COMPDYN 2015
5th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, V. Papadopoulos, V. Plevris (eds.)
Crete Island, Greece, 25–27 May 2015

PERFORMANCE BASED EARTHQUAKE ENGINEERING APPROACH APPLIED TO BRIDGES IN A ROAD NETWORK

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Keywords: Performance based earthquake engineering, RC Bridges, Infrastructure, Networks.

Abstract. *The most recent developments in earthquake engineering design are based on the concept of prescribed performance levels, rather than the traditional prescriptive approaches. A relatively well-known software tool for conducting PBEE studies for bridge-ground systems (BridgePBEE), which is based on three-dimensional (3D) finite element model analyses carried out in OpenSees, considering a simplified soil-structure interaction macro element, has been recently applied to a limited number of individual bridge structures. This study aims to apply a Performance-Based Earthquake Engineering (PBEE) methodology to the seismic assessment of an existing road network, in which the nodes correspond to bridges, rather than to a single structural configuration, using BridgePBEE to study the response of different real bridges. Such versatile Finite-Element (FE) environment provided the possibility of processing detailed information on effective damage, safety and economic assessment. In addition, by definition of PBEE, informed decisions of the evolution of seismic losses in terms of cost and time repair quantities were derived at different Peak Ground Acceleration (PGA) levels. Nonlinear dynamic analyses were then run by using properly selected seismic records as input, by means of the conditional spectrum method applied to the seismic hazard outputs for the considered target site, using peak ground acceleration as reference intensity measure. The conclusions provide performance considerations of the investigated population of bridges in terms of repair costs and time, deriving relevant considerations for design procedures.*

1 INTRODUCTION

Performance-Based Earthquake Engineering (PBEE) is a recent methodology developed by the Pacific Earthquake Engineering Research (PEER) Center based on the concept of design for prescribed performance as opposed to more conventional prescriptive approaches. The contributions of PEER to theoretical development, applications in academia and industry and the inclusion of precepts into building/design codes have also been applied to bridges: Mackie et al. [1] used the numerical PEER platform OpenSees [2] with the inclusion of ground-foundation interaction and the definition of a list of performance groups (PGs), on the basis of the commonly used repair methods and aggregation of decision data of typical prestressed, single-column bent, multi-span, box girder bridges in California. In addition, damage to these PGs was tied to explicit repair procedures and repair quantities allowing the cost estimation of repair efforts necessary to bring back the bridge to the pre- seismic event functionality level. Simultaneously, Elgamal et al. [3] developed a graphical user interface for three-dimensional (3D) ground-foundation systems, OpenSeesPL [4], which uses OpenSees as the finite element (FE) analysis engine, adding pre- and post-processing capabilities, which include material properties, boundary conditions and mesh characterization together with the possibility of performing pushover or seismic response analyses of single piles or group of piles.

The aforementioned tools for the application of PBEE methodologies to bridge structures on one side and for the evaluation of the soil-structure interaction effects on the other, resulted in the development of bridge PBEE user interface [1, 3] to evaluate effective damage, safety and economic assessment either when nonlinear static analysis or nonlinear dynamic response of bridge structures are considered, following the trend of many other studies focusing on the nonlinear behaviour of bridge and building structures [5, 6, 7 ,8, 9].

Past studies were carried out in order to scrutinize such framework [10, 11] however, in all of them, the pre-defined seismic input and individual bridge structures with certain properties were considered. On the other hand, the present paper considers the employment of such analysis framework to classes of existing bridges of a road network located in Italy, made up of structures with homogeneous and similar elements.

Nonlinear dynamic analyses were run using properly selected seismic records [12] as input, compatible with the seismic hazard of the target site for 2% probability of exceedance in 50 years. The response of the bridge class was then assessed in terms of repair cost, time quantities such as Crew Working Days (CWD), and the ratio between the cost of repair and the cost of the new construction as function of PGA. The selection of PGA as reference intensity measure was motived by the findings of recent studies [4, 5], which place PGA among the optimal intensity measures to predict reliable seismic response of populations of bridges.

2 PBEE METHODOLOGY

The PBEE methodology aims to assess structural performance in terms of the probability of exceeding threshold values of socio-economic decision variables (DVs) in a seismic hazard environment. The PEER PBEE framework is essentially based on the application of the total probability theorem to disaggregate the problem into several intermediate probabilistic models that involve intermediate variables, such as repair items or quantities (Qs), damage measures (DMs), engineering demand parameters (EDPs), and seismic hazard intensity measures (IMs) [1, 3].

The definition of Performance Groups (PGs) representing a collection of structural and non-structural components, used as indicator of global structural performance for decision-making strategies, in which the decision variables in terms of repair cost ratio (RCR) and repair time (RT) are based on the most common repair methods, is a crucial phase of the PBEE methodology. The notion of a PG allows grouping several bridge components according to the associated repair work and, as such, PGs are not necessarily the same as the individual load-resisting structural components.

The interface is built with 11 PGs [3] and 29 Quantities (Qs), representative of typical Californian bridge schemes, which in the present study are applied to a class of Italian bridges. Event though the US-based PGs were not updated to the Italian context, due to lack of information for the present case study, such limitation is expected to be overcome in future work with additional investigations. The total repair cost was generated through a unit cost function based on the considered Quantities and for each Quantity an estimate of the repair effort was obtained through a production rate. More information regarding the derivation of the default DSs, Qs, unit costs, and production rates and numerical implementations inside the interface can be found in previous studies [1, 3].

3 CASE STUDY

The PBEE methodology is herein applied to the seismic assessment of a group of bridges of similar characteristics, which are part of an existing road network and can be represented by a single structural configuration. More specifically, the present study focuses on a set of 13 bridges (Figure 1) located in one of the major north-south connections in Italy, the A14 Italian freeway (16.9 km total length), which connects Bologna to Taranto. General information for each bridge in terms of geographical position and connection functionality is provided in Table 1.

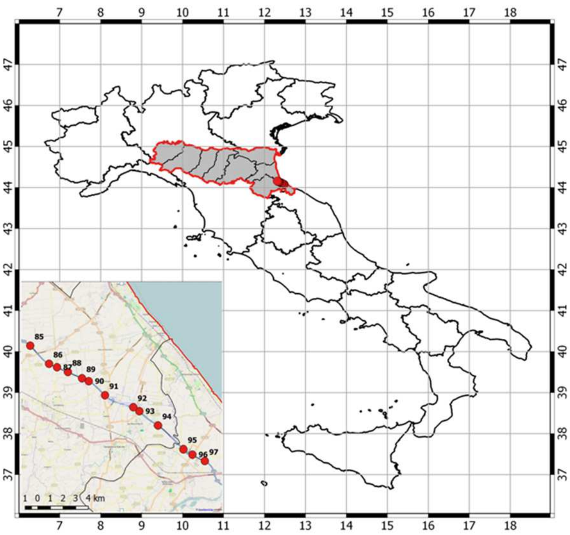

Figure 1: Selected bridges in a road network.

A functionality level (FL) has been defined considering a scale of values, in which “3” means that the connected network is a primary way of connection, such as the entrance to the cities nearby; “1” means a very small way such as connections to the countryside; and “2” is an average connection between the previous two (Table 1). Such piece of information might be particularly useful when priorities need to be established for the allocation of funds for retrofitting operations in in the immediate aftermath of an earthquake.

In order to apply the PBEE methodology to a set of bridges, due to some practical limitations of such software package, either in terms of bridge geometry or computational onus, some simplified assumptions were made. BridgePBEE is capable of performing different types of analyses in bridges with two spans only and solid reinforced concrete pier sections hence the selection of typical Italian overpass bridges was carried out to match such features. Consequently, the set of selected bridges can thus constitute a bridge class and be represented by a single configuration.

Bridge ID	Location		Latitude	Longitude	FL
97	via Variano	Rimini	44° 4'42.63"N	12°29'0.06"E	1
96	via Tolemaide	Rimini	44° 4'58.52"N	12°28'15.83"E	2
95	via orsoleto	S.Vito	44° 5'11.59"N	12°27'43.92"E	3
94	sp13bis	S.M. Pascoli	44° 6'10.59"N	12°26'13.38"E	3
93	sp10	S. M. Pascoli	44° 6'46.34"N	12°25'6.87"E	3
92	via del lavoro	S.M. Pascoli	44° 6'56.31"N	12°24'45.19"E	2
91	Sp33	Gatteo	44° 7'26.32"N	12°23'4.50"E	3
90	sp97	Cesenatico	44° 8'1.38"N	12°22'6.65"E	3
89	via vicinale Sala	Bulgarnò	44° 8'8.73"N	12°21'42.78"E	1
88	via Pavirana	Pavirana	44° 8'24.48"N	12°20'52.30"E	1
87	via Capannaguzzo	Bulgarnò	44° 8'36.32"N	12°20'13.68"E	1
86	SP123	Cesena	44° 8'45.00"N	12°19'45.30"E	3
85	via Sant'Agà	Cesena	44° 9'30.22"N	12°18'38.30"E	1

Table 1: Bridge database

The considered bridges have similar geometry, characterized by 38m long, 2-span deck (Figure 2), supported on rectangular columns (8m x 1m) 6m tall (Figure 3). The steel-concrete deck is 12m wide and 1.80m deep and exhibits a self-weight of 140 kN/m. The connections to the abutments consist of elastomeric bearings (represented in Figure 4) and each abutment is 10m long with 10000 kN as total weight.

Figure 2: Typical characteristics of the considered bridge class

4 FINITE ELEMENT MODEL

The seismic response of the representative bridge was assessed through the nonlinear dynamic analyses of the numerical model obtained with the open-source computational interface BridgePBEE, implemented within the FE code OpenSees [13]. A 3D soil model soil is represented by 208 nodes, effective stress fully coupled (solid-fluid) brick elements built up with 20 nodes in order to investigate soil structure interaction behaviour (Figure 3).

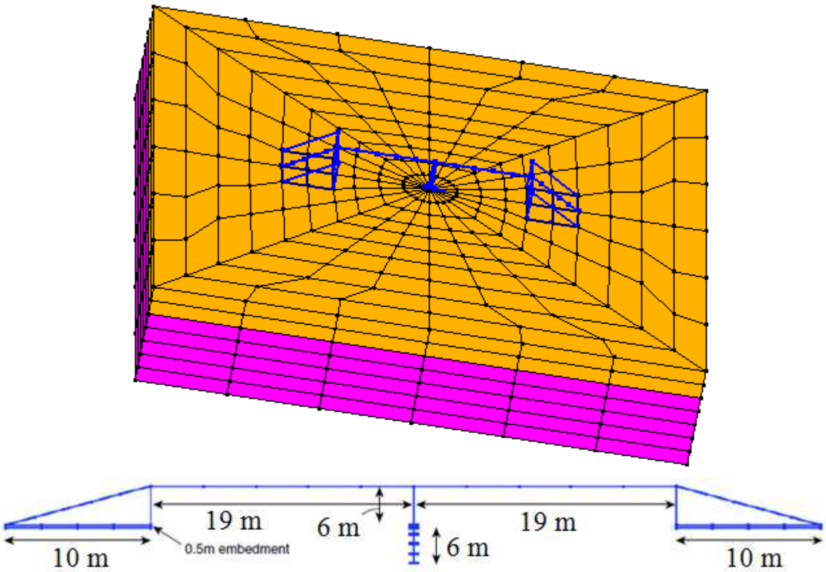

Figure 3: Benchmark bridge model.

In a preliminary stage of the study, a very stiff soil was considered. The ability of Opensees to simulate the real wave propagation adopting realistic boundaries is of particular importance in order to realistically reproduce the above seismic input scenarios. Assuming that at any special location, symmetry conditions can be adopted, periodic boundaries [14] have been considered therefore displacement degrees of freedom of the left and right boundary nodes were tied together both longitudinally and vertically using the penalty method. To this extent, base and lateral boundaries were modelled to be impervious, as to represent a small section of a presumably infinite (or at least very large) soil domain by allowing the seismic energy to be removed from the site itself (more details can be found in [15]).

The reinforced concrete column of the adopted bridge model adopted (Figure 3) is modelled with nonlinear beam-column elements and fibre cross section, whilst the deck is modelled with five separate elastic beam-column (BC) elements considering the equivalent characteristic of a homogenous material (Table 2). As far as abutments properties are concerned, no additional resistance of the soil (at large relative bridge-abutment displacement) is considered and simple roller link connections between the deck and the abutments have been used. The approach ramps make the connection with the longitudinal boundaries. Each abutment has been embedded 0.5m, 10m long and with 10000 kN as total weight.

Deck property	Crossing
Young modulus [kPa]	$1.03 \cdot 10^7$
Shear modulus [kPa]	$6.22 \cdot 10^6$
Cross Section Area [m^2]	5.40
Moment of inertia (trasverse) [m^4]	0.81
Weight per unit length [kN/m]	140

Table 2: Deck properties.

The dynamic characteristics of the bridge have been assessed by comparison with a 3D model performed in SAP2000 (Figure 4) and simple hand calculation considerations. The comparison between the calculated longitudinal period of vibration, the SAP 2000 output

(Figure 5) and the BridgePBEE result (Table 3) denotes a fair agreement, confirming BridgePBEE potentialities and the assumptions done in performing 3D model.

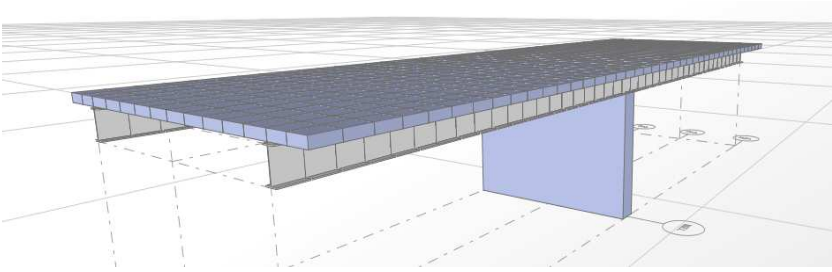

Figure 4: 3D Model (SAP 2000).

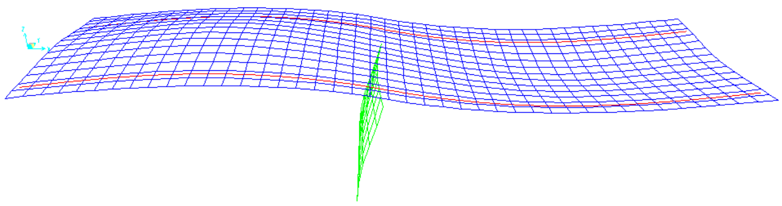

Figure 5: Fundamental shape mode, scale 40:1(T_1= 0.459 s, SAP 2000).

Model	T [s]
Manual	0.473
SAP2000 (3D)	0.459
PBEE (3D)	0.460

Table 3: Fundamental period comparison

5 SEISMIC HAZARD

The considered set of bridges are located in a central region of Italy, close to Rimini, and a full hazard characterization was carried out in order to properly select a suite of records as input for the nonlinear dynamic analyses performed through BridgePBEE. For such purpose, Openquake [16] was used for seismic hazard and disaggregation computations in order to coherently establish the connection between seismic hazard and structural assumptions. The stochastic seismic hazard analysis was undertaken using the seismic hazard harmonization in Europe (SHARE) model for Italy [17], available in Openquake, which.was used to evaluate the hazard corresponding to 2% probability of exceedance in 50 years, considering a circular area of 250 km around the target location (Longitude: 12.483, Latitude: 44.079). “Very stiff soil” site condition (class B) was assumed, according to Eurocode 8 ground type classification, assuming 600 m/s as vs_{30} value. Such simplified assumption, particularly when dealing with road networks that can refer to broad areas, can be nonetheless seen as reasonable considering the soil modelling assumptions described in Section 4. The obtained hazard curve for the site is illustrated in Figure 6.

Future developments can consider the effect of the variation in the soil type when different nodes of the network are considered.

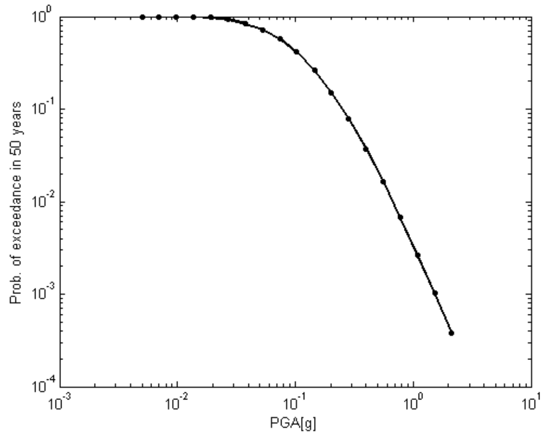

Figure 6: Hazard curve for the target site.

6 RECORD SELECTION AND SEISMIC ANALYSES

The main source of uncertainty associated with seismic excitation is record-to-record variability [18], which has led researchers to investigate innovative record selection and scaling strategies for the reduction of such variability. In the present work the conditional spectrum method [12], a recently proposed selection and scaling procedure, recently employed with success in bridge assessment studies [19], has been employed for the selection of nine records, considered with the two horizontal and vertical components, as seismic input. To accomplish so, disaggregation results conditioned at the spectral acceleration of the target site at the fundamental period of 0.40s was considered in terms of mean values of magnitude, distance and epsilon, which for the target site were evaluated as 6, 14.5 and 1.5 respectively. The main characteristics of the selected records are summarized in Table 4 whereas the corresponding scaled response spectra are shown in Figure 7.

Record ID	Scaling factor	Horizontal component (longitudinal) PGA [g]	Horizontal component (transversal) PGA [g]	Vertical component PGA [g]
CHICHI/NST	2.42	0.748	0.939	0.2625
DUZCE/MDR	2.99	0.360	0.169	0.178
CHICHI04/CHY035	2.22	0.262	0.298	0.104
LOMAP/G	0.67	0.275	0.3170	0.140
CHICHI05/HWA2	2.52	0.253	0.322	0.062
LOMAP/SFO	1.29	0.305	0.425	0.084
WHITTIER/A-EJS	1.16	0.494	0.514	0.239
LOMAP/GMR	1.50	0.338	0.484	0.173
NORTH392/E-RRS	0.82	0.534	0.353	0.491

Table 4: Ground motion records.

In the present study, only PGA was taken as intensity measure with which draw the functions of the main findings whereas the consideration of alternative intensity measures is envisaged in future developments.

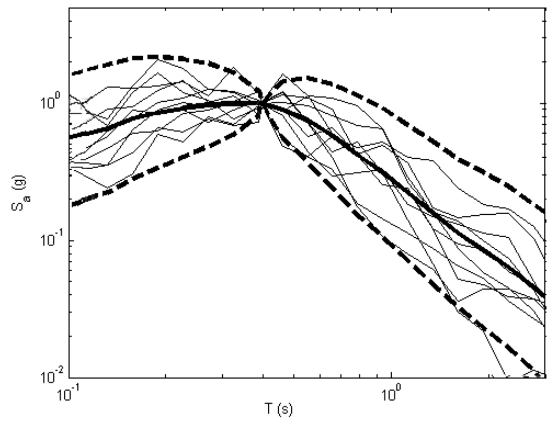

Figure 7: Response spectra of the ten selected ground motion records for Sa(T1=0.40s).

7 RESULTS

Due to the assembly-based (vector) nature of the applied method and according to the total probability theorem, it is possible to disaggregate the results (repair costs and time) into individual contributions. Nine performance groups have been taken into consideration (Table 5).

Performance group (PG) #	Performance group names
1	Max column drift ratio
2	Residual column drift ratio
3	Max relative deck-end/abutment displacement (left)
4	Max relative deck-end/abutment displacement (right)
5	Max bridge-abutment bearing displacement (left)
6	Max bridge-abutment bearing displacement (right)
7	Approach residual vertical displacement (left)
8	Approach residual vertical displacement (right)
9	Abutment residual pile cap displacement (left)
10	Abutment residual pile cap displacement (right)
11	Column residual pile displacement at ground surface

Table 5: Performance Groups.

The results in terms of costs for each performance group, total repair cost ratio (%) and total repair time in crew working days (CWD) are illustrated in Figures 8 to 11. In particular, the main contribution to the Total Repair Cost and Time is shown to be the longitudinal residual drift ratio (PG2), representing column damage (Figure 9). The abutments seem to provide longitudinal and transverse resistance to the bridge deck displacement, not affecting the seismic demand on the bridge. Figures 10 and 11 show that damage (and thus costs and time) is activated only at high PGA levels (approximately 0.95g), for which the bridge will be under severe conditions hence in need for re-design or retrofitting to avoid collapse.

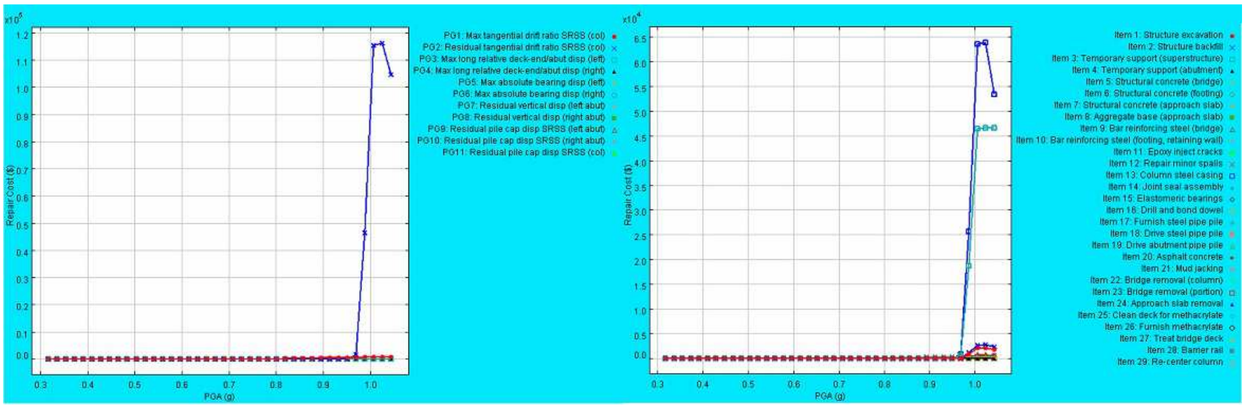

Figure 8. Contribution to expected repair cost ($) from each performance group (left) and from each repair quantity (right)

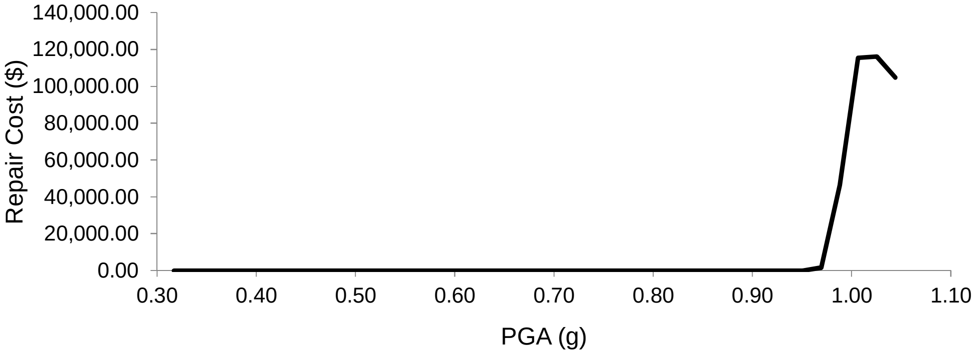

Figure 9. Costs ($) for column damage (PG2).

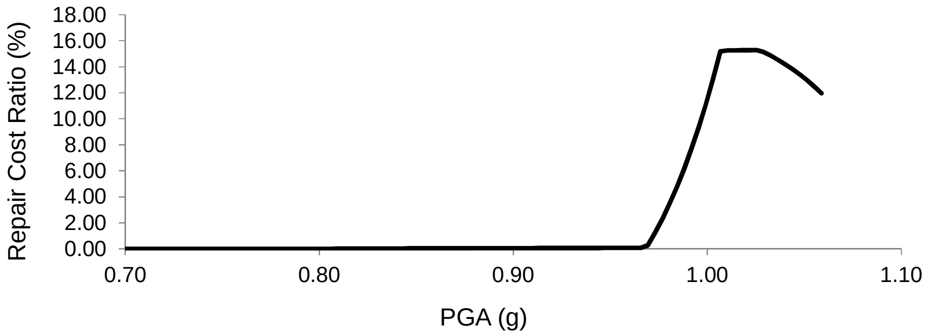

Figure 10. Total Repair cost ratio (%).

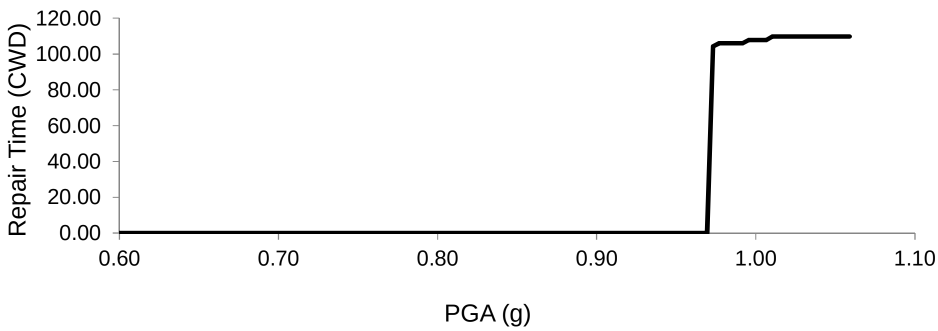

Figure 11. Total Repair Time (CWD).

8 CONCLUSIONS

This study presents a probabilistic based earthquake engineering based tool that enables a comprehensive analysis of single-column bridge-abutment models, applying the overall PBEE analysis framework. The utilization of such software tool allows for the expedite assessment of individual bridges that can represent a wider class of bridges with similar characteristics. The outputs in terms of expected damage ratios, losses and vulnerable elements represent an important information asset for different stakeholders involved with seismic risk analysis. In particular, a single benchmark bridge, representative of those overpassing one of the Italian freeway named A14, connecting Bologna to Taranto, was herein studied as the first step towards a full seismic assessment of the involving network. The results have shown that the damage is essentially concentrated on the pier but only for relatively high values of PGA. The abutments seem to provide longitudinal and transverse resistance to the bridge deck displacement, improving the seismic demand on the bridge. Further analysis will aim to consider more realistic configurations, especially for what concerns bridge connections, in order to properly assess the structural performance. The model for soil structure interaction also requires improvement in order to reproduce more realistically the abutments influence on the entire bridge-soil system.

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