase the ease of fitting a best fit line, following established practices in fragility analysis (Cornell et al. 2002). In particular, a graph of $\text{Ln}(\text{WIM})$ and $\text{Ln}(u_y)$ was plotted to establish linear ($\text{DM} = a \cdot \text{WIM} + b$) and polynomial ($\text{DM} = a \cdot \text{WIM}^2 + b \cdot \text{WIM} + c$) lines of best fit using trendline functions,

with a , b and c being constants of the regression. Using these regression lines is possible to predict the DM for any WIM, and to calculate the WIM that corresponds to the mean DM (Table 2) for each damage state (DS). This WIM value becomes the median (IM_{mi}) for the lognormal distribution fragility curve for each DS (Equation 1).

Uncertainties

Uncertainty values allow the variability within the numerical model and the fragility analysis to be quantified and in this case, the uncertainties are expressed by the logarithmic standard deviation (β_{tot}) of the fragility function (Equation 1). The uncertainties are grouped into three key components; capacity (β_C), demand (β_D) and limit state (β_{LS}).

The β_C is associated to the uncertainty within the properties of the soil and the geometry of the structure within the numerical model and represents the variation in the capacity of the soil-structure system due to the uncertainty in the parameters adopted. Commonly used values for β_C is 0.3 to 0.35 for tunnels and bridges, i.e. concrete and steel engineered structures (Argyroudis et al. 2019). The uncertainty in the capacity of soil embankments has not been previously investigated. In this respect, a relationship based on the partial factors of safety (γ_M) from Eurocodes 2, 3 and 7 [EN 1992-1-1 (2004), EN 1993-1-1 (2005), EN 1997-1 (2004)] has been adopted. These factors are specifically to reduce the characteristic values to design values, to provide robustness by dealing appropriately with the arithmetic expression of uncertainty within the design assessment. The partial factors in Eurocodes are based on the certainty of the parameters of the material. For example, the capacity of steel is well known, due to the controlled environment in which it is forged and the strict controls on its dimensions, therefore a partial factor of 1.15 is considered appropriate to derive the design strength. For soil materials there is more variability and heterogeneity, less control on genesis and source and increased partial factors are therefore adopted. In Eurocodes, a partial factor of 1.25 is adopted for the tangent of effective angle of friction (ϕ') and the intercept cohesion (c), whereas a partial factor of safety equal to 1.4 has to be applied to the undrained shear strength (c_u). On this basis, the partial factors are considered to be comparable with a lower range of uncertainty value, i.e. 95% probability of exceedance. The literature indicates that a β_C equal to 0.30 is generally assumed for structural elements. Assuming a linear relationship, a β_C value of 0.37 could be assumed for the soil.

The parameter β_D relates to the variability within the demand or loading history placed on the system, which in this case corresponds to the selection of the water intensity measure and the

scour depth. Numerous analyses have been performed to reduce the uncertainty in performance, based on the groundwater profile and scour extents. However, uncertainty remains in the actual water profile that may exist or the geometry and location of any scour. These uncertainties are almost impossible to decrease without exhaustive modelling. The literature indicates that values between 0.4 and 0.7 are typically adopted for seismic demand, with 0.5 recommended in Argyroudis et al. (2019). In lieu of any specific advice in the literature for embankments subjected to flooding, the 0.5 recommendation has been adopted.

The β_{LS} is associated to the uncertainty within the definition of the damage states, and a value of 0.25 is adopted based on engineering judgment and recommendation in the literature for other transport assets (Stefanidou and Kappos 2017). The total uncertainty value (β_{tot}) was calculated based on the square root sum squared (SRSS) of each of the uncertainty values assuming that each component is probabilistically independent. Therefore, β_{tot} was estimated equal to 0.67.

Results for moisture ingress and scour effects

The numerical model outlined was used to undertake 49 different analyses for the non-scour assessment, including the combinations of the seven different groundwater depths and the seven different lateral offsets from the crest representative of the moisture ingress within the embankment. Subsequently, the seven different scour depths were simulated for the above combination. Some of the analyses for both the non-scour and scour assessment were not completed due to soil collapse as described earlier and they were not considered in the fragility assessment. A typical output showing the vertical displacement with no scour is shown in Figure 3a with the plastic point output from the same assessment in Figure 3b. The same profile with 3m scour is shown in Figure 4a and Figure 4b respectively.

(a)

(b)

Figure 3. (a) Total vertical displacement (u_y) output for 3m lateral offset from the crest and 7m deep groundwater profile with no scour. Displacement at the base of the model is equal to 0m, therefore maximum value at the road level is 30.87mm. (b) Plastic points shown in red for the same output indicating slope stability issues on both shoulders of the embankment.

(a)

(b)

Figure 4. (a) Total vertical displacement (u_y) output for 3m lateral offset from the crest and 7m deep groundwater profile with 3m deep scour. Displacement at the base of the model is equal to 0m, therefore maximum value at the road level is 31.64mm. (b) Plastic points for the same output indicating slope stability issues on both shoulders of the embankment.

Fragility functions for moisture ingress

Figure 5 shows the $\text{Ln}(\text{WIM})$ plotted against $\text{Ln}(u_y)$ for the output data before the scour assessment. The horizontal lines correspond to the minor, moderate and extensive/complete mean damage thresholds (Table 2). As described in step III of the methodology, both linear and polynomial best fit regressions were calculated. The linear regression is consistent with the literature for fragility curve derivation (Baker 2015; Cornell et al. 2002; Argyroudis and Kaynia 2015, Mai et al. 2017), whilst the polynomial regression appears to better reflect the trend within the data and having higher regression coefficient, R (Argyroudis and Pitilakis 2012). The polynomial best fit regression also indicated a greater increase in displacement as the groundwater approached the surface, which was consistent with the observed soil failure indicated by Plaxis models. It is difficult to accurately extrapolate the data for the case where the groundwater is in the very shallow surface, as it is unknown to what extent the embankment would deform.

Figure 5. $\text{Ln}(\text{WIM})$ vs $\text{Ln}(u_y)$ graph for output data (excluding scour) with linear and polynomial best fit regressions.

Fragility functions are based on cumulative lognormal distribution and require a median and a standard deviation (equation 1). For each damage state the median is the WIM calculated by

solving the chosen best-fit regression equation for the threshold displacement to each damage state (Table 2). The median WIM for the minor damage is estimated equal to 0.59 and 0.63 for the linear and polynomial regression, indicating that where sufficient data is available the fragility estimates for the two regression fits have small discrepancies. The standard deviation is the total uncertainty value and was adopted consistent with the discussion in the methodology section (Step IV). The polynomial best fit regression was used to derive the fragility curves shown in Figure 6. It is noted that the adoption of a normalised water table means that the intensity measure is limited to a maximum value of one, equivalent to surface groundwater. The fragility curve for extensive/complete damage is based on the projection of the results to higher WIM. Table 3 shows the probabilities of exceeding or being in a damage state for the extreme scenario of surface groundwater (WIM=1.0). The estimated probabilities of being in minor, moderate and extensive/complete damage have been calculated as per Figure 1 and are equal to 0.30, 0.19 and 0.26, respectively.

Figure 6. Fragility functions for moisture ingress based on polynomial best fit regression.

Table 3. Probability for each damage state with surface groundwater (WIM=1.0) the fragility curves of Figure 6.

damage state (DS)	probability of exceeding a DS $P(\geq DS)$	probability of being in a DS $P(DS)$
no damage	-	0.25
minor	0.75	0.30
moderate	0.45	0.19
extensive/ complete	0.26	0.26

Fragility functions for combined moisture ingress and scour effects

The output data from the numerical analyses was plotted on a graph as shown in Figure 7. Following the findings from the non-scour assessment a polynomial best fit regression was selected for the derivation of the scour fragility functions. In this case, and after reviewing the results of the numerical simulations and the expected damage modes for the analyses which were not completed due to soil collapse, additional data points were plotted based on engineering judgement. In particular, based on the range of the maximum u_y at the step of the analyses where the soil body collapsed, additional points corresponding to settlements at moderate damage state for groundwater at the surface or at 1m below the surface were amended, as shown in Figure 7. In the same figure, the lower and upper bound of the regression curves are shown, corresponding to scour of 0.5m and 3m respectively. Fragility curves were derived for each scour depth, however, due to the tight grouping of the fragility curves, the lower and upper bounds are shown in Figure 8. The fragility curves for the intermediate scour depths (1.0, 1.5, 2.0, 2.5m) lie between these bounds, while the fragility curve for scour of 0.0m is identical to the fragility of moisture ingress only (Figure 6). It is noted that the fragility curves for extensive/complete damage were generated based on the extrapolation of the results using the best fit regressions. The extrapolation of the regression line to higher intensity measure levels is a common work hypothesis when such levels are not covered by the analyses (Argyroudis and Pitilakis 2012; Gehl et al. 2015), however, can lead to less reliable results as the response of the structure may alter beyond the range covered by the available analyses. From the fragility functions produced it is clear that there is a trend of increasing damage probability for all damage states with increased scour depth. The probabilities with surface groundwater (WIM=1.0) for each damage state and the two bounds of scour depth are shown in Table 4. The increase in exceedance probability from 0.5m to 3.0m scour for minor, moderate and extensive/complete damage is 8%, 17% and 10%, respectively.

Figure 7. Amended $\text{Ln}(\text{WIM})$ vs $\text{Ln}(u_y)$ graph for all output data (including scour) with additional points for surface groundwater and moderate damage.

Figure 8. Fragility functions for scour overlain from 0.5m to 3.0m depth.

Table 4. Probability for each damage state with surface groundwater (WIM=1.0) for the lower and upper bounds of the fragility curves in Figure 8.

scour depth (m):	probability of exceeding $P(\geq DS)$ or being $P(DS)$ in a damage state			
	0.5m (lower bound)		3.0m (upper bound)	
damage State (DS)	$P(\geq DS)$	$P(DS)$	$P(\geq DS)$	$P(DS)$
no damage	-	0.24	-	0.18
minor	0.76	0.30	0.82	0.28
moderate	0.46	0.18	0.54	0.19
extensive/ complete	0.28	0.28	0.35	0.35

Vulnerability curves and discussion

Based on the fragility functions, generated in the previous sections, it is possible to calculate the probabilities being at each damage state for all the possible intensity measures (WIM), i.e. from 0 to 1. These probabilities can be used to calculate the loss ratio for each WIM as per step 5 in Figure 1. The relation between the hazard intensity measure and the loss ratio represents the vulnerability curve of the embankment. Figure 9 shows the vulnerability curves for the variable moisture ingress (red line) and the combination of a relatively small scour hole (dashed blue line) and a relatively large scour hole (continuous blue line) with water ingress, using the repair ratios provided in Figure 1 (step 4). When the scour hole is relatively small (dashed blue line) the vulnerability of the embankment is similar to the one due to moisture ingress only (red line), as the fragility of the embankment depends on the settlement of its stem, which is not expected to change significantly. Antithetically, when the scour hole becomes significant, the vulnerability increases and as a result the solid blue line is shifted and represents higher losses.

The direct losses, which is essentially the monetary loss due to the physical damage of the asset, can be calculated for given hazard intensities, using the vulnerability curves of Figure 9, as per step 6 in Figure 1. For example, the expected direct loss for WIM=1, is equal to €311k for moisture ingress only, €326k for the combination of flooding and low scour effects and obtains the maximum value of €397k for the combined flooding and maximum scour hole effect. This estimation assumed an embankment length of 100m and a replacement cost of €1m. These costs estimations can facilitate decision making by network owners or operators for managing their infrastructure and adapting the critical infrastructure assets to aggravation of water ingress due to climate change effects.

Figure 9. Vulnerability curves for an embankment exposed to moisture ingress due to flood, and combined flood and scour effects at the toe.

Modelling simplifications and inherent limitations of the generated fragility functions are discussed in the following. The accuracy of the fragility functions is dependent on the quantity and quality of the data from the numerical model. Due to soil body collapses that occurred in the numerical model for the highest water intensities, damage was justified based on failure modes that were identified from local analysis results into stresses and strains. In this respect, the fragility functions for the non-scour assessment indicate that where there is a sufficient amount of data, i.e. minor damage state, the selected best fit regression does not change significantly the resulting fragility curve. Where the data is insufficient and extrapolation is undertaken, the fragility functions derived, i.e. for extensive/complete damage, should be treated with caution as the best fit regression adopted fundamentally changes the fragility curves.

The damage states adopted are mean values, whilst the qualitative descriptors (refer to Table 2) are expressing the operability of the asset, without justifying the extent of the damage. However, differential settlement is far more adverse to structures than uniform settlement. The vertical displacement output from Plaxis in Figure 4 shows that the displacement is a gradation across the width of the embankment. As the model is in-plane strain it must be assumed to have a similar displacement profile longitudinally along the embankment also. Therefore, the differential settlement is likely to be low and the user experience driving the road after these displacements have occurred is unlikely to be significantly affected. It is noted that the major longitudinal differential is expected to be at the start and end of the flooded section.

Furthermore, the following remarks are made with respect to the assumptions made in this study. The experience from previous flood events, reveals that when roads flood they do not always suffer damage at the road surface level, which is the location of the damage state considered in this investigation. It is noted that the majority of roads have a bound surface, and this was not assessed within the numerical model. A bound surface may help to reduce vertical displacements by bridging, and therefore reducing the magnitude, of settlement and the size of any settled areas and distributing the load over a larger area. This means that the (partial) loss of capacity or safety of the slope does not necessarily lead to the loss or reduction of the functionality, i.e. traffic, of the road, yet the capacity of the asset is reduced. The opposite might also be possible as explained in the next paragraph.

It is common that in extreme flooding -higher water intensities and/or overtopping- roads are closed to traffic and this was not allowed for in the numerical model, yet this might lead to loss of the slope capacity; to simplify analyses and produce comparable results, all water intensities and scour depths were assessed with full traffic load. Additionally, the dynamics of flooding was not considered, in that the duration of flooding has not been explicitly assessed. The higher water intensities are immediately present to the groundwater profiles modelled. In reality, the permeability of the soils will affect the water ingress over the duration of any flood. High intensity flooding usually has short duration and most likely the saturation of the embankment to its total height and width would be the most onerous conditions, which is rather unlikely.

Conclusions

Highway geotechnical assets, such as slopes, embankments and earth-filled dams, are exposed to diverse and multiple hazards, e.g. floods, earthquakes or ground movements, and their failures may cause disruptions in networks and services. This paper provides, for the first time in the literature, fragility and vulnerability curves for highway embankments when exposed to moisture ingress and scour effects, which are fundamental components of the quantitative risk analysis of a network under any natural hazard. The fragility curves for non-scour and scour scenarios were produced based on the results of numerical simulations including the embankment and foundation soil under drained conditions. The multiple hazard effects were modelled with gradual increase of the groundwater level and width of saturation throughout the embankment, followed by gradual extent of the scour hole at the toe of the embankment slope. The fragility curves are used to derive representative vulnerability curves, which can be used for the evaluation of expected physical losses in cost terms.

The findings of this investigation confirm that increasing groundwater level within a granular embankment increases the vertical displacement at the road surface level and thus its vulnerability. Linear and polynomial best fit regressions were employed for the derivation of the fragility curves for the non-scour data. The polynomial best fit regression was taken forward for the scour assessment as it appeared to fit the data well, and with additional data points added for moderate damage at the highest water intensity. The latter was based on identification of local damageability and damage modes of typical embankments. The effect of the scour to a maximum depth of 3.0m was to increase the probability of exceeding all three damage states, but by a relatively small percentage (between 3 and 6%). The fragility functions for the minor to moderate damage state were derived based on sufficient data, while for greater damage states an extrapolation of the data was required on the basis of local analysis and observation. Therefore, these fragility curves should be used with caution, whilst further research is required for the validation and improvement of this particular case of extensive/complete damage.

The importance of this paper is the generation of fragility functions for geotechnical transport assets, such as highway slopes/embankments, exposed to flooding effects, for making realistic predictions of losses and consequences. This is an acknowledged gap as the current research is focused on the fragility analysis of bridges, tunnels or retaining wall. The proposed approach is a starting point for further analyses that will include refinement of the numerical results for higher WIM and extensive/complete damage level, while it can be extended to other asset such as railway embankments/slopes, levees, dams and similar earthworks. The results of the QRA enables decision making by insurance companies, and sustainable development by owners and network operators toward efficient allocation of their resources in risk management as well as in the resilience-based design for existing and new infrastructure.

Future work should include further assessment of granular materials for the high-water intensity range as well as investigation of the effect deterioration of the embankment materials due to water ingress considering climate change effects. Dependent on the geological history of the embankment materials and their mineral composition, dissolution and collapse compression may be issues that could be investigated with regard to high groundwater inundation. This would be especially so for fine-grained materials and mixed fine- and coarse-grained materials should form the focus of future investigations. The study undertaken was for an embankment with a symmetric cross section. For sidelong embankments that are mid-slope, the potential for water runoff from the overlying slopes and the reduction in bearing capacity associated with the inclined foundation beneath the embankment mean that stresses will concentrate, and it will

likely deform asymmetrically. Additional investigation as to the effect of cross-sectional geometry on the displacement would be worthwhile including 3D modelling, especially for deriving fragility functions for mountain roads. Finally, future research should include the comparison and validation of the derived numerical fragility curves with empirical damage data on similar types of embankments, as well as the refinement of analyses for higher water intensity levels, to verify and/or improve the fragility curves for extensive/complete damages.

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