ABSTRACT

This paper presents a review of the different methodologies developed for the fragility assessment of critical transportation infrastructure subjected to geotechnical and climatic hazards with emphasis placed on geotechnical effects. Existing information on fragility analysis is synthesized, along with its parameters and limitations with particular emphasis on the numerical modeling of transportation infrastructure subjected to geohazards. The definition of system of assets (SoA) is introduced and numerical fragility curves are developed for a representative SoA subjected to flooding and seismic excitations. The paper concludes with the opportunities for future developments of fragility analyses for systems of assets under multiple hazards considering mitigation measures and ageing effects.

Keywords: Fragility curves; Highway and railway infrastructure; Earthquake; Flooding; Embankments

1. INTRODUCTION

Geo-hazards, such as earthquakes, ground movements, debris flow and floods are major threats to critical infrastructure in many regions around the world. In addition to life and physical losses, damage to transportation infrastructure may cause significant socio-economic losses and disruption. The risk of failure of transportation infrastructure due to geo-hazards is frequently underestimated or neglected. For example, in 2005 the flooding event after few hours of a very intense rainfall in Southern Italy, resulted in six deaths, numerous injuries and severe damage of roads, railways and other infrastructure (Polemio and Lollino 2011). The 2010-2011 Canterbury earthquake sequence in New Zealand caused extensive damage to road networks due to rock falls and liquefaction, which resulted in settlements, lateral spreading, sand boils and water ponding on the road surfaces (Kongar et al., 2017). In Europe, weather stresses represent 30% to 50% of road maintenance cost (up to 13 billion € p.a.), while 10% of these costs are associated with effects of extreme weather events (Nemry and Demirel 2012). In the United States recent studies on bridges revealed that 53% of the failures between 1989 and 2000 were due to hydraulic causes (i.e. flooding, scouring). Moreover, highway embankment damage caused by flooding in coastal and riverine environments is one of the current challenges faced in the United States (Briaud and Maddah 2016). Thus, reliable assessment of the vulnerability of, and the associated risks on, transportation infrastructure subjected to critical geo-hazards is of paramount importance, since it will enable the efficient allocation of resources toward resilient transportation networks.

The importance of risk assessment is proven by the recent research interest in quantitative risk analysis (QRA), which refers to the protection of critical infrastructure assets subjected to natural hazards.
2. FRAGILITY CURVES FOR TRANSPORTATION INFRASTRUCTURE

Physical vulnerability is a fundamental component in risk analysis under any hazard, and its accurate estimation is essential in making reasonable predictions of losses and associated impacts. It can be quantified using vulnerability or fragility functions. Vulnerability functions describe the degree of losses (e.g., monetary costs, casualties, down-time, environmental degradation etc) of a given asset or system of assets as a function of the hazard level. Vulnerability of transportation infrastructure can be expressed by repair costs, life-safety impacts or loss of functionality and is related to damageability of the assets. The latter is commonly expressed through fragility functions, which give the probability that the asset exceeds some undesirable limit state (e.g., serviceability) for a given level of hazard intensity such as force, deformation, or other type of loading to which the asset is subjected. The vulnerability and fragility functions can be derived by empirical, analytical, expert elicitation and hybrid approaches and they provide a valuable tool to explicitly assess the vulnerability of structures (NIBS 2004, Pitilakis et al. 2014). Analytical approaches validated by experimental data and observations from recent events, have become more popular, as they are more readily applied to different structure types and geographical regions, where damage records are insufficient.

A substantial increase of interest in the seismic fragility assessment of transportation infrastructure is evidenced in the literature. The studies concern mainly bridge assets (Tsionis and Fardis 2014, Billah and Alam 2015, Gidaris et al. 2017). Empirical fragility curves for bridges have been developed based on post-earthquake damage observations such as after the 1994 Northridge and 1995 Kobe earthquakes, using different statistical approaches (e.g., Basoz and Kiremidjian 1998, Shinozuka et al. 2001, Elnashai et al. 2004). Analytical methods have been widely applied, including elastic spectral analysis (e.g. Hwang et al. 2000), nonlinear static analysis (e.g. Stefanidou and Kappos 2017), nonlinear time history analysis (e.g. Kwon and Elnashai 2010), incremental dynamic analysis and Bayesian approaches (e.g. Gardoni et al. 2002). SSI effects on fragility analysis of bridges have been addressed in several studies (e.g. Stefanidou et al. 2017), while liquefaction-sensitive fragility curves were developed based on numerical modeling including SSI effects (Kwon and Elnashai 2010, Aygün et al. 2011). The influence of corrosion on the seismic fragility has been investigated by Ghosh and Padgett (2010) and Zhong et al. (2012) among others. The effect of retrofitting measures has also been studied (e.g., Kim and Shinozuka 2004, Padgett and DesRoches 2009). The combined effect of flood-induced scouring and earthquake to the fragility of bridges has been studied analytically by Wang et al. (2012), Alipour and Shafei (2012), Prasad and Banerjee (2013), Banerjee and Prasad (2013), Dong et al. (2013), Wang et al. (2014), Guo et al. (2016) and Torres et al. (2017). In most of the studies, to account for scouring, the spring elements between the soil and the foundation piles were removed down to a depth equal to the scour depth. Gehl and D’Ayala (2017) developed multi-hazard fragility functions, through the use of system reliability methods and Bayesian networks.

The available fragility models for railway and highway infrastructure other than bridges (i.e. tunnels, embankments/cuts, slopes, retaining walls) subjected to seismic shaking are summarized by
argyroudis and kaynia (2014). empirical and expert-based fragility curves for tunnels were proposed by sasaki et al. (2000), ala (2001), nibs (2004) and corigliano (2007). analytical fragility curves were developed for tunnel structures under seismic shaking (argyroudis and pitilakis 2012, andreotti and lai 2014, mayoral et al. 2016). the effect of corrosion of the lining has been studied by argyroudis et al. (2017), whilst kiani et al. (2016) proposed experimental fragility curves for circular tunnels as a function of fault rupture. empirical fragility curves for road embankments have been generated by sasaki et al. (2000), maruyama et al. (2010) and nakamura (2015) as a function of peak ground acceleration (pga) or peak ground velocity (pgv) based on damage observations in japan. argyroudis et al. (2013) and argyroudis and kaynia (2015) developed analytical fragility curves for cantilever bridge abutments-backfill system and embankments and cuts respectively under seismic shaking for different soil conditions. lagaros et al. (2009) proposed analytical fragility functions for embankments based on pseudostatic slope stability analyses, while yin et al. (2017) investigated the influence of retaining walls on embankment seismic fragility following an incremental dynamic analysis. tsubaki et al. (2016) developed fragility curves for railway embankment fill and track ballast scour based on recorded observations of railway damage in japan and simulated overtopping water depth. sasaki et al. (2000) proposed empirical fragility curves for slopes, while wu (2015) developed analytical fragility functions for a combination of slope geometries and loading conditions, e.g. earthquake and rainfall events. the fragility model provided by pitilakis et al. (2010) considered the slope characteristics through the yield coefficient. fragility curves for roads subjected to debris flow were developed by winter et al. (2014) as a function of the landslide volume based on an expert judgment approach. in general, the available models for ground failures are limited. generic fragility functions for tunnels, roads and bridges subjected to ground failure due to liquefaction and fault displacement are provided by nibs (2004), yet not accounting for the typology of assets or the soil conditions.

as a conclusion, numerous studies have assessed the seismic vulnerability of individual transportation assets, such as retaining walls, tunnels, and mainly bridges. it is worth noticing, that most common intensity measure types used, are the peak ground acceleration (pga) when ground shaking is the cause of damage or the permanent ground displacement (pgd) in case of ground failure. regarding hazards, other than earthquakes, past studies have focused on the effects of liquefaction, landslides, debris-earth flow and flood and the combined effects of scouring and earthquakes. again, these studies mainly concern bridges, and this is also the case for those investigating the effects of potential mitigation measures, deterioration due to previous hazard events or aging effects on the fragility of the assets.

3. system of assets (soa)

the review of the literature concluded that available vulnerability and risk assessment frameworks typically consider individual assets of the transportation infrastructure, exposed to one hazard and are static, i.e., they neglect changes of the asset performance during its life. however, infrastructure assets comprise systems of assets (soa), i.e. a combination of interdependent assets exposed to multiple hazards, whilst their performance changes during their life due to deterioration or improvements that take place. in figure 1 a representative transportation soa is illustrated along with common hazard effects. it includes slopes responding together and interacting with a road pavement or railway tracks on embankments and supported by retaining structures, exposed to landslides, potentially triggered by precipitation or earthquakes, flooding effects or/and ground shaking. degradation of the soa, in this case, may be the result of scouring of the embankment or the foundation soil and potential residual dislocations of the retaining structures. the stability of the soa may deteriorate during its lifetime as a result of increases in the stresses or traffic loads, decreases of soil shear strength due to changes in pore water pressure and presence of organic materials. potential improvement measures may include shotcreting, soil anchors, nailing, vegetation, and drainage.
4. DEFINITION OF DAMAGE

The performance levels of an asset are defined through damage thresholds called limit states, which define the boundaries between different damage conditions or damage states. Various damage criteria have been used depending on the typology of the asset and the method used for the fragility analysis. In analytical methods, damage is related to limit state mechanical properties that are described through appropriate damage indices. Threshold values of the indices are correlated to damage states. The number of damage states is variable (e.g. none, minor, moderate, extensive, complete) depending on the type of asset. Damage states are usually correlated to the restoration time and traffic capacity of the assets and in some cases to the replacement, repair and enhancement costs (NIBS 2004, Werner et al. 2006, Mackie and Stojadinovic 2006, Tsionis and Fardis 2014, D’Ayala et al. 2015).

Bridge damage is related to the response of components of the bridge, i.e. the deck, the piers and foundation, bearings, abutments and expansion joints. For piers, the damage indices used in practice are the drift ratio, the curvature, rotation, and displacements. The seismic response of the abutments is usually described based on its displacement and rotation, while the damage index for bearings is its longitudinal and transverse shear deformations and for bridge foundations is the sliding and soil bearing capacity. Damage states have been defined for the specific bridge components and for the whole bridge (Tsionis and Fardis 2014, D’Ayala et al. 2015). Regarding damage after floods, it has been observed that the most common components to be affected by flood hazards are the pier/abutment foundations (due to scour) and the bridge deck in specific situations (i.e. overtopping). Floods can also result in more spatially distributed disruptions, such as the modification of the waterway channel, which may affect larger areas around the bridge. In case of river crossings, failure mechanisms of rock bank and riprap protections include slope instabilities, sliding, movement of rock cover, migration of sub-layers, etc (Melville and Coleman 2000, CIRIA 2007). Most of these mechanisms are related to flow characteristics such as discharge, flow velocity and water levels and also to geotechnical characteristics such as density of materials or pore water pressure (Roca and Whitehouse 2012).

Earthquake effects on tunnels include slope instability leading to tunnel collapse, portal failure, roof or wall collapse, invert uplift spalling, cracking or crushing of the concrete lining, slabbing or spalling of the rock around the opening, bending and buckling of reinforcing bars, pavement cracks, wall
deformation, local opening of joints and obstruction at the tunnel portals (rock falls). Non-seismically induced landslides can cause similar damage modes. Flooding is not considered as a crucial hazard for tunnels; however, underground water can have a damaging effect on the tunnel lining during their lifetime to corrosion of reinforcement or degradation of concrete strength (ITA 1991). In terms of fragility assessment, damage states commonly describe the response of the main tunnel components (i.e. liner, portal, support systems). In numerical methods damage states are described based on the exceedance of lining capacity (Argyroudis and Pitilakis, 2012, Argyroudis et al. 2017), number of activated plastic hinges in the liner (Andreotti and Lai 2014, Lee et al. 2016), lateral displacement (Huh et al. 2017) or permanent rotations of longitudinal joint (Fabozzi et al. 2017).

Failure modes of embankments are related to ground failures due to soil liquefaction or dynamic loading, including sliding or slumping of the embankment, cracking at the surface and settlement. Damage states are defined in the literature based on the extent of settlement or ground offset (NIBS 2004, Maruyama et al. 2010, Argyroudis and Kaynia 2015). The failure mechanisms commonly encountered during flooding involve hydrostatic and hydrodynamic forces that result from overtopping, seepage forces and the lateral pressure caused by headwater elevation. Common failure modes in coastal and riverine environments include: overtopping erosion, softening by soil saturation, underseepage, and piping, through seepage (internal erosion) and piping, wave erosion, lateral sliding on foundations, other failure modes including culvert failures and pavement failures (ALA 2005, Briaud and Maddah 2016). Damage states are not provided in the literature; however, the ones proposed in case of earthquake damage can be adopted for floods.

Roads and railbeds constructed on slopes are subjected to potential failure mechanisms due to large movements of the slopes or slumping of the sides of the road or railbed. Landslides and rock falls can cause partial or complete closure of the road or railbed as well as potential structural damage to the pavement or the rail track. Damage states are defined according to the extent of settlement or ground offset (NIBS 2004, Argyroudis and Kaynia 2015). The main seismic failure of backfills behind bridge abutments or retaining walls is the backfill settlement or heaving. Structural damage of the abutment wall includes permanent dislocation (i.e. sliding, rotations) and cracking. In addition, pounding of the deck to the abutment can seriously affect the overall response of the bridge due to collision forces.

The damage states in Table 1 (as per Argyroudis and Kaynia 2015) are given in terms of permanent ground displacement (PGD) for roadway and railway components based on a range of values from a review of the literature.

### Table 1. Definition of damage states for highway and railway assets.

<table>
<thead>
<tr>
<th>Typology</th>
<th>Damage State</th>
<th>Permanent vertical ground displacement [m]</th>
<th>Serviceability</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Min</td>
<td>Max</td>
</tr>
<tr>
<td>Highways</td>
<td>Minor</td>
<td>0.02</td>
<td>0.08</td>
</tr>
<tr>
<td></td>
<td>Moderate</td>
<td>0.08</td>
<td>0.22</td>
</tr>
<tr>
<td></td>
<td>Extensive/ Complete</td>
<td>0.22</td>
<td>0.58</td>
</tr>
<tr>
<td>Railways</td>
<td>Minor</td>
<td>0.01</td>
<td>0.05</td>
</tr>
<tr>
<td></td>
<td>Moderate</td>
<td>0.05</td>
<td>0.10</td>
</tr>
<tr>
<td></td>
<td>Extensive/ Complete</td>
<td>0.10</td>
<td>0.30</td>
</tr>
</tbody>
</table>

5. NUMERICAL FRAGILITY CURVES FOR EMBANKMENT UNDER MULTI-HAZARD EFFECTS

In this section, fragility curves are developed for a transportation asset, i.e. an embankment, subjected to flooding followed by earthquake based on a series of full dynamic time history analyses. The
analyses were performed under plane strain conditions using the finite element code PLAXIS 2D ver.2015.02.

5.1 Description of the numerical model

The embankment considered herein has a width of 20 m and height equal to 6 m, with an inclination of slopes 1.5(H):1(V). A distributed vertical load equal to 20kN/m/m is applied on the top of the embankment to account for the live loads. The construction material is clay having variable shear wave velocity ($V_s$) and initial shear modulus ($G_{\text{max}}$) with depth (Figure 2). An initial value for the undrained shear strength ($S_u$) at the top layer was assumed to be equal to 20 kPa, while a gradient model for $S_u$ is defined for the next layers ($S_u = S_{u0} + 0.25\sigma_v'$, where $\sigma_v'$ is the average vertical effective stress in layer n).

The foundation soil is a 29 m soft clay deposit corresponding to ground type D according to EC8, having variable values of $V_s$ and $G_{\text{max}}$ as shown in Figure 2. The variation of $S_u$ with depth follows the same gradient model described for the embankment layers above. The values of the other soil parameters were taken as $v = 0.35$, $\gamma_{\text{unsat}} = 18$ kN/m$^3$, $\gamma_{\text{sat}} = 20$ kN/m$^3$, $c_{\text{init}} = 0.5$ for both the embankment and foundation soil. The period of the soil profile is about 0.90 s. The initial water level was assumed to be at the bottom of the embankment, while it was gradually increased to 0.5, 1.0, 1.5 and 2.0 m above the bottom of the embankment (Figure 3b). The flooding effect was taken into account by modifying the properties of the flooded (i.e. saturated) layers of the embankment. In particular, a reduced shear strength of the saturated layers was applied using a model for $S_u$ similar to the initial condition ($S_u = S_{u0} + 0.25\sigma_v'$, where $S_{u0}$ was set equal to 5 kPa and $\sigma_v'$ was the vertical effective stress corresponding to saturated unit weight).

To account for the dependency of both the stiffness and the damping on the primary shear strain level during the earthquake, a reduced value of the shear modulus ($G$) was introduced for each layer during the seismic excitation phase. In particular, an average $G/G_{\text{max}}$ ratio equal to 0.6 was assumed based on previous studies (Argyroudis and Kaynia 2015). The Rayleigh damping parameters were chosen for the frequency interval 1.0-3.0 Hz based on an average value of the critical damping ratio of 8.5%.

The model width was 260.0 m to reduce the boundary effects on the structure (Figure 3a). The domain was discretised with a total of 2219 15-node plain strain triangular elements. All the analyses included initial stages simulating the initial geostatic stresses and the activation of the traffic load. The base of the model was fixed in both horizontal and vertical directions, during the initial steps and the simulation of inundation conditions. For the dynamic analyses, the horizontal direction was released and the seismic input was uniformly applied at the basis of the model. The normally fixed and the tied degrees of freedom were selected for the lateral boundaries during the initial and the dynamic phase respectively.

![Figure 2. Variation of shear wave velocities ($V_s$) and initial shear modulus ($G_{\text{max}}$) with depth for the embankment and foundation soil](image-url)
Eight real acceleration time histories from different earthquakes recorded on rock or very stiff soil were selected as outcrop motion for the analyses: Friuli-Venezia Giulia, $M_w=6.4$, Italy, 1976; Kocaeli (Gebze), $M_w=7.4$, Turkey, 1999; Parnitha (Kypseli), $M_w=6.0$, Greece, 1999; Kozani (Prefecture building), $M_w=6.5$, Greece, 1995; Duzce (Ldeo Station No. C1058 Bv), $M_w=7.2$, Turkey, 1999; Umbria Marche (Gubbio-Piana), $M_w=4.8$, Italy, 1998, Hector Mine (Hector), $M_w=7.1$, USA, 1999; Loma Prieta (Diamond Height), $M_w=6.9$, USA, 1989. The normalized mean of the acceleration spectra of the selected motions matches EC8 spectrum for soil class A. In the dynamic analyses, the time histories are scaled so that their PGAs increases from 0.15 to 0.75g with a step of 0.15g. The seismic excitations are applied separately for each inundation depth in order to simulate the combination of flooding and earthquake hazards. A representative example of the analysis output is given in Figure 4 where the vertical displacements of the embankment are illustrated.

![Figure 3. Layout of the model (a) and increase of the water level corresponding to inundation equal to 0.5, 1.0, 1.5 and 2.0 m (b)](image)

![Figure 4. Distribution of the permanent vertical displacements of the embankment for the input motion Kypseli 0.30g, (a) no flood, (b) inundation depth 2.0m. (Note: The maximum permanent vertical displacement i.e. top minus bottom of the embankment, is 0.041 m for 4a and 0.068 m for 4b)](image)

### 5.2 Derivation of fragility curves

Fragility functions describe the probability of exceeding different limit states (LS) for a given earthquake intensity measure, IM, here defined by PGA at bedrock conditions. Fragility curves are usually described by a lognormal probability distribution function. Their development requires the definition of two parameters, $\text{IM}_{\text{mi}}$ (i.e. the median threshold value of IM required to cause the $i^{th}$ damage state) and $\beta_{\text{tot}}$ (i.e. the total lognormal standard deviation). It is based on the correlation between the damage indices and the increasing seismic intensity in terms of PGA, which provides the regression curve. In this study, the damage is defined in terms of maximum permanent vertical ground displacement ($U_y$) of the embankment. In particular, the relative displacement between the top and bottom of the embankment is considered. The $\text{IM}_{\text{mi}}$ can be obtained for each damage state using the regression curve (Figure 5) and the definitions of damage states given in Table 1 for highway and railway assets. The corresponding values for highways and railways are given in Table 2, for minor, moderate and extensive/complete damage. The $\beta_{\text{tot}}$ includes three sources of uncertainty. The one associated with the definition of damage states ($\beta_{\text{ds}}$) was taken 0.4 as per NIBS (2004) for buildings.
The uncertainty due to the capacity ($\beta_C$) that was considered 0.3 based on engineering judgment. The third uncertainty is associated with the seismic demand and calculated by the dispersion in response (i.e. $U_y$) due to the variability of the seismic input motion (Figure 5). The total variability was estimated by the combination of the three contributors, assuming that they are statistically independent and lognormally distributed random variables (Table 2). The derived fragility curves are given in Figure 4 for highways and railways on the embankment, for different levels of inundation depth (i.e. 0, 0.5, 1.0, 1.5, 2.0 m).

Figure 5. Evolution of damage (permanent vertical displacement $U_y$) with intensity measure (PGA bedrock) for no flood (left) and inundation depth 2.0 m (right). Limit states (LS) correspond to highway assets.

Table 1. Seismic fragility parameters for highway and railway assets subjected to different flood inundation.

<table>
<thead>
<tr>
<th>Inundation</th>
<th>Damage State</th>
<th>Median PGA (g)</th>
<th>Median PGA (g)</th>
<th>$\beta_{tot}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Highways</td>
<td>Railways</td>
<td></td>
</tr>
<tr>
<td>0 m</td>
<td>Minor</td>
<td>0.29</td>
<td>0.20</td>
<td>0.76</td>
</tr>
<tr>
<td></td>
<td>Moderate</td>
<td>0.60</td>
<td>0.39</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Extensive/Complete</td>
<td>1.17</td>
<td>0.73</td>
<td></td>
</tr>
<tr>
<td>0.5 m</td>
<td>Minor</td>
<td>0.28</td>
<td>0.20</td>
<td>0.78</td>
</tr>
<tr>
<td></td>
<td>Moderate</td>
<td>0.57</td>
<td>0.38</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Extensive/Complete</td>
<td>1.09</td>
<td>0.69</td>
<td></td>
</tr>
<tr>
<td>1.0 m</td>
<td>Minor</td>
<td>0.26</td>
<td>0.19</td>
<td>0.79</td>
</tr>
<tr>
<td></td>
<td>Moderate</td>
<td>0.55</td>
<td>0.36</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Extensive/Complete</td>
<td>1.04</td>
<td>0.66</td>
<td></td>
</tr>
<tr>
<td>1.5 m</td>
<td>Minor</td>
<td>0.24</td>
<td>0.17</td>
<td>0.76</td>
</tr>
<tr>
<td></td>
<td>Moderate</td>
<td>0.51</td>
<td>0.33</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Extensive/Complete</td>
<td>1.01</td>
<td>0.63</td>
<td></td>
</tr>
<tr>
<td>2.0 m</td>
<td>Minor</td>
<td>0.22</td>
<td>0.15</td>
<td>0.77</td>
</tr>
<tr>
<td></td>
<td>Moderate</td>
<td>0.49</td>
<td>0.31</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Extensive/Complete</td>
<td>0.99</td>
<td>0.60</td>
<td></td>
</tr>
</tbody>
</table>
Figure 6. Seismic fragility curves for highway (left) and railway (right) embankments for different inundation levels.

6. CONCLUSIONS

This paper concludes that there is a substantial increase of research efforts on the vulnerability and risk assessment of transportation infrastructure against geo-hazards. However, advanced numerical modelling of transportation assets is limited and focused on bridges. Hence, there is a need for more systematic analyses and validation of the results and their applicability. Also, the effects of deterioration and mitigation measures in their fragility response should be taken into account. Yet, there is a lack of systematic vulnerability assessment for System of Assets (SoA), which is the missing link between the assessment of the component and the condition of the network.

The paper also presented the derivation of fragility curves for a transportation asset, subjected to flooding effect and seismic excitations based on 2D coupled non-linear dynamic analysis. It is recognized that the modeling of the flood was simplified and static in comparison to the sophistication level attempted for the earthquake excitations. However, the modeling of water flow would be crucial for scouring and erosion effects but is not expecting to affect significantly the permanent displacement. The results showed that the saturation effect can modify the seismic response of the embankment. For example, the probability that a highway on embankment will experience moderate damage when subjected to seismic excitation of 0.4g can be increased by about 10% when the inundation depth is 2.0 m compared against the non-flooded condition. Moreover, the response of the embankment may vary significantly for different input motions (e.g. duration, frequency content, seismotectonic environment). It is also noted that the dispersion of the fragility for different flooding depths, is higher when PGA is larger than ~0.2g, whereas for smaller PGA it is evident that the inundation depth plays an insignificant role. This is attributed to the fact that the embankment remains essentially elastic for low intensities of ground shaking. Finally, the lower tolerance of railway assets to deformation resulted in higher vulnerability compared to highways. For example, a railway is expected to have about 20% higher probability for moderate damage compared to a highway, when subjected to a ground shaking of 0.4 g in the non-flooded conditions.

7. ACKNOWLEDGMENTS

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