Fragility assessment of transportation infrastructure systems subjected to earthquakes

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ABSTRACT

This paper provides a review of the different methodologies for the fragility assessment of critical transportation infrastructure subjected to earthquake excitations with emphasis placed on geotechnical effects. Available approaches to fragility analysis are summarized, along with the main parameters and limitations. Additionally, definitions of damage are synthesized for the individual transportation assets and subsequently the definition of system of assets (SoA) is introduced. Numerical fragility curves are developed for a representative SoA subjected to seismic excitations. The paper concludes with the gaps in the area of fragility analysis and the needs for future development.

INTRODUCTION

Earthquakes and other geo-hazards, such as ground movements, debris flow, and floods are major threats to 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 impact. In recent years, the 2008 Wenchuan earthquake in China triggered more than 15000 landslides and caused more than 20000 fatalities, while a lot of urban areas were cut-off due to the extensive damage of highways (Tang et al. 2011). The 2010-2011 Canterbury earthquake sequence in New Zealand caused extensive damage to road networks due to liquefaction that resulted in settlements, lateral spreading, sand boils and water ponding on the road surfaces. Moreover, rock falls led to several road closures (Kongar et al. 2017). Extensive bridge damage was reported after the 2010 Maule earthquake in Chile due to inadequate seismic design. The effects of structural irregularity and soil liquefaction were proven to be critical for the performance of bridges (Kawashima et al. 2011). Thus, reliable assessment of the vulnerability of, and the associated risks on, transportation infrastructure subjected to seismic and other hazards is of paramount importance, since it will enable the efficient allocation of resources toward resilient transportation networks.

During the last decade research interest in the quantitative estimation of vulnerability of transportation infrastructure due to natural hazards grew as part of a broader focus on the protection of critical infrastructure. In particular, quantitative risk analysis (QRA) is important because it quantifies the probability of a given level of loss and the associated uncertainties. It also quantifies risk in an objective and reproducible manner and therefore allows cost-benefit analysis and provides the basis for the prioritization of management and mitigation actions. QRA includes hazard identification, vulnerability evaluation of the infrastructure exposed to the given hazards and risk assessment in terms of economic, functional and social losses (Corominas et al. 2014). Risk analysis classifies the most vulnerable parts of the network and prioritizes the assets that require detailed analysis and potentially mitigation measures. Most studies have addressed direct seismic shaking effects, focusing on bridges, which are the most critical components.

This paper provides a review of the different methodologies for fragility assessment of critical transportation infrastructure subjected to earthquake excitations with emphasis placed on geotechnical effects. Numerical fragility curves are developed for a system of assets comprising the backfills and a three-span integral bridge subjected to seismic excitation.

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 ease of damage 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). 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 combined effect of flood-induced scouring and earthquake to the fragility of bridges has been studied by Prasad and Banerjee (2013) and Alipour and Shafei (2012). Gehl and D'Ayala (2017) developed multi-hazard fragility functions, through the use of system reliability methods and Bayesian networks. The influence of corrosion, which causes deterioration of the assets, on the seismic fragility has been investigated by Ghosh and Padgett (2010) and Zhong et al. (2012) among others. The effect of retrofitting/mitigation measures has also been studied (e.g., Kim and Shinozuka 2004; Padgettt and DesRoches 2009).

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 fragility curves for road embankments have been generated by Maruyama et al. (2010) 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. 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. Wu (2015) developed analytical fragility functions for a combination of slope geometries. The fragility model provided in Argyroudis and Kaynia (2014) 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 judgement approach. In general, the available models for ground failures are limited. Generic fragility functions for assets subjected to ground failure due to liquefaction and fault rupture are provided by NIBS (2004), yet not accounting for the specific 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. 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 ageing effects on the fragility of the assets. It is also worth noting, 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. A summary of available fragility curves is provided in Table 1 (not exhaustive).

Table 1. Summary of fragility curves for transportation infrastructure.

Typology	Type of analysis	Reference
Bridges	Review of all types	Tsionis and Fardis (2014); Billah and Alam (2015); Gidaris et al. (2017)
	Empirical	Basoz and Kiremidjian (1998); Shinozuka et al. (2001)
	Analytical (earthquake)	Hwang et al. (2000); Gardoni et al. (2002); Stefanidou and Kappos (2017); Stefanidou et al. (2017)
	Analytical (liquefaction)	Kwon and Elnashai (2010); Aygün et al. (2011)
	Analytical (scouring and earthquake)	Alipour and Shafei (2012); Prasad and Banerjee (2013)
	Analytical (corrosion)	Ghosh and Padgett (2010); Zhong et al. (2012)
	Analytical (retrofitting)	Kim and Shinozuka (2004); Padgettt and DesRoches (2009)
	Bayesian network (multi hazard)	Gehl and D'Ayala (2017)
Embankments	Empirical	Maruyama et al. (2010)
	Analytical	Lagaros et al. (2009); Argyroudis and Kaynia (2015); Yin et al. (2017)
Cuts	Analytical	Argyroudis and Kaynia (2015)
Slopes	Analytical	Wu (2015)
Bridge abutments-backfill	Analytical	Argyroudis et al. (2013)
Railway tracks/roadbeds	Empirical/expert judgement (ground failure)	HAZUS (NIBS 2004)
Roads	Expert judgement (debris flow)	Winter et al. (2014)

DEFINITION OF DAMAGE AND SYSTEM OF ASSETS (SOA)

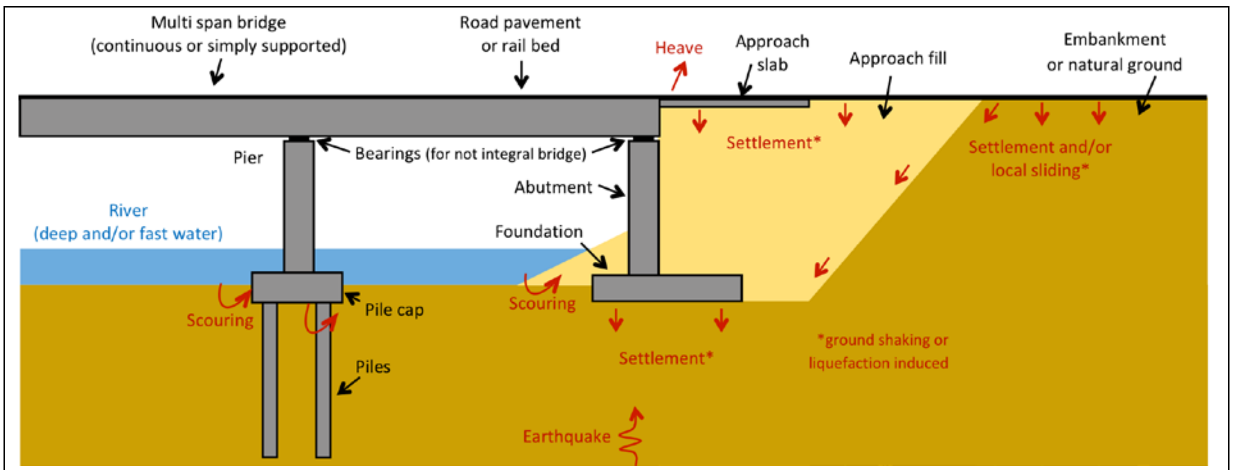
Bridge damage is related to the response of components of the bridge, i.e. the deck, the piers and foundation, bearings, abutments and expansion joints. For piers, the damage indices used in practice are the drift ratio, the curvature, rotation and displacements. The response of the abutments is usually 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). 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). 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 of 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 (see

Table 2, as per Argyroudis and Kaynia 2014). 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. Damage states are usually correlated to the restoration time and traffic capacity of the assets (NIBS 2004; Tsionis and Fardis 2014; D'Ayala et al. 2015).

The literature review came up with the conclusion 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 typical transportation SoA is illustrated along with common hazard effects. Degradation in this case may be the result of corrosion of the reinforced concrete elements, the scouring of foundation soil and residual dislocations of the abutments; similarly degradations of the approach fill can be due to traffic loads and residual deflection of the backfill such as settlements or heaving. Improvements may include strengthening of the piers and/or the abutments and use of alternative materials for the backfill, such as rubber-sand mixtures (Argyroudis et al. 2016).

Table 2. Definition of damage states for highway and railway assets.

		Permanent vertical ground displacement [m]			Serviceability
Typology	**Damage State**	**Min**	**Max**	**Mean**	
Highways	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
Railways	Minor	0.01	0.05	0.03	Open, reduced speeds
	Moderate	0.05	0.10	0.08	Closed during repair works
	Extensive/Complete	0.10	0.30	0.20	Closed during reconstruction works



Pier height: variable depending on bridge type and local geomorphology, commonly 5.0 to 20.0 m. Abutment height: variable depending on bridge type and local geomorphology, commonly 2.0 to 10.0 m. Deck cross section height: commonly 1.0 to 2.0 m. Span length: variable, commonly 15.0 m to 35.0 m.

Figure 1. Geo-hazard effects to transportation SoA: bridge, abutment and embankment.

NUMERICAL FRAGILITY CURVES FOR INTEGRAL BRIDGE-BACKFILL SYSTEM

In this section fragility curves are developed for a SoA, comprising an integral bridge-backfill system 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 and the numerical model was validated based on the study of Caristo et al. (2016), where both reinforced and conventional backfills had been examined, with the latter being the case for this paper.

Description of the numerical model

The bridge considered herein is a three span pre-stressed concrete bridge, with a total length of 100.5 m. The bridge has no expansion joints or bearings, thus it is a fully integral bridge. It has three equal spans of 33.5 m and two piers and two full height integral abutments. The deck is a box girder and has total width of 13.5m. The height of the abutments is 8 m, the footing has thickness of 1m and is 5.5 m long. The piers are wall-type section with dimensions 1x4.5 m in the longitudinal and transversal direction respectively and a height of 10 m. The footing has thickness of 1 m and 3.5 m long. A distributed load equal to 18.5 kN/m/m is applied on the bridge, accounting for the self-weight of the deck and the live loads (Eurocode 8-Part1). The bridge elements used C30/37 concrete. The unit weight was taken equal to γ=25 kN/m^3 and the elastic modulus as E=3.5E+06 kN/m^2.

The foundation soil is very stiff clay classified as ground type B according to Eurocode 8-Part1, with mechanical properties that gradually increase with the depth. It is divided in 10 layers, the first with 2 m and the following with 3 m thickness. Its unit weight is γ=19.5 kN/m^3 and the Poisson’s ratio is ν=0.35. A calibration procedure was followed in order to account for the dependency of both the stiffness and the damping on the primary shear strain level during the earthquake (Argyroudis and Kaynia 2015). The backfill material is a well compacted sand with a friction angle equal to φ = 42°, unit weight γ= 18.5 kN/m^3 and Poisson’s ratio ν= 0.40. The parameters of the Rayleigh damping were computed based on an average value of damping for all the layers ranging between 7.0 and 12.8 % for the frequency interval 0.5-1.0 Hz. The rest of the soil properties can be found in Caristo et al. (2016).

The model width was 400.0 m to reduce the boundary effects on the structure (Figure 2a). The domain was discretised in a total of 9759 15-node plain strain triangular elements. All analyses included initial stages simulating both the initial geostatic stresses and the construction of the bridge. The base of the model was fixed in both horizontal and vertical directions, during the initial steps. 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. The boundary conditions were tested and chosen based on the available in the PLAXIS 2D ver.2015.02 and validated against published results (Caristo et al. 2016). For all the analysis phases an elasto-plastic soil behavior was assumed (i.e. Mohr-Coulomb criterion). Proper interface elements having a friction coefficient of R_{inter}=0.70 were used to model the interface between the abutment and the backfill, and the footings and the foundation soil. The analyses were performed in total stresses, assuming undrained conditions, a valid and commonly adopted hypothesis for fine-grained soils subjected to severe shaking. Therefore, the effect of water pressure was not considered.

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. A representative example of the analysis output is given in Figure 2b where the vertical displacements of the backfill are illustrated.

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, IM_{mi} (i.e. the median threshold value of IM required to cause the ith damage state) and β_{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, damage is defined in terms of maximum permanent ground displacement (Uy) of the backfill behind the abutment. The IM_{mi} can be obtained for each damage state using the regression curve (Figure 3) and the definitions of damage states given in Table 2 for highway and railway assets. In particular, the PGA values were calculated based on the mean displacement provided for each damage state in Table 2 (different for highways and railways) using the regression equation in Figure 3.

The corresponding IM_{mi} values were estimated equal to 0.21, 0.46, 0.92 g for highways and 0.15, 0.29, 0.56 g for railways, for minor, moderate and extensive/complete damage respectively. The β_{tot} includes three sources of uncertainty. The one associated with the definition of damage states (β_{ds}) was taken 0.4 as per NIBS (2004) for buildings. The uncertainty due to the capacity (β_C) that was considered 0.3 based on engineering judgment. The third uncertainty is associated with the seismic demand and calculated equal to 0.47 by the dispersion in response (i.e. Uy) due to the variability of the seismic input motion. The total variability was estimated equal to 0.69 by the combination of the three contributors, assuming that they are statistically independent and lognormally distributed random variables. The resulted fragility curves are shown in Figure 4.

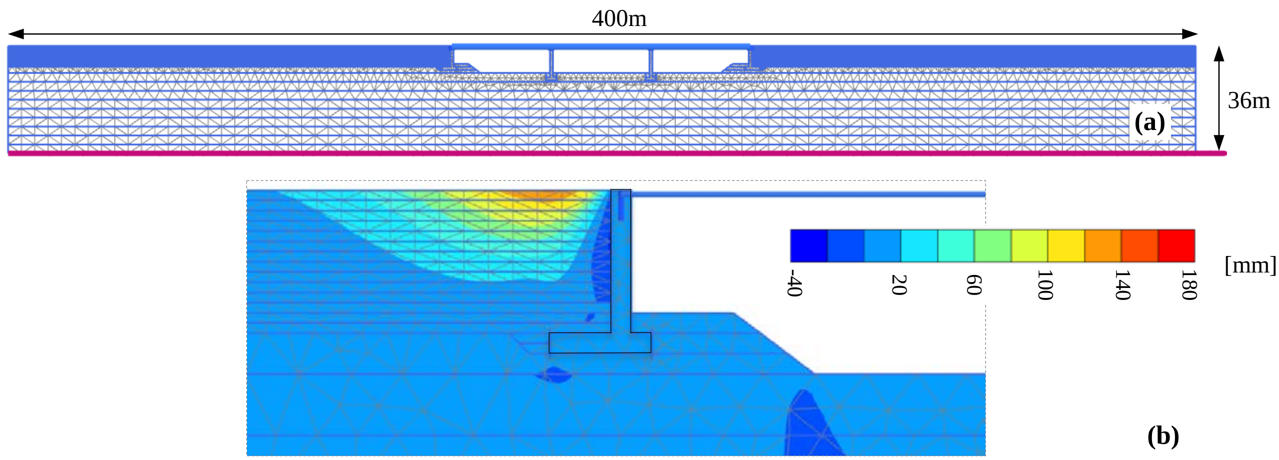


Figure 2. Layout of the model (a) and distribution of the permanent vertical displacements of the backfill for the input motion Gebze 0.45g.

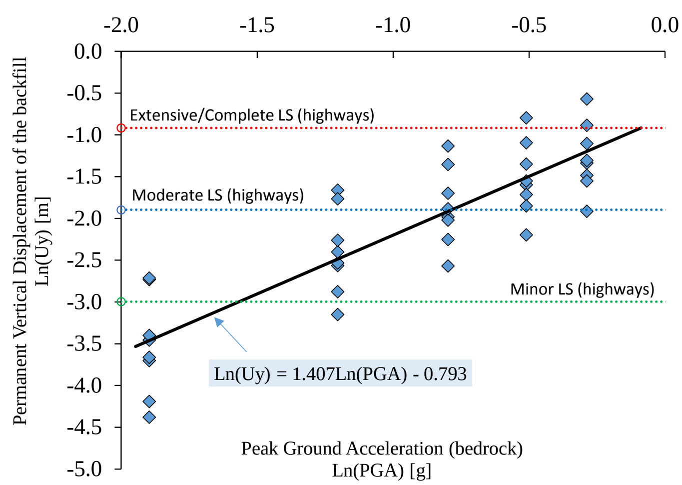


Figure 3. Evolution of damage (Uy) with intensity measure (PGA bedrock)

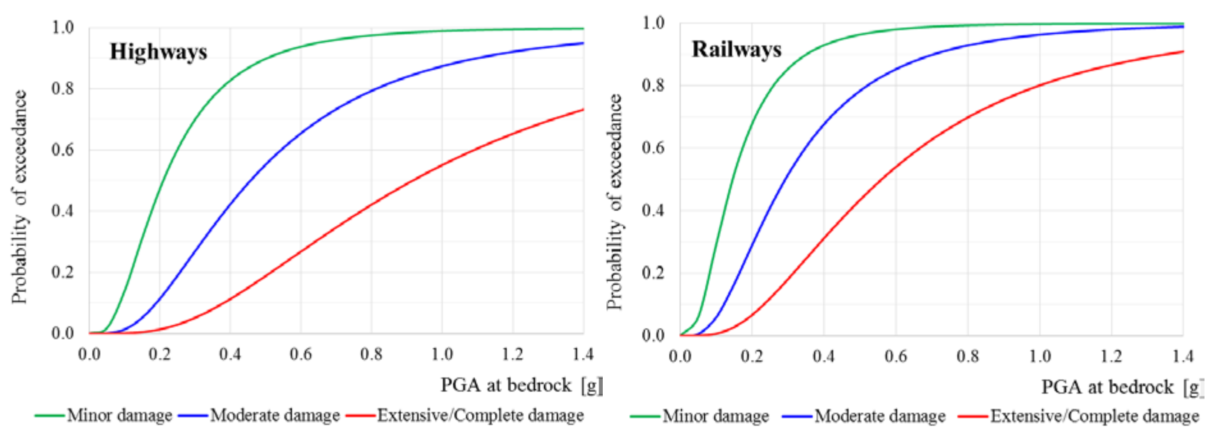


Figure 4. Fragility curves for highway and railway integral bridge abutment-backfill system.

CONCLUSIONS

This paper concludes that there is a substantial increase of research efforts on the vulnerability and risk assessment of transportation infrastructure against earthquakes and other natural 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 SoA, subjected to seismic excitations based on 2D coupled non-linear dynamic analysis. The novel element in that case is the fact that the model contains both the entire together and the two backfills as opposed to previous studies where the backfill is modelled by simple approaches (e.g. springs). The results showed that the response of the backfill may vary significantly for different input motions (e.g. duration, frequency content, seismotectonic environment). Also, the lower tolerance of railway assets to deformation resulted in higher vulnerability compared to highways.

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