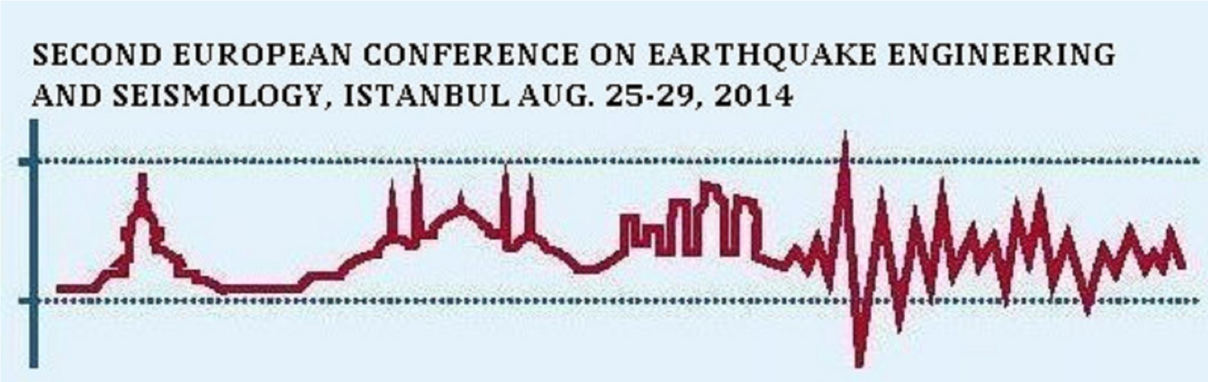

PERFORMANCE-BASED SEISMIC ASSESSMENT OF A CONTINUOUS CONCRETE BRIDGE WITH THE LACK OF CONFINING REINFORCEMENT

Nina SERDAR[1] and Srđan JANKOVIĆ[2]

ABSTRACT

This paper presents seismic risk assessment of bridge structure, based on the probabilistic performance-based seismic assessment methodology, carried out on the continuous eleven span concrete bridge with prestressed slab deck. Minimum amount of transverse reinforcement in bridge piers existed, so total transverse reinforcement ratio in piers was approximately 20% lower than required confining reinforcement in EN 1998-2. Aim of the performed analysis is to show behavior of the bridge, with lack of confining reinforcement in piers, under seismic load. This is common problem for existing bridges designed according to old regulations. Seismic risk assessment is carried out by obtaining fragility curves and by calculating final probability curve that relates probability of damage to seismic intensity. Non-linear model was constructed and time-history analysis was performed using ten pairs of horizontal ground motions selected according to EN 1998-2 to match site specific hazard. In performance based analysis, peak drift ratio was selected as engineering demand parameter (EDP), as well as concrete cover spalling and longitudinal bar buckling as damage measures (DMs). For seismic intensity measure (IM) spectral acceleration was selected.

INTRODUCTION

In past few decades engineering community and researches were focused on improving regulations related to designing of structures in seismic active zones. Knowledge gained through studies of consequences of past earthquakes, most of the times led to stricter requirements that structures need to fulfil. In such way, many existing structures are not designed to meet expectations embedded in current regulations. This does not necessarily mean that these structures are not suitable for use. If considered structure is a bridge than, in most cases it can not be easily put out of service, because of it's position or importance in transportation network, without prior investigation and evidences of its unsuitability. If the goal of investigation is to relate bridge behaviour under seismic load to damages and losses, than performance-based design (PBD) is a concept that should be adopted.

Existing methods in seismic assessment of bridge structures, used in common engineering practise, have many disadvantages regard to answering questions about consequences of representative earthquake event in terms of monetary losses, downtime, casualties etc. Alternative approach is to use performance-based earthquake engineering (PBEE) methodology developed by Pacific Earthquake Engineering Research Centre (PEER). This method was developed in order to overcome disadvantages of conventional design of structure, like force-based approach, and to check if design

[1] Assistant, Faculty of Civil Engineering, Podgorica,ninas@ac.me
[2] Professor, Faculty of Civil Engineering, Podgorica, srdjanj@t-com.me

procedure results in desirable structure response. These procedures can also be used in evaluating performances of existing structures under earthquake, especially if we know that result of PBEE analysis is probability of reaching predefined damage state (any failure mechanism that is relevant for local or global response of bridge). Based on the results of PBEE analysis responsible decision can be made whether the bridge structure is suitable for use or repair, or whether the losses are acceptable or not.

PBEE methodology has four main steps: hazard analysis, demand analysis, damage analysis and loss analysis. The results of these four steps are formulated in probabilistic manner and they are combined to achieve global performance of system.

In first step of analysis, probability of occurrence of different earthquake events, represented with proper intensity measure (IM), is identified. Choice of the representative IM for structure specific site is very important for the analysis, because it can significantly affect probabilistic model by introducing greater uncertainty. Many paper and authors have dealt to this issue, and IM like spectral acceleration, peak ground acceleration or peak ground velocities are in common use for this type of analysis.

Second step-demand analysis relates specific IM to particular response parameter of structure called engineering demand parameter (EDP). For EDP peak drift ratio, residual drift ratio, plastic hinge rotation etc. can be chosen. In order to carry out demand analysis nonlinear structural model should be built, simulations of earthquake load on model should be done in order to determine EDP.

The goal of the damage analysis is to relate chosen EDP to damage state in probability manner. In other words probability that structure will experience predefined damage state-damage measure (DM) for given EDP should be established. This is often done by creating fragility curves which are results of observed damages on objects which experienced earthquake, experimental results or analytical estimation of damage.

Finally, in last step called loss analysis, probable losses are represented with decision variables (DV). Result of the analysis is probability of occurrence of chosen DV at given level of earthquake intensity. Cost of bridge repair, state of bridge functionality in post earthquake period, structure downtime etc. can be chosen for DV.

In this paper performance of continuous concrete bridge under seismic load will be evaluated using previously described methodology.

DESCRIPTION OF BRIDGE STRUCTURE

In this study seismic risk assessment of a continuous eleven span concrete bridge with prestressed slab deck will be performed. Check of existing design will be carried out on the base of PBEE methodology using first tree steps in analysis mentioned earlier. Result of this study will be probability of occurrence of certain damage state. Selected bridge can be named as representative in terms of structural design for certain period in Montenegro, before the introduction of European standards.

The bridge crosses a dry barrier and it is partly in horizontal curve of radius R = 742 m. Also bridge disposition is in vertical curve of radius R = 10000 m. The total length of the structure is 296 m. Bridge is eleven span frame (22 +28x9+22=296m), and has a deck that is pre-stressed slab 12.72m wide and 1m high. The cross section of the bridge is shown in Fig. 1. At each bent there are two columns joined with bent cap. Height of piers decreases toward the abutments from 18m to 12m. Piers cross section is also shown in Fig. 1. Bridge deck is supported on expansion bearings at the abutments and over first and last column. Over interior columns bridge deck is rigidly (monolithically) connected to deck and bent cap. During the actual design it was assumed that bridge is founded on stiff rock (soli class A according EN1998-1). All the mentioned characteristics in bridge alignment, except vertical skew, were taken into account in the analysis.

The column concrete was assumed to have unconfined compressive strength of 38 MPa. Total longitudinal reinforcement is 64ϕ25 bars, which corresponds to reinforcement ratio of 2%. Transverse reinforcement are closed hoops on longitudinal distance of 12.5cm and transverse distance in cross section of 20cm. This corresponds to transverse reinforcement ratio of 0.0032.

Here to mention that in original design seismic forces were reduced introducing behavior factor q=3 in elastic response spectrum. According to EN 1998-2 for this bridge ductile capacity should be provided, in order to compensate strength reduction, by series of rules established for so called bridges with ductile behavior. No such specific rules were applied in design of structure, which lead to 20% less confining reinforcement ratio than required confining reinforcement when EN 1998-2 is applied.

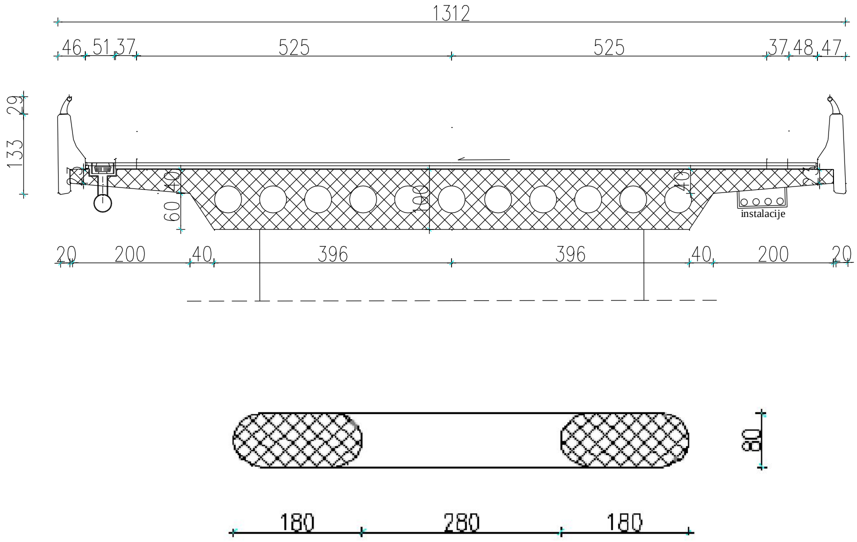

Figure 1. Bridge deck and column cross section

HAZARD ANALYSIS

The performance of described bridge is evaluated for earthquake events with moderate probability of occurrence, which corresponds to 10% probability of being exceeded in 50 years. The mean return period of this hazard level is 475 years.

For specific site condition there were no enough ground motion data registered in Montenegro and region so selection of ground motion was conducted using European and PEER strong motion data base. Design ground acceleration was 0.28g.

The ground motion records used in dynamic analysis are shown in Table 1. Set of motions consisted of ten pairs of horizontal ground motions selected to meet the following conditions: magnitude is equal or larger than 6.5, station's distance from the fault is greater than 10km and consistency to relevant damped elastic response spectrum of designed seismic action was established by scaling the amplitude of motions as it is specified in EN 1998-2.

Table 1. Ground motion set for analysis

No	Earthquake	Magnitude	Station
1	Kozani, Greece	6.5	Kastoria
2	Kozani, Greece	6.5	Veria
3	Superstition Hills, California	6.54	El Centro Imp
4	Imperial Valley, California	6.53	El Centro, Array #10
5	Loma Prieta, California	6.93	Sunnyvale Colton Ave
6	Avej, Iran	6.5	Gilvan (Dehari)
7	Kozani, Greece	6.5	Florina
8	Loma Prieta, California	6.5	Saratoga Aloha Ave
9	Duzce, Turkey	7.2	Izmit
10	South Iceland	6.5	Hrauneyjafoss

Selected IM in this study was the spectral acceleration at first mode period $S_a(T_1)$. This is common choice for selection of IM in PBEE analysis recommended in literature. SRSS spectra of individual earthquakes (N_1-N_{10}) established by taking square root sum squares of 5% damped spectra of each component of ground motions, average spectra of the individual earthquakes (S_{avg}) and elastic response spectra of design seismic action (S_{dEC8}) are shown in Fig. 2.

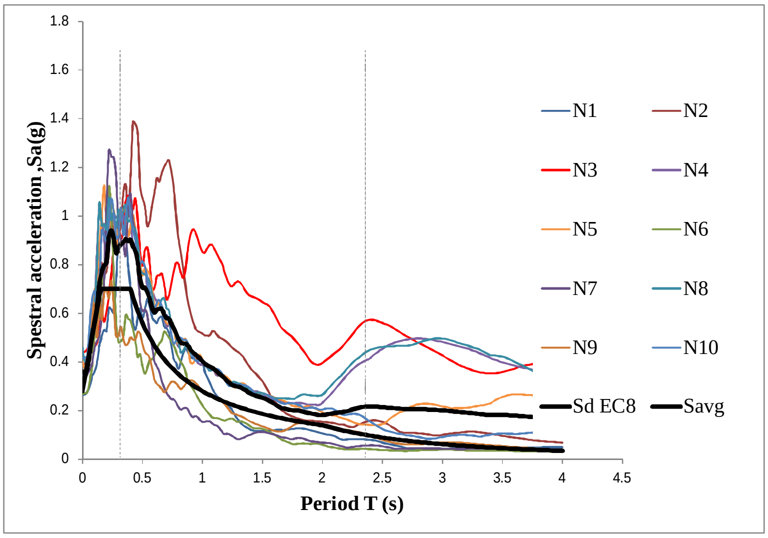

Figure 2. Individual and Average SRSS spectra of selected ground motions

DEMAND ANALYSIS

In order to relate ground motions to structure specific response (represented with EDP) structural analysis was performed. The bridge was modelled using software package SeismoStruct v.6.(see Fig.2.). To develop simulation model following elements were used: force based inelastic frame elements, elastic beam elements and zero-length spring elements.

Force based inelastic elements that consider spread of plasticity along the element were used to model the columns. Column cross section was divided in fibre section, with core and cover fibres. Peak stress and strain for concrete core were increased due to confinement effect based on Mandel et al.. Integration along the element was done in five integration points. Sectional discretization is shown in Fig 3. Soil-structure interaction was not considered and columns were assumed to be fixed at the base. No significant yielding expected to occur in bridge deck so elastic frame elements were used to model bridge deck and bent caps. Each span was discretise with four elements. Based on design drawings deck section properties were calculated and assigned to elastic frame elements placed at the deck gravity centre. Deck centre of gravity and top of the pier were connected with rigid arm, also elastic frame element. Longitudinally movable bearings at the abutments and first and last pier were modelled as link elements. Link element is a zero-length spring element with infinitive stiffness at vertical and transverse direction and fully flexible in other directions. Bridge deck and column masses were considered as distributed along elements. Masses of the bent caps were taken as lumped at the top of the each pier.

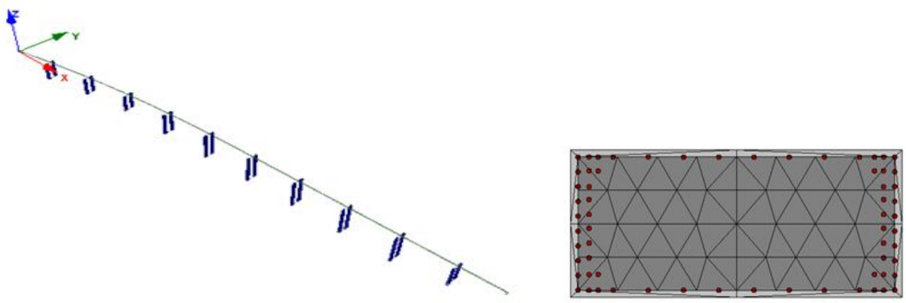

Figure 3. Bridge model developed in SeismoStruct v.6. and column cross section discretisation

Time history analysis was performed by applying fault normal and fault parallel ground motion components simultaneously. Both ground motions components were previously scaled, as explained earlier. More severe component was always applied in transverse direction. Geometric nonlinearity was included in analysis.

In this phase of the analysis peak drift ratio at the top of the pier was chosen for EDP. Drift values were calculated as square root sum square of longitudinal and transversal motion. Maximum value for ten piers was selected. Maximum value of drift ratio for all applied ground motions sets was always at the top of the pier at witch bridge crosses from straight alignment to curve.

Using calculated drift ratios next step in analysis was to derive relation between selected EDP and IM. Distribution of EDP conditioned on IM is assumed to have log-normal distribution shown in Eq.(1). When this relation is obtained probability of exceeding of certain EDP limit can be evaluated as shown in Eq. (2). Best fit line relating peak drift ratio and spectral acceleration Sa(T1) was derived applying log-normal distribution and last square fit to the results. From regression analysis coefficients A and B were determined. These coefficients are used for establishing demand model given by Eq. (2).

$$\ln\left(\hat{EDP}\right) = A + B\ln\left(IM\right) \qquad (1)$$

$$\mathrm{P[EDP > edp^{LS}} \;|\mathrm{IM}\;] = 1 - \Phi\left(\frac{\ln\left(EDP\right) - A - B\ln\left(IM\right)}{\sigma_{\ln(\mathrm{EDP|IM})}}\right) \qquad (2)$$

where Φ is standard normal distribution function and $\sigma_{\ln(EDP|IM)}$ is standard deviation of the natural log of the EDP. Best fit line relating EDP to IM, and probability of exceeding demand for considered hazard level are shown in Fig 3a. and 3b.

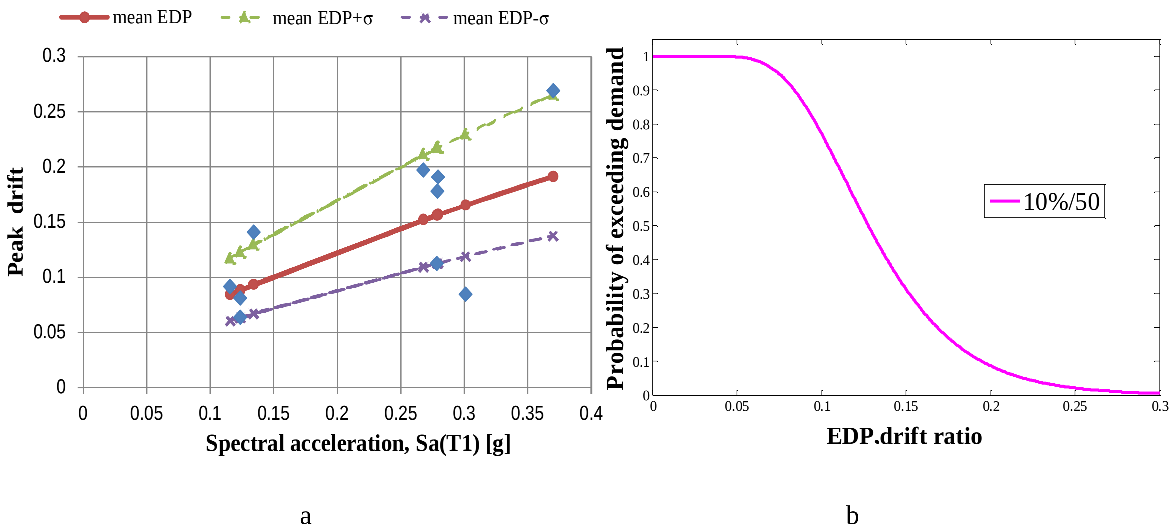

Figure 3. (a) Derived relation between EDP and IM (b) Probability of exceeding demand for considered hazard level

From the results presented, for considered hazard level, probability of exceeding drift ratio of 0.15 is fairly large: 31%. As analysis showed, for almost all values of drift greater than 0.15 significant yielding of column cross section occurred and limit values of capacity of rotation of column cross section were exceeded. Also results showed that no plastic behaviour occurred in bridge deck, what is recognised as desirable behaviour for bridge structure. At this point of analysis it was concluded that unsatisfactory behaviour of bridge, even under moderate seismic hazard level, can be result of insufficient rotation capacity of critical column cross section. Further analysis was carried out with the aim of the qualitative and quantitative description of the effects of this unsatisfactory behaviour. This was done in damage analysis which can be easily understood and interpreted even by non-engineers.

GLOBAL RESPONSE OF STRUCTURE

In order to determine global response of the structure, capacity of plastic curvatures were calculated for column cross section. These values were compared to maximum values of curvatures under seismic load. Also maximum value of bending moment in bridge deck during earthquake was compared to deck flexural resistance. Following conclusions were made: bridge deck had essentially elastic behaviour and no plastic deformations had developed during earthquake events; significant yielding occurred in bridge columns. For several earthquake events demand/capacity ratios were larger than 1.

Table 2. Demand capacity ratios for selected earthquake events

Earthquake No	$Sa(T_1)$	Max curvature	demand/capacity
1	0.1237g	0.00510	0.489
2	0.6705g	0.008561	0.598
3	0.2679g	0.03913	3.751
4	0.2791g	0.02823	2.707
5	0.301g	0.00559	0.536
6	0.1236g	0.004553	0.437
7	0.1157g	0.00845	0.810
8	0.2793g	0.05903	4.128
9	0.1341g	0.007926	0.760
10	0.2785g	0.006312	0.6052

At this point of the analysis it was concluded that bridge deck had satisfactory behaviour but bridge columns could expect yielding which exceeds their ductile capacity. In order to fully describe observed disadvantages of bridge design, damage analysis was performed.

DAMAGE ANALYSIS

To perform PBEE methodology it is necessary to relate EDP to certain damage measures (DMs). DMs considered in this study are onset of spalling of concrete cover and onset of buckling of longitudinal bars. First DM represents minor damages in bridge columns but still important because it means that first flexural damage in columns during earthquake happened. In this damage state functionality of bridge can be reduced and repairing cost are not negligible. Bar bucking represent major damages, because under this damage state duration and costs of repairs can be significant and loss of bridge functionality can happened.

Progression of damage in concrete column is complex. To develop practical model to be used in earthquake engineering, numerous experimental data should be collected relating cover spalling and bar buckling. For the purpose of this study, Barry and Eberhard equations are used to determine drift ratios on onset of cover spalling and bar buckling (Eq.(3) and Eq. (4) see references).

$$\frac{\Delta_{spall\_cal}}{L}\% = 1.6\left(1-\frac{P}{A_g f_c'}\right)\left(1+\frac{L}{10D}\right)$$

(3)

$$\frac{\Delta_{bb\_cal}}{L}\% = 3.25\left(1+k_e\rho_{eff}\frac{d_b}{D}\right)\left(1-\frac{P}{A_g f_c'}\right)\left(1+\frac{L}{10D}\right)$$

(4)

where P is the axial load, A_g is the gross section area, f_c' is the concrete compressive strength, L is the distance from point of fixity to point of inflection, D is the column diameter, k_e is taken as a constant value 50 for rectangular cross section, ρ_{eff} is the volumetric transverse reinforcement ratio and d_b is the longitudinal bar diameter. Statistic of accuracy of EDP/DM equations (μ, COV, σ), taken from literature, and the resulting mean drift ratios (Δ_{spall} / L and Δ_{bb} /L) for bridge columns are given in Table 3.

Table 3. Statistic of accuracy of EDP/DM and calculated mean drift ratios

Damage state	$\mu\,(\frac{\Delta}{\Delta_{cal}})$ mean	COV=σ/μ $(\frac{\Delta}{\Delta_{cal}})$ Coefficient of Variance	$\sigma\,(\frac{\Delta}{\Delta_{cal}})$ standard deviation	mean Δ/L
Cover Spalling	0.97	43.3%	0.42	0.0155
Bar Buckling	1	26.3%	0.263	0.063

Using the standard deviation and the mean drift, given in Table 3., and assuming log-normal distribution, fragility curves for these damage states were obtained and shown in Fig.4.

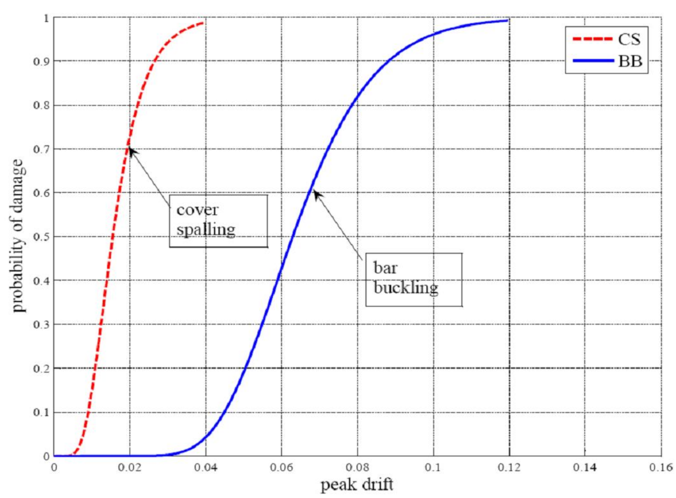

Figure 4. Fragility curves for bar buckling and cover spalling

Probability of column damage can be read from fragility curve. When demand peak drift is Δ/L=6% probability of damage of concrete cover is 100% and probability that longitudinal reinforcement will begin to buckle, at demanded level of drift, is 40%.

By solving integral in Eq. (5), final probability curve which relates probability of damage to IM was derived. First probability in expression under integration in Eq. (5) is above presented fragility curve, and second is probability density function, assuming that EDP is log-normally distributed. Probability density function of log-normal distribution of EDP conditioned by IM, can be easily constructed knowing standard deviation and mean value. Graphical representation of numerical solution of integral in Eq. (5), for this case study, is shown in Fig.5.

$$P[DM< dm^{LS} \,|IM = im] = \int_{edp} P[DM < dm^{LS} \,|EDP = edp]\,dP[EDP < edp|IM = im\,]\,d\,edp \quad (5)$$

If obtained probability shown in Fig 5., are analysed it can be noticed that, for S_a=0.181g, which corresponds to spectral acceleration for fundamental period of structure on elastic response spectrum, probability of bar buckling is 75% and probability of damage of concrete cover is 86%. This means that likelihood of major damages for considered moderate hazard level is fairly high.

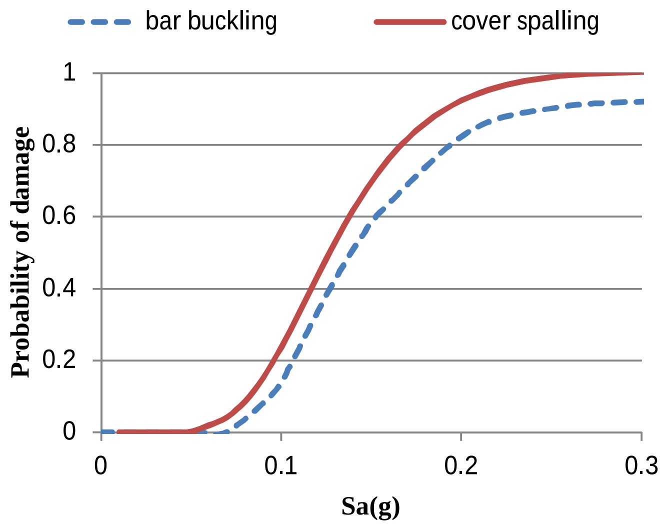

Figure 5. Probability of damage for specific seismic level

CONCLUSIONS

This paper presented PBEE methodology used to determine behaviour, in terms of damage, for bridge structure with lack of confinement reinforcement in columns. This is a common case for existing bridges, designed according to regulations that do not prescribe detailing requirements for providing ductility of compression concrete zones in bridge columns.

This bridge is designed to have ductile behaviour, but the amount of provided confinement reinforcement was significantly lower that required. Column cross section experienced plastic deformations and bridge deck had essentially elastic behaviour under seismic load. For some earthquake events demand plastic rotations exceeded column cross section capacity. In order to investigate if flaws in bridge design would lead to serious damages, probabilistic approach was included. Probability of major damages, like bar bucking, are fairly high for moderate seismic hazard level. It can be concluded that request for confining reinforcement provided in EN 1998-2 is justified, and it certainly leads to better bridge performance. In this case study, lack of confining reinforcement in bridge columns caused unsatisfying behaviour regard to calculated probability of damage. In case of cover spalling and bar buckling, exposure of reinforcement is inevitable what would lead to bridge closure, or reduced load in post earthquake period. At this state, for this case study, it can be said that collapse prevention design probably failed, which can be checked by estimating residual displacements. This is left for future research of similar bridge structures.

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