1η ΠΡΟΚΗΡΥΞΗ ΕΡΕΥΝΗΤΙΚΩΝ ΕΡΓΩΝ ΕΛΙΔΕΚ ΓΙΑ ΤΗΝ ΕΝΙΣΧΥΣΗ ΜΕΤΑΔΙΔΑΚΤΟΡΩΝ ΕΡΕΥΝΗΤΩΝ/ΤΡΙΩΝ

Title of Proposal

Seismic reliability of railway bridges under high-speed trains using a novel formulation for constrained dynamical systems

Delivelable 1.2

State of the art review considering available methodologies for earthquake reliability assessment of railway bridges (fragility curve development) and life cycle cost-benefit analysis.

Work Packages 1.3, 1.4

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1 GENERAL

The main target of **WP1** is to provide a detailed state-of-the-art review considering both mathematical description of the mechanical problem and earthquake reliability as well as life cycle cost benefit analysis of railway bridges. All drawbacks of existing methodologies will be analytically described and discussed, as well as their effect on bridge analysis and decision making, outlining the need for the new methodology and its expected impact. Regarding seismic reliability, methodologies with and without high-speed train and bridge interaction will be reviewed, highlighting the need for a new holistic methodology for the fragility estimation of the coupled system.

Deliverable **D.1.2** is related to **WP1.3** and **WP1.4** and aims to provide a detailed state-of-the-art report regarding all existing methodologies for fragility and life cycle cost-benefit analysis of railway bridges with and without high-speed train and bridge interaction, highlighting the need for a new holistic methodology for the fragility estimation of the coupled system. In particular, state of the art review regarding earthquake reliability of railway bridges and bridge damage state definition with and without bridge and high speed train interaction (**WP1.3**) is provided in §3 and state of the art review regarding life-cycle cost benefit analysis of railway bridges (**WP1.4**) is provided in §4.

2 INTRODUCTION

2.1 General

The investment planning and decision-making for an upgrade and expansion of existing roadway and railway networks is based on detailed technical studies accounting for various social and economic parameters. The decision-making procedure includes assessment of critical infrastructure (e.g., bridges) and evaluation of development potentials. In this context, a robust and reliable analysis of the coupled train-railway-bridge system can provide a valuable tool for assessment of the existing network considering the effects of high-speed train and bridge interaction during dynamic (i.e., seismic) loading.

The dynamic behavior of a train-bridge system is a coupled and complex model consisting of two main subsystems, the bridge and the train. In general, the train subsystem can be described as a multibody assembly and the bridge subsystem can be modeled using classical structural finite element formulations. The subsystems interact through contact/impact forces, i.e., forces between the vehicle wheels and the rail on the bridge deck. Due to the nonlinear interaction of subsystems and the resonance existance, it is widely accepted that the dynamic analysis of train-bridge system during earthquakes cannot be a simple combination of bridge seismic design and train-bridge interaction calculation. Therefore, several studies have investigated the response of coupled systems under dynamic (earthquake) loading and a detailed state-of-the-art review can be found in Delivelable 1.1 (D1.1).

Bridge fragility curves are essential for the estimation of the road systems' resilience, recovery planning, as well as pre- and post-earthquake retrofit prioritization, strongly related to government investment and decision-making. Seismic fragility is the quantification of the probability that bridge damage will exceed a specific limit state threshold for a given level of earthquake intensity and can be used for pre-earthquake retrofit prioritization. Both analytical and empirical fragility curves were proposed by various research groups, the latter being less frequent (e.g. Basoz & Kiremidjian, 1999) since earthquake damage data for bridges is sparse. The methodologies available in literature are mainly referring to roadway bridges, however there are several (yet limited) methodologies extended to railway bridges. The main aspects of the methodologies available for the assessment of the seismic performance for different levels of earthquake intensity and the development of fragility curves of **railway** bridges are critically reviewed, namely:

- The dynamic (earthquake) analysis of the complex system, with or without moving mass consideration.
- The critical components considered and damage state definition for the coupled bridge-train system.
- The probabilistic framework for the fragility curve development

The applicability of the methodologies to railway bridges under high-speed trains is also discussed. Finally, state of the art review regarding life-cycle cost benefit analysis of railway bridges is also provided.

3 SEISMIC RELIABILITY OF RAILWAY BRIDGES: STATE-OF-THE-ART REVIEW

3.1.1 Earthquake Damage to Railway Bridges

According to Byers (2004), the behavior of railway bridges in earthquakes varied widely. There was no railroad bridge damage recorded during the Mw 7.5 Kern County, California, the Ms 7.8 Kocaeli, Turkey or the Mw 8.4 Atico, Peru earthquakes. In all 3 of these earthquakes, there was severe damage to other railroad facilities in the immediate vicinity of bridges. On the other hand, bridges were damaged in 48 of the 91 earthquakes, with severe to extreme damage in 25 and moderate to significant damage in an additional 17. In 40 of the 48 earthquakes for which bridge damage was reported, another railroad damage was also reported. The type and extent of bridge damage was strongly related to design details, foundation conditions and liquefaction potential at the bridge site. In the 1995 Kobe earthquake, there was extensive collapse of concrete rigid frame viaducts up to 45 km from the epicenter but a significantly smaller distance from the fault rupture. This appeared to be attributed to inadequate ductility of the viaduct columns. Damage due to ground movement were also recorded. The Southern Pacific Railroad bridge over the Pajaro River had recorded pier movement up to 3.5 ft. (1.2 m) in the 1906 California earthquake, due to movement along the fault rupture which rotated them at an angle to the bridge of about 45 degrees. Movement of an end span relative to the abutment is shown in (Figure 1)

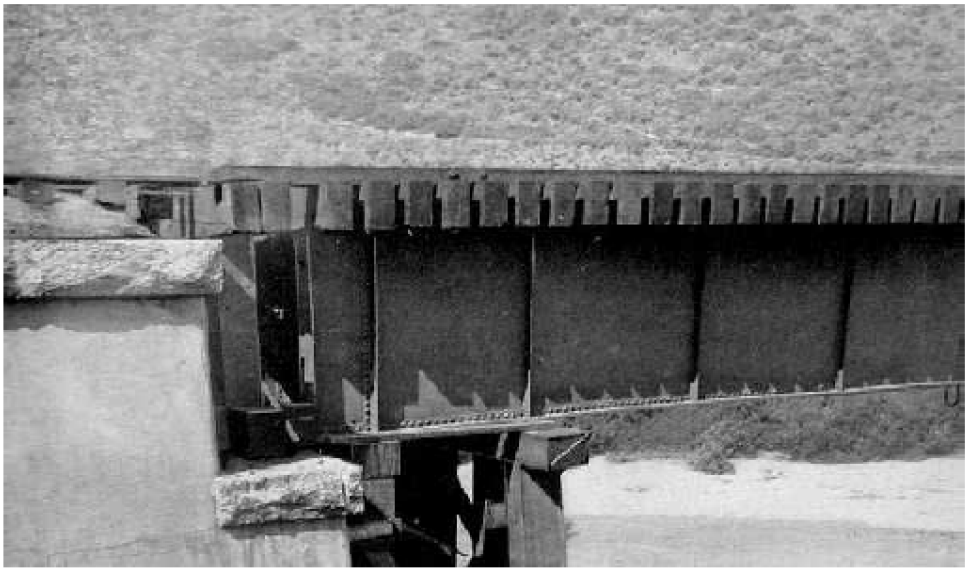


Figure 1 – Span displacement of railway bridge [Byers (2004)]

Earthquakes are a serious threat to moving high-speed trains, especially for those moving on bridges, since bridges may significantly magnify the seismic load due to the resonance that may occur among bridges, trains, and earthquakes. Montenegro et al., (2016) outlined that derailment is a frequent failure mode of high-speed railway bridges subjected to earthquake loading. Two recent examples are mentioned, namely the derailments that occurred during the Kobe Earthquake in January 1995 and the Shinkansen high-speed train derailment at 200 km/h during the Niigata Earthquake in October 2004, while an overturn may also occur e.g. 1999 Kocaeli Turkey earthquake. A moderate earthquake, i.e. the Jiasian-earthquake on March 4, 2010 in southern Taiwan resulted in the derailment of a high-speed train, for a maximum ground acceleration near the derailment location equal to 0.17 g. (Ju, 2013). Furthermore, at Mid Niigata Prefecture Earthquake in October 2004, a Shinkansen train derailed while being operated at the speed of 200 km/h, which was the first case of the derailment of a Japanese high speed train under commercial operations through its long history. Since bridges are often built for high-speed trains, avoiding train derailment on bridges is an important issue that deserves additional attention.

Earthquake damage, serviceability and repair time & cost should be related in order to prioritize retrofit and be utilized for decision-making. Regarding the recovery of a railroad from a significant earthquake, Byers (2004) outlines that it depends on the severity and extent of damage, the resources available for repair and the urgency of restoring service. As a minimum, inspection to ensure the safety of the track and related systems is required after moderate and larger earthquakes in the vicinity of the railroad. This typically prevents normal operation for 5 hours or more although undamaged lines were returned to operation within 3 hours after the 2001 Gujarat, India earthquake due to a sizeable number of strategically located inspection personnel. Where operation is not possible, partial repairs necessary to allow restricted operation are typically made as rapidly as possible, with permanent repairs for normal operation completed after limited service is restored. After the 2001 Gujarat, India earthquake, some bridges required temporary repairs before they could carry traffic. At others, trains were required to stop before crossing the bridge and to cross at a specified slow speed until repairs were completed. Following the 1952 Kern County, California earthquake, which caused extreme damage to several tunnels, a temporary shoofly with undesirable alignment and grades was opened within 4 weeks and permanent repairs were completed after 21 weeks. After the 1995 Kobe earthquake where there was extensive collapse of long viaducts in a metropolitan area, operation over temporary facilities was not an option but alternate bus service was provided for passengers during a 23-week reconstruction period. Following the 1999 Chi Chi, Taiwan and 2001 Gujarat earthquakes, main lines were restored to service considerably before branch lines. When only part of a railroad is in the affected area and the damage is extensive, personnel and equipment are usually brought in from other areas to expedite repairs.

3.1.2 Dynamic Analysis of Train-Bridge System Subjected to Earthquake Action

High-speed railways play a growing important role in solving traffic problem between major cities and promoting further economic and social development. In high-speed railway lines, more and more bridges including viaducts are built in order to reduce the influence of railway lines on the existing built environment between major cities. The number of trains running on a railway line is also increasing because of high speed. As a result, the probability that an earthquake occurs when a train is running over a bridge in earthquake-prone regions is much higher than before ever. Dynamic analysis of coupled bridge–train systems during earthquakes becomes imperative for the safety of vehicles and human lives, which has been confirmed by the accident that a Taiwan high-speed train running on the elevated bridge at 250km/h derailed during Tainan earthquake on 4 March, 2010. (Du et al., 2012).

Seismic resistance of bridge structures is an issue of great concern in many countries, especially those located in earthquake-prone regions. As for railway bridges, it is possible that the bridge itself may remain safe during an earthquake but may not be safe enough for the trains to move over it due to excessive vibrations. Evidently, the safety of moving trains over the bridge under earthquake excitations is a subject of great concern in railway engineering (Yang et al., 2010). As stated in Ju, (2013), a number of researchers proposed linear numerical simulations to study the dynamic behavior of moving trains subjected to earthquakes (Xia et al. (2006), Yang & Wu (2002)), while several researches combined theoretical and numerical methods to achieve this. In cases that the separation mode of the rail and wheel was not considered (i.e. the derailment case), the results are suitable for small or moderate seismic loads (Fryba, (2001), Zhang et al., (2015)).

Yang & Wu (2002), outlined that unlike seismic analysis of structures containing a single subsystem, the seismic analysis of a bridge sustaining a passing train requires information about acceleration, velocity and displacement of the ground motion. They studied the stability of trains on bridges under seismic excitations, accounting for the effect of the track system. The bridge was considered elastic, whereas some simple criteria, such as derailment index, were adopted for evaluation the safety of moving train (acceptable range of Q=H/V ratio where H is the lateral and V is the vertical force). Xia et al. (2006), proposed a model for the dynamic analysis of train-bridge system subjected to non-uniform seismic excitations, based on the theory of dynamic wheel-rail interactions. The bridge model is based on the modal comprehension analysis technique while the seismic loads are imposed on the bridge by using the influence matrix and exerted on the vehicles through the dynamic wheel– rail interaction relationships. The influences of train speed and seismic wave propagation velocity on the dynamic responses of the bridge–vehicle system are studied and critical train speeds are proposed for running safety on high-speed railway bridges under earthquakes of various intensities. Considering the dynamic response of train vehicles, the lateral car body accelerations, derailment factors, offload factors and lateral wheel–rail forces were all found to increase with the train speed. Therefore, it was concluded that the influences of train speed should be taken into account in order to evaluate the running safety of vehicles on the bridge during earthquakes.

Resonance vibrations have been observed on railway bridges subjected to high speed trains, however, a few studies performed train nonlinear or derailment analyses under seismic loads. An elementary theoretical model of a bridge was investigated by Fryba, (2001), using the integral transformation method which provides an estimation of the amplitudes of the free vibration. The analysis provided the critical speeds at which the resonance vibration may occur, caused by two reasons: repeated action of axle loads and high speed itself. Nishimura, et al., 2009 studied vehicle safety in terms of the dynamic stability and the possibility of derailment directly caused by track excitations during earthquakes, and four major outcomes for train derailment were obtained. Furthermore, it has been shown that modifications in structural characteristics of bridges may result in safety increase of moving train during earthquakes, i.e. larger pier stiffness results in smaller derailment coefficients. Ju, (2012), investigated the derailment of high-speed trains moving on multispan simply supported bridges. When a high-speed train moves on the multispan simply supported bridges, the derailment coefficients were found to enlarge with the increase in train speed. When the train is very fast, such as over 350 km/h, the derailment coefficients were greatly increased. This was due to the fact that a certain rail was almost disconnected from the wheel during a period of time. It was also indicated that the bridge system may significantly magnify the seismic load, due to a resonance between bridges and earthquakes. Another critical issue was that gaps between simply supported girders during seismic loads may produce large derailment coefficients. (Ju, 2012)

Zeng & Dimitrakopoulos, (2018), proposed a seismic vehicle-bridge interaction analysis method that simulated directly different wheel-rail contact states including flange contact, detachment, uplifting, wheel-rail climping up, recontact and ultimately, derailment. The proposed model determines the contact point and the direction of the contact forces over practical nonlinear profiles of wheels and rails. It then classifies the wheel–rail contact, as double contact, single contact or double detachment, and tackles accordingly the kinematics.

The proposed vehicle-bridge interaction scheme is presented in Figure 2. The authors have also explored the derailment mechanism of trains running over bridges during strong earthquakes (Zeng & Dimitrakopoulos, 2017), using the proposed contact model to capture wheel-rail phenomena as flange contact, detachment, uplifting and recontact.

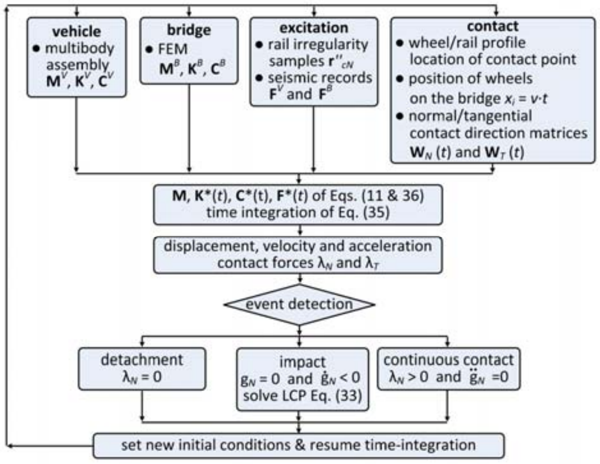


Figure 2 – Vehicle-Bridge Interaction Scheme [(Zeng & Dimitrakopoulos, 2018)*]*

Du et al., (2012) presented a finite element method-based framework for dynamic analysis of coupled bridge–train systems under non-uniform seismic ground motion, in which rail–wheel interactions and possible separations between wheels and rails are taken into consideration. The equations of motion of the coupled system were formed in terms of displacement seismic ground motions, without considering the decomposition of seismic responses into pseudo-static and inertia-dynamic components, while the mode superposition method is applied to the bridge structure to make the problem manageable. Based on the case study results, they concluded that the ignorance of pseudo-static component when using acceleration seismic ground motions as input may underestimate seismic responses of the bridge–train system, whereas the opposite may underestimate the separation number and time duration compared with the displacement seismic ground motion inputs.

Very recently, Montenegro et al., (2016), studied running safety of trains moving over bridges subjected to moderate earthquakes with relatively small return periods and high probability of occurrence. A nonlinear train-bridge interaction method was proposed by the authors and the influence of the seismic intensity levels on the train running safety, train running speed, and track quality was evaluated. Since no significant nonlinearity was expected to columns for moderate levels of seismicity, the analyses were performed in the elastic domain, however, the reduction in the columns stiffness due to cracking was accounted for. Furthermore, the randomness of the earthquake was accounted for, considering different time offsets between the beginning of the earthquake and the entry of the vehicle in the viaduct. It was found that even for the moderate seismic intensities considered in the present study, the train safety is put at risk in a considerable number of scenarios, especially because of the risks of derailment caused by wheel flange climbing.

Finally, it should be mentioned that research studies related to the improvement methods to increase the safety of moving trains during earthquakes are available. In particular, Ju, (2013) investigated the bridge improvements required to increase the safety of moving trains during earthquakes, using finite element analyses with nonlinear moving wheel elements to simulate the contact and separation modes of rails and wheels. Based on case studies analyzed, it was found that large gaps between two simply supported girders during earthquakes will increase the train derailment coefficient and thus a reduction in the eccentricity between two girders can enhance the safety of moving trains. Furthermore, smaller pier stiffness was found to produce larger train derailment coefficients, since the first natural frequencies of trains are often in the low frequency range. To this end, large pier stiffness is suggested to increase the safety of moving trains during earthquakes.

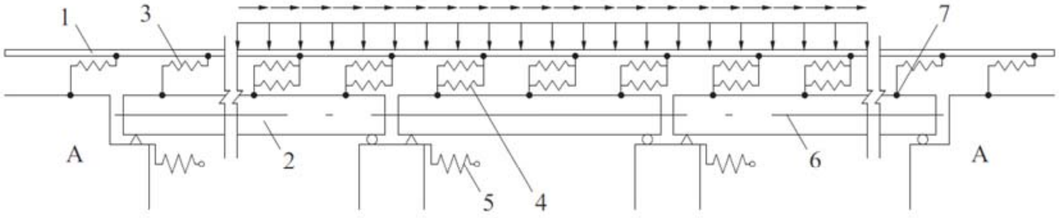
3.1.3 Methodologies for the development of fragility curves of railway bridges

In order to assess the seismic performance of existing bridges, fragility curves are proposed and developed, quantifying the probability of the seismic demand to exceed the systems' capacity for different levels of earthquake intensity. Numerous methodologies are available in literature for the development of bridge fragility curves, mainly analytical, since earthquake damage data for bridges is sparse. The analytical methodologies can be classified based on whether they consider multiple components (Mander & Basöz (1999), Nielson & DesRoches (2007a), Moschonas et al. (2008), Zhang et al. (2008), Tsionis & Fardis (2012), Stefanidou & Kappos, (2017)) or only the most critical one (piers) in fragility analysis (Banerjee & Shinozuka, 2007); classification can also be based on the procedure for estimation of component or system capacity (limit state thresholds) and seismic demand (analysis method used), the uncertainty treatment and the probabilistic model used. Specifically, regarding component capacity, either local (Avşar et al. (2011), Tsionis & Fardis (2012), Choi et al. (2004)) or global (Shinozuka et al. (2000), Cardone (2013)) demand parameters are used, while quantification of damage, namely the limit state thresholds, is commonly based on experimental results (Dutta & Mander, (1998), Berry & Eberhard, (2003), HAZUS, (2015)). Regarding the estimation of seismic demand, different analysis methods have been put forward, namely inelastic static (pushover) analysis (e.g. (Cardone et al. (2007), Moschonas et al. (2008)), elastic response spectrum method (e.g. (Mander & Basöz (1999), HAZUS (2015)) and nonlinear response-history analysis (Mackie & Stojadinović (2007), DesRoches et al. (2012)). The maximum likelihood method (Shinozuka et al. 2000) or the probabilistic seismic demand model (Nielson & Desroches 2007b) have been used for the derivation of fragility curves. The way capacity and demand estimation is made in the frame of analytical methodologies for the derivation of fragility curves is summarized in Stefanidou & Kappos, (2019).

Even though many methodologies are available for the seismic fragility analysis of highway bridges, few of them are extended to railway bridges. The most critical issues of the methodologies in order to be extended to railway bridges are:

- The modeling of the bridge-track system and consideration of vehicle-bridge interaction.
- The quantitative and qualitative damage state definition of the coupled system and the engineering parameters selected.
- The consideration of contact phenomena of the wheel-rail, crucial for failure modes (i.e. derailment, etc)

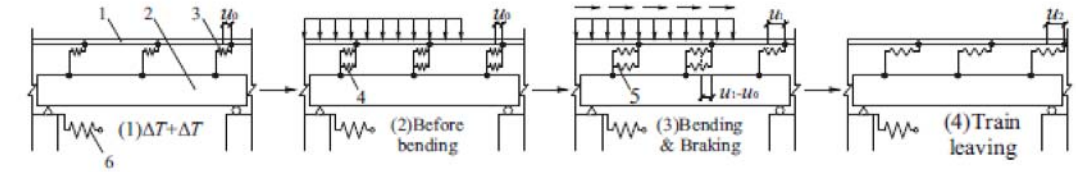
The effect of vehicle-bridge interaction consideration on the dynamic and seismic response of bridges has been extensively discussed in §3.1.2. Furthermore, it has been established (Ghosh et al., 2014) that the impact of traffic loads on the seismic fragility of bridges is important and should be considered in the frame of a joint seismic and live load assessment. It should be outlined that Zhang et al., (2015), proposed a nonlinear TBI model allowing for loading history effects (Figure 3). Two major advancements are achieved in this model compared with the conventional model. First, all of the loads are imposed on the track–bridge system in an actual sequence, and therefore the nonlinear behavior of the fasteners and interactions of the various loads are fully considered. Second, a double-spring method is proposed to represent the changes in fastener resistance during the loading histories, and to modify the resistance.



1. Track; 2. Bridge; 3. Nonlinear spring modelling the unloaded fastener
4. Additional nonlinear spring; 5. Linear spring modelling the piers and abutments;
6. Brigde centroid; 7. Fastener nodes located at the bridge deck; A. Embankment

Figure 3 – Schematic planar model for the proposed TBI model allowing for loading history effects, Zhang et al., (2015)

The double-spring model including the unloaded and additional springs is proposed to better simulate the behavior of the fasteners before and after the train leave. Their use is schematically presented in Figure 4 and the steps are summarized as follows: (1) activate the unloaded springs at the initial stage and calculate the responses of the TBI system under thermal variations, (2) activate the additional springs without initial forces and maintain the balance state of the system, (3) calculate the bending- and braking-induced responses of the TBI system with the combined stiffness of both springs and (4) deactivate the additional spring when the train leaves the bridge and causes a rebalance of the system.



1. Track; 2. Bridge; 3. Nonlinear spring modelling the unloaded fastener 4. Additional nonlinear spring activated without initial strain;
5. Additional nonlinear spring in synchro-deformation with the unloaded spring; 6. Linear spring modelling the piers and abutments;

Figure 4 – Schematic model of double-spring model, Zhang et al., (2015)

Several researchers were involved with modeling of critical parts in order to consider vehicle-bridge interaction in seismic vulnerability analysis. Wei et al., (2018) proposed the system of Figure 5, considering in detail the track structure and almost all of the potentially damaged components that are simulated by non-linear elements in the track-bridge FE model.

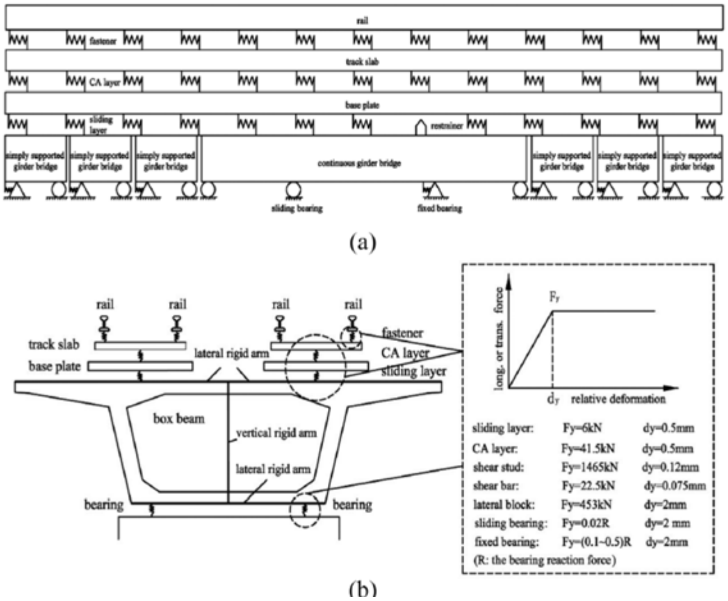


Figure 5 – Finite element model of the truck structure Wei et al., (2018)

Regarding limit states for running safety of train during earthquakes, details can be found in Xia, et al., (2018). The Railway Structure Design Standards and Interpretation—Aseismatic Design (RTRI 1999) in Japan proposed the limits for the relative displacement between piers and abutments as well as for the deformation fold angle of track on the bridge. However, the derailment of train results from the interaction between the train and the track, thus even if the train and the track satisfy their respective safety criteria, the coupling effect of them under some special conditions may also lead to the derailment. The Nadal equation (Nadal 1896) based on static analysis of wheel derailment is widely used in the safety evaluation of trains. In UIC Code 518 (2003) and EN 14363 (2005), the safety limit for the derailment factor is Q/P=1.2, and in the experimental standard for German ICE train, it is Q/P= 0.8. In American standard (AAR 2011), the limits are separately set for a single axle and the whole axle (Wienstock limit), expressed as

$$Q/P \leq 1.0 \text{ (for a single axle)}; \ \sum Q/P \leq 1.5 \text{ (for the whole axle)}$$

In China, there is no special standard for evaluating the running safety of high-speed trains under earthquakes, so the general running safety indices are adopted. What should be mentioned is when a train runs on the bridge during earthquakes, the derailment factor, offload factor, or lateral wheel–rail force may exceed the limits, and even occurs the separation between the wheel and the track, however, a safety index exceeding the limits in a very short time does not definitely mean the derailment, which has been verified by derailment test (Zhang et al. 2008). The JNR (Japanese National Railways) standards classified the train derailment into cases of gradual climbing-up rail and dynamic jumping-on rail, and proposed the evaluation standards for derailment considering the acting time of lateral wheel–rail force, expressed as

$$Q/P \leq \begin{cases} \lambda & t \geq 0.05\,\mathrm{s} \\ \frac{0.05}{t}\lambda & t < 0.05\,\mathrm{s} \end{cases}$$

where t is the acting duration of lateral wheel–rail force; k is the target value of the derailment factor, with k = 0.8 and k = 1.0 corresponding to the dangerous limit and maximum tolerance limit, respectively.

In the JNR standard, the tolerant Q/P gets larger with smaller action time t, which is not strict enough and may lead to unsafe results. In this regard, the Japan Railway Technical Research Institute proposed a new criterion for evaluating derailment, namely the duration of Q/P exceeding 0.8 should not longer than 0.015 s. Besides, the JNR standard stipulates that the static limit for offload factor is 0.6 and the dynamic limit is 0.8. There exists a dynamic geometrical restriction between wheel and rail, which ensures the normal running of the train on the track, while the break of this restriction leads to the derailment. Zhai and Chen (2001) simulated the process of climbing-up derailment and jumping-on derailment of a single wheel based on the train–track coupling dynamics theory and proposed the evaluation criteria for freight car derailment according to the geometric status of wheel–rail contact, expressed as

$$Q/P \leq \begin{cases} 1.0 & (t \geq t_0) \\ 1.0 \times \frac{t_0}{t} & (t < t_0) \end{cases}; \quad \begin{cases} \frac{\Delta P}{P} \leq 0.60 & (t \geq t_0) \\ \frac{\Delta P}{P} > 0.60 & (t < t_0) \end{cases}$$

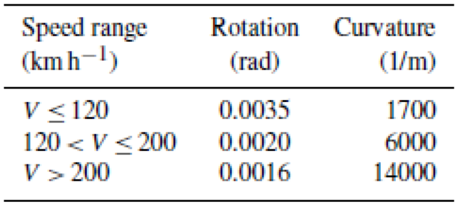
where t is the sustaining duration of lateral wheel–rail force, or the time of offload factor exceeding the target value; t0 = 0.035 s is the maximum tolerant time for derailment factor or offload factor exceeding the limit. Compared with freight trains in normal running status, high-speed railway trains are greatly different in running speed, track irregularity standard, vehicle parameters, and many other aspects. As a result, the indices for freight trains cannot be directly used for the running safety evaluation of high-speed trains during earthquakes, which should be further studied.

Ju, (2012) proposed the use of the Western European standard Q/P < 0.8 with the average train movement distance of 2 m for investigation of the train safety, where Q/P is the maximum wheel derailment coefficient from all the train wheelsets.

$$Q/P = \mathrm{Max}(Q_i/P_i), \quad i = 1, n$$

Qi and Pi are the average horizontal and vertical forces of the ith wheelset during the 2-m movement of the train, while n is the total number of train wheelsets. In the numerical procedure at each time step, Qi and Pi are averaged from the forces of a certain number of time steps in which the train just moves 2 m during these time steps. It is also mentioned that the derailment coefficients of the multi-span continuous bridge (NC = infinite) are significantly smaller than those of other bridge types. This is because the earthquakes cause large sharp gaps between two simply supported girders, so that large derailment coefficients are generated when the train wheels pass these gaps.

The majority of the methodologies for fragility analysis of railway bridges were developed during the last the last decade. Park & Choi, (2011) developed fragility curves of track-on-steel-plate girder bridges in Korea. The methodology proposed was based on fragility of critical components (i.e. piers, bearings, abutments) and the definition of limit states and engineering demand parameters was not differentiated from relevant methodologies for highway bridge. Vehicle-bridge interaction was not considered within this methodology. However, it should be highlighted that a generic damage measure was defined, in order to describe the system damage, based on damage of critical components. They developed component fragility curves and showed that the bearings are the most vulnerable components against seismic loading, whereas the piers were found to be the less vulnerable one and finally system fragilities were derived based on generic damage measure. Park & Towashiraporn, (2014) used the response-surface statistical model to develop fragility curves of railway bridges, following a methodology identical to the one proposed for highway bridges. (Wang, et al., (2014), calculated seismic fragility of cushioning high-speed railway bridges, using a methodology similar to the ones available in literature for highway bridges, proposing the use of displacement as engineering demand parameter. Salcher et al., (2016), addressed the reliability analysis of high-speed railway bridges with stochastic methods to account for uncertainties in the mechanical model, emphasizing on ballasted steel bridges. Interaction of bridge and train was considered, while the equations of motion of the bridge and train subsystems derived independently and subsequently coupled via a linear interaction model. The acceleration response of the bridge deck is assumed to be the governing response quantity for bridge serviceability, and thus, defines in the reliability assessment the limit state of the bridge, therefore, the limit states considered were differentiated from the relevant for highway bridges (operational reliability was considered). Rail irregularities and their effect on dynamic response were also accounted for. Yilmaz & Çaglayan, (2018), performed seismic assessment of a multi-span steel railway bridge in Turkey, based on nonlinear time history. The methodology followed is a component-based methodology, based on the probabilistic seismic demand model proposed for highway bridges as well. Since seismic action may cause train derailment or even overturn, e.g. 1999 Kocaeli Turkey earthquake (Byers, 2004), they also considered serviceability, considering lateral displacement as the serviceability limit state. EN1990-Annex A2 includes lateral displacement limits for railway bridges (EN1990-prANNEX A2, 2001), including maximum angular variation and minimum radius of curvature to limit lateral displacement for different velocities, as shown in Table 1.



Speed range ($km\,h^{-1}$)	Rotation (rad)	Curvature (1/m)
$V \leq 120$	0.0035	1700
$120 < V \leq 200$	0.0020	6000
$V > 200$	0.0016	14000

Table 1 – Engineering demand parameters; Serviceability limits Yilmaz & Çaglayan, (2018)

Very recently, Bellotti et al. (2019) proposed a methodology for fragility curve development and assessment of R/C railway bridges, introducing an application to define fragility curves in the framework of large scale vulnerability assessment. In order to evaluate the overloads associated with the transit of trains and the part of them to be used for seismic checks, the adaptation coefficient and dynamic coefficient were used for the estimation of the uniformly distributed load. The limit states considered in the calculation of fragility curves vary in relation to structural elements and are: 1) for piers: the ductile and fragile mechanisms. The ductile mechanisms depend on the total chord rotation amount while the fragile ones depend on the element shear. 2) for decks: the loss of support mechanism towards the supports or the pier cap, controlled by the relative excursion of the support devices. 3) for supports bearings the achievement of the maximum capacity in terms of resistance. For all the limit states considered, two levels of capacity are foreseen during the verification: one related to the necessity to carry out repair interventions (damage limit state) and the other concerning the safety of the bridge (collapse limit state). An exception to this rule is the fragile shear mechanism on the piers, for which only the collapse limit state is defined. The operation limit state is, instead, linked to the deformations limit defined in relation to the train's safe transitability of the bridge and the limits imposed are not connected with the damage to the structures. The amount of demand for determining the risk related to the loss of support mechanism isthe relative excursion between super-structure and sub-structures for each alignment of supports, equal to the average deformation of the bearings of each alignment. For girder railway bridges the application generates fragility curves, defined by cumulative probability distributions that allow to estimate the probability of reaching or exceeding a given level of damage for a given severity of ground shaking, for each boundary state. In addition, very recently Cui et al.,(2019) performed seismic fragility and risk assessment of high-speed railway continuous girder bridge, under track constraint effect. The methodology proposed is based on IDA and probabilistic seismic demand model, while the estimation of system fragility is based on component fragility. On the basis of the damage criterion for deformation, in this study, the curvature of the pier bottom and the sliding displacement of the moveable bearing were defined as the damage indexes, and the damage states were quantified. Based on the track-bridge longitudinal interaction model and the considered nonlinear behavior of bridge piers and bearings, they studied the seismic fragility and risk assessment of a high-speed railway pre-stressed concrete continuous-girder bridge under ground motions. The probabilistic capacity model was defined using the damage limit state while the probabilistic demand models were developed based on bridge responses under earthquakes with different intensity levels. Then, the component fragility curves were derived and compared with that of the bridge model without the track system. The bridge system fragility curves were derived using the first-order bound reliability method. The fragility and failure probability of bridge were assessed the seismic performance was evaluated.

Based on the above, the need for a new, holistic methodology for the fragility estimation of the coupled train-railway bridge system is outlined, emphasizing on the limit state definition and the dynamic analysis on a coupled system level.

4 LIFE-CYCLE COST BENEFIT ANALYSIS OF RAILWAY BRIDGES: STATE-OF-THE-ART REVIEW

As a fundamental infrastructure in transport networks, railway bridges are responsible for numerous material and energy consumption through their life cycle, which in turn leads to significant environmental burdens. However, present management of railway bridge infrastructures is mainly focused on the technical and financial aspects, whereas the environmental assessment is rarely integrated. **Life cycle assessment** (**LCA**) is deemed as a systematic method for also assessing the environmental impact of products and systems, but its application in railway bridge infrastructures is rare. Very limited literature and research studies are available in this area. In order to incorporate the implementation of LCA into railway bridges and set new design criteria Du & Karoumi (2014) performed an elaborate literature survey presenting current developments regarding the LCA implementation for railway bridges. Several critical issues are discussed and highlighted, focusing on the methodology, practical operational issues and data collections. A systematic LCA framework for quantifying environmental impacts for railway bridges is therefore introduced and implemented to a railway bridge in Du & Karoumi (2013).

Served as a systematic tool, LCA has been widely applied in the industrial fields of production, agriculture and building service, but very rarely for the railway bridge infrastructures. The railway bridge management is still mainly focused on the technical, safety and economic perspectives without considering the environmental impact. It has been noticed that the LCA for railway bridges is still new, lacking internationally agreed guidelines and criteria. There are some limited literature and research studies available for the LCA of roadway bridges, but very few for railway bridges, while the implementation of the LCA approach in roadway or railway bridge infrastructures is very scarce. Due to limited research, most of the case studies are done without following a generally accepted methodology or framework, whereas they only emphasise on a few emission types and part of life cycle.

In order to provide a generalized **LCA framework of railway bridges** to the practitioner and decision-maker, Du & Karoumi (2014), explicitly reviewed the current available LCA studies for bridge structures, including 14 for roadway bridges and 4 for railway bridges, with the intention to partially combine the LCA knowledge from the roadway bridges with railway bridges. The railway bridges differ from the road bridges in several aspects, including the structural component, construction technique, maintenance and EOL scenarios. A systematic LCA framework was developed and suggested for modelling the whole life cycle of the railway bridge infrastructures that can be implemented as a guideline, either for the whole railway bridge or for a specific life cycle stage or part of the structural components. Each bridge element is covered from the railway track to the superstructure and substructure, with the components associated with a certain material type. The LCI (life cycle inventory) data with the detailed manufacture procedures are linked with the selected material. The selected LCIA (life cycle impact assessment) method is further assigned to the inventory data in accordance with the ISO standards. The results can be presented in terms of specific impact damage indicators for the human health, ecosystem and resource depletions. The recommendations of a broad set of specific life cycle stages for the railway bridge are described in detail in Du & Karoumi (2014) and the phases that should be considered are:

- **Material Manufacture phase**: It has been found that the final environmental performance largely relies on the selection of material types, which is a key factor further affecting the necessary consumption quantity and on-going maintenance schedules. The embodied environmental profile of each material is dominated by the constituted raw materials, manufacture technology and the supply chains. Each of these mentioned processes can be illustrated by a long list of LCI data.

- **Construction phase**: The construction technique adopted is related to different energy efficiency in the construction machine, affecting the environmental performance. The construction phase focuses on a wide range of operational systems including energy consumption from the construction machinery, establishment of associated scaffoldings and supporting systems.
- **Maintenance and use phase**: The maintenance and use is longest life stage, responsible for a large proportion of environmental burdens due to replacing of structural components and related traffic disturbances. The realistic maintenance or repair activities such as structural strengthening and component replacement are influenced by the design type, service life, train loading, infrastructure durability, periodic inspection and the budget plans. Due to the uncertainties, a further sensitivity analysis is imperative for testing the influence from the significance of each scenario. Different design solutions also affect the maintenance scenarios, which further influence the environmental performance. Due to the single-track design, most of the maintenance activities require a traffic closure that cause extra environmental burdens. The high-quality materials have been proved to efficiently prolong the service life and improve the environmental performances in a long term.
- **EOL (end of life) phase**: This phase concentrates on the energy consumption from the demolition, recycling processes and involved transportations. With an attempt to model the future waste treatment scenarios based on today's technologies, the EOL covers a series scenarios of bridge demolition, waste sorting, material reuse or recycling, incineration and final landfill. In general, the material recycling and waste treatment in the EOL stage are expected to benefit the environment, in terms of producing the co-products and energy, recycling and reuse of materials. Concrete, aggregate, reinforcement and steels are the basic materials in bridges, from which the metal of ferrous iron, zinc and aluminium are 100% recyclable without losing original properties. From the construction plate and beams, the steel recycling rates were up to 88%. The environmental benefits due to the steel recycling during the processing can be quantified by the avoided burden method. Besides, the concrete is commonly crushed and reused as lower quality aggregates in road, whereas the aggregates can be either reused or crushed into the backfills if not contaminated. The selection of EOL strategies is imperative for the final environmental performance of the bridge, which may potentially eliminate environmental burdens.

Based on the literature review, the most detailed framework for LCA of railway bridges is outlined in in Du & Karoumi (2013) (Figure 6). The model takes account of the bridge structural elements including the railway track, superstructure and substructure; each of them is connected to a specific material type.

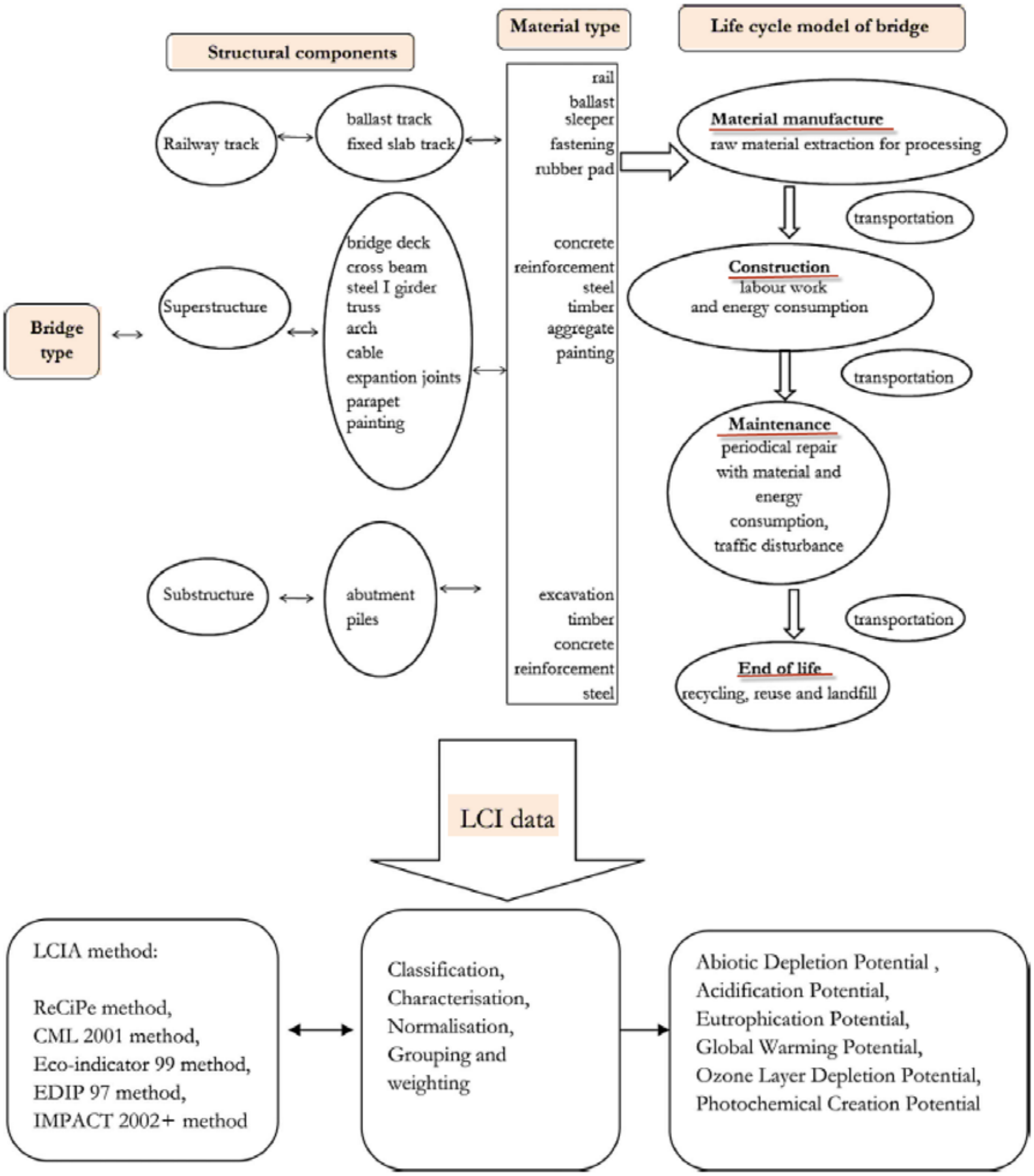


Figure 6 – Methodology for LCA of railway bridges - Du & Karoumi (2013)

The framework proposed is applied in order to comparatively evaluate the environmental performance evaluator of two design options of a railway bridge (designed with ballast track and fixed-slab track), considering the four life stages (phases) described above, namely material manufacture, construction, maintenance and EOL stages. The software used for LCA was SimaPro (vers.7.0) The material manufacture stage includes the entire raw resource and energy flows for the material processing. The related environmental burdens are obtained from the selected LCI database. The construction phase considers the diesel and fuel burned in the construction machine, while the transportation and labor work are omitted due to lack of information the maintenance phase focused on the scheduled periodic renewal of the structural components as well as the goods transportation. Usually, if a small part of the structural components needs a replacement on site, the whole component will be replaced at the same time, thus the same material and energy flow are assumed as in the initial construction stage. The traffic disturbance due to the maintenance activities are considered separately in the sensitivity analysis. At the EOL stage, the bridge will be demolished and sorted for different waste treatments. The current steel recycling rate for the construction plate and beam is considered to be 88%. Based on those considerations, several EOL scenarios are assumed, that the concrete is modelled by crushing into gravel and disposal for landfill, the steel is modelled by 88% recycling and 12% landfill. All of the material and energy inventories involved in those activities are obtained from LCI database.

The comparative LCA analysis is focused on the whole life cycle of two bridge design alternatives, from the material manufacture phase, through construction phase, use and maintenance phase, till the end of the life with a life span of 120 years. The study covers the railway track system, bridge slab and steel I-girder beams. However, the bridge substructure, which assumes to be identical for both design options, is excluded from the study. The functional unit is chosen as 1 m unit length of the bridge system in the longitudinal direction, serving the same annual traffic volume with a life span of 120 years. This bridge is originally designed with ballast track, however, due to the improved railway efficiency, Gillet (2010) did an alternative design with fixed-slab track option for the whole superstructure based on the static and dynamic test. The main body of the bridge consists of a reinforced concrete deck supported by two steel I-girder beams. The two design alternatives are differed from the railway track systems, bridge slab and the main steel I-girders. The results of LCA of the design alternatives are presented in Figure 7 (ballast and fixed slab option).

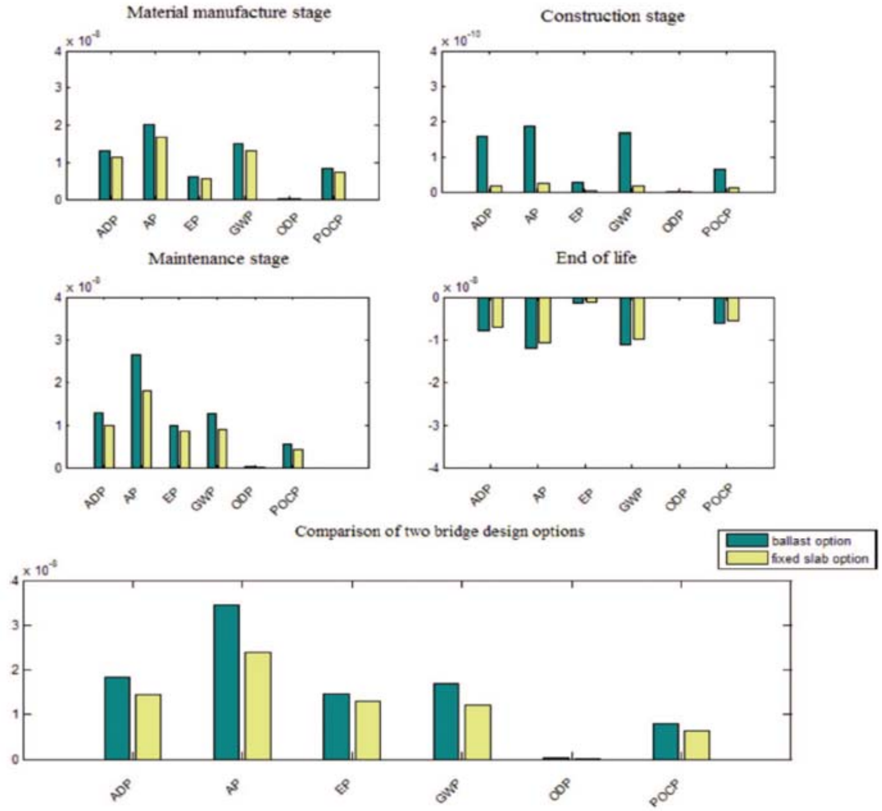


Figure 7 – Environmental comparisons between the two bridge alternatives through the whole life cycle - Du & Karoumi (2013)

Based on the above research, it is concluded that a change in life-cycle scenarios and the applied LCA method can greatly influence the final environmental results, since contradictory conclusions may be reached, dependent on the methodology followed for LCA. The structural type was found to affect life cycle scenarios, further influencing the final environmental impact, while the great uncertainties involved in LCA model were found to affect the results. Finally, the availability of data and project information were found to affect the LCA application and results.

5 CONCLUSIONS

Based on the literature review presented herein the conclusions are summarized below:

- Many analytical methodologies are available in literature for the seismic fragility assessment of bridges, however few of them are extended to railway bridges.
- There are limited methodologies available for the seismic reliability assessment of railway bridges accounting for the effect of moving high-speed train load on seismic fragility.
- The damage state definition of the coupled train-railway bridge system is the most critical issue, not mentioned in the methodologies in literature. Based on the parameters affecting the seismic performance of a railway bridge, the need for the definition of new damage states related to derailment is outlined. This is related to the dynamic analysis of the coupled system and monitoring of the engineering demand parameter selected.
- Limited methodologies are available for LCA of railway bridges with great uncertainties on application.

Based on the above, the need for a new, holistic methodology for the fragility estimation of the coupled train-railway bridge system is outlined, emphasizing on the limit state definition and the dynamic analysis on a coupled system level. Furthermore, it is outlined that limited methodologies are available for LCA of railway bridges with great uncertainties on application.

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