Abstract

Horizontally curved concrete girder bridges have complex dynamic characteristics because of their asymmetry and non-uniform mass and stiffness distribution. Their seismic behaviour is considerably affected by various such as structural characteristics, radius of the superstructure curvature and local site conditions. The computational, three-dimensional (3D) bridge models consisting of concrete girders with
concrete deck, single pier columns and caps were created in OpenSees analyzing a representative, three-, four- and five-span continuous curved concrete girder bridges in N. Macedonia. Different seismic hazard levels and soil conditions were chosen in order to perform site-specific hazard calculations to investigate the effect of critical curved bridge parameters on the seismic response using a group of representative bridges. Two levels of capacity were considered: damage and collapse limit state (DLS and CLS). The results from the performed extensive nonlinear analyses including uncertainties were used to estimate the influence of the number of spans, deck width, pier height, deck horizontal curvature radius and local site effects on the bridge performance. The influence of soil conditions and superstructure curvature are significant for the seismic vulnerability of girder bridges, especially those with more spans. With the increase of the superstructure curvature radius, the bridges become more vulnerable, particularly if they have more spans and are founded on soil of weaker characteristics. The difference in the probability of damage occurrence to bridges with smaller number of spans, regardless the curvature radius, is small if these are founded on a good base. In the case of CLS, such probability differs extensively, particularly in multi-span bridges founded on weak soil. Regardless the curvature radius and the soil characteristics, the width of the superstructure has a favorable effect upon the seismic response of the selected type of bridges. Bridges with piers of a greater height and greater curvature radius exhibit considerably less favorable seismic behavior than bridges of smaller curvature radius founded on better soil.
INTRODUCTION

Bridge structures are critical elements of transportation systems providing continuity of road networks and should therefore be permanently in good conditions. The growth and dynamic character of traffic loads, the variability of wind, seismic and hydraulic forces, and the natural deterioration of constituent materials of bridges tend to increase their vulnerability. Since their collapse may impose high costs for their users and the local economies, they require proper and timely maintenance. Without adequate maintenance, the risk of collapse and the costs for their repair will be increasingly higher over time, especially at the end of their serviceability period.

Multi-span bridges are strategic elements of modern transportation networks. They provide an exceptional solution to the needs of traffic congestion, but if they are curved, their behaviour becomes quite complex due to their coupled bending and torsion response. Dynamic excitations further complicate their performance, especially if the bridge has a small radius and is founded on soil with weak characteristics.

Different researchers performed various studies. Recently, a simple spatial dynamic model of this type of bridge has been analysed by the Amjadian, M. & Agrawal, A. K. (2017). They have shown that the translational motion of the deck of horizontally curved bridges in the direction that is perpendicular to their axis of symmetry is always coupled with the rotational motion of the deck, regardless the location of the stiffness centre. On the other hand, seismic performance of curved bridge structures is significantly affected by their column height, design era, and horizontal deck curvature. Mangalathu, S. et al., have found out that an increase in the deck horizontal curvature increases the bridge vulnerability irrespective of the design era and the height range. In general, unseating and column drift have been the components that dominate the system fragility of curved bridges. Influence of the horizontal curvature radius and bent skew angle on seismic response of RC bridges has been analysed by Serdar, N et al. (2017). They concluded that the increase of the bent skew angle causes shortening of the longitudinal vibration mode periods and an increase in the transverse vibration mode periods, while the decrease of the bridge deck horizontal curvature radius results in an increase of the seismic vulnerability of the bridge. Regarding the slope, they have proved that the bridge vulnerability increases with a decrease in radius and increase in skewness. For the last decade, a lot of studies and experiments have been done in order to examine the influence of the local site effects and ground motions upon the behaviour of curved bridges. In their study through a shake-table test on a 1/10-scale typical curved bridge, Li, X et al. (2014) have proved that both have significant influence on the seismic responses of these bridges. They have concluded that the curvature radius causes curved bridges to be more sensitive to ground motion spatial variations, especially the local site effect, so that curved bridges may be
damaged more seriously than straight bridges during the same earthquake. Furthermore, they have shown that seismic responses of irregular curved bridges are more sensitive to ground motion multidimensionality than those of straight bridges.

Seismic vulnerability sensitivity studies of bridges with vastly different configurations, confirm that shorter-span bridges with a tighter radius of curvature are less sensitive to seismic excitations than those with straighter decking, indicating that a span length-to-radius of curvature ratio (S/R) of both single and multi-span bridges is statistically determined to be the most influential parameter (Rogers L.P & Seo, J., 2017).

The research of continuous girder bridges with a small radius established that, the bearing capacity of the pier is very important due to the fixed pier of curved box girders. This must be completely considered in the design of similar structures. The results obtained for the stability coefficient with all kinds of overturning axes in multi-span continuous curved box girders show that the anti-overturning performance is very strong because more support means more effective constraints on the structure, especially when the radius is small (Luo, X., 2018).

Minavand, M., & Ghafori-Ashtiany, M. (2019) have modelled the box deck of Sadr Bridge located in Teheran in straight and horizontally curved forms with four central arc angles. They have modelled the bridge in non-isolated and isolated states with a friction pendulum bearing (FPB). In addition, they have selected and scaled near and far-fault ground motions and have performed modal and nonlinear time history analysis. In their study, they have concluded that reducing the arc radius results in a decrease of the shear force and displacement of the pier in the tangential direction and an increase of the shear force and displacement of the pier in the radial direction. The results have shown that near-fault ground motions generate more force and displacement demand in the pier and deck of bridges. The displacement of the deck of the straight and horizontally curved bridge under near-fault ground motions, due to its pulse-like nature, is considerably larger than in the case of far-fault ground motions. Seismic isolation by a friction pendulum bearing plays a major role in reducing the force and displacement demand and improving the seismic performance of horizontally curved bridges.

These studies have indicated that concrete bridges with horizontal curvature are more sensitive to ground motion spatial variations, especially the local site effect and they may be damaged more seriously than straight bridges during the same earthquake. Therefore, a research that includes computational parametric studies that incorporate parametric variations
of the geometric characteristics of structural elements, taking into account different soil conditions and curvature radii, would be beneficial for better understanding the seismic behaviour of horizontally curved bridges.

LOCAL SITE CONDITIONS

Six representative sites of different seismic hazard levels and soil conditions were chosen in order to perform site-specific hazard calculations and disaggregation analyses (D1.2 and D1.3, Infra-NAT 2018).

The adopted seismic hazard model (Milutinovic et al., 2016) takes four GMPEs into account, chosen to be most appropriate for Western Balkan region (Salic et al., 2017). These are: Aetal14 (Akkar et al., 2014), Betal14 (Bindi et al., 2014), BSSA14 (Boore et al., 2014) and CY14 (Chiou and Youngs, 2014). Site amplification in all of the selected GMPEs is based on broad soil categories defined upon the $V_{s30}$ values. Aetal14 assumes inclusion of a non-linear site amplification function that is a function of $V_{s30}$, Betal14 models the site amplification based on EC8 soil classification (four classes from A to D), BSSA14 adopts site classes, which corresponds to NEHRP site categories, and CY14 uses non-linear site response as continuous function of $V_{s30}$. Accounting for the unavailability of subsoil data and keeping in mind that all of the used GMPEs site-specific computation requires only the $V_{s30}$ or the information about soil category at the bridge location, it was decided to use the USGS topographic based $V_{s30}$ data of Wald and Allen (2007).

Three main criteria for the selection of site characteristic points for the analysis were taken into account: bridge locations, different 475 years-return period PGA hazard level and different soil category (according $V_{s30}$ values). The characteristics of the six selected sites are listed in Table 1.

Site-specific calculations were performed at the six locations. In particular, three of them are representative of bridges located in regions characterised by stiff soil conditions, corresponding to EC8 soil class B, while other three are representative of bridges located on soft soil conditions, corresponding to EC8 class C. Representative values of $V_{s30}$ =600 m/s and $V_{s30}$ =300 m/s were assumed for soil category B and C, respectively. Figure 1 shows the Uniform Hazard Spectra (UHS), taking into account proper soil conditions for the six selected sites, for two different return periods.

| Table 1 Chosen locations for site-specific hazard analyses (D1.2, Infra-NAT 2018) |
|-----------------------------------------------|
| **Lon (WGS-84):** | 21.576018 | 22.428045 | 20.952442 | 22.390784 | 21.559054 | 20.581745 |
| **Lat (WGS-84):** | 41.999761 | 41.301515 | 41.519969 | 41.902090 | 41.330642 | 41.534374 |
| **$V_{s30}$:** | 294 | 199 | 472 | 236 | 497 | 441 |
| **EC8 Soil Class:** | C | C | B | C | B | B |
Figure 1 UHS for 224-years (left) and 975-years (right) return periods, for the six selected sites, taking into account proper soil conditions (D1.2, Infra-NAT 2018)

The disaggregation analysis were performed for the six sites shown in Table 2 and 3. Only the most representative branch of the Logic Tree (LT) was adopted, which is associated with the gridded source model and the GMPE by Chiou and Youngs (2014). The disaggregation results of PGA are summarized in Table 2 (for soft soil sites: numbers 1, 2 and 4), and Table 3 (for stiff soils sites: numbers 3, 5 and 6), in terms of mean values of the magnitude-distance scenario.

Table 2 Disaggregation results for PGA at 6 different return periods for sites 1, 2 and 4, located on soft soil conditions (D1.2, Infra-NAT 2018).

| Site 1 | Site 2 | Site 4 |
|-------|-------|-------|
| RP    | Mag   | Dist  | Mag   | Dist  | Mag   | Dist  |
| 98    | 5.3   | 23.9  | 5.5   | 23.2  | 5.6   | 32.5  |
| 224   | 5.4   | 20.5  | 5.7   | 20.6  | 5.7   | 28.4  |
| 475   | 5.5   | 18.6  | 5.9   | 18.7  | 5.9   | 25.2  |
| 975   | 5.5   | 17.4  | 6.0   | 17.2  | 6.0   | 22.6  |
| 2475  | 5.6   | 16.2  | 6.3   | 15.8  | 6.3   | 19.9  |
| 4975  | 5.7   | 15.5  | 6.4   | 15.2  | 6.4   | 18.5  |

Table 3 Disaggregation results for PGA at 6 different return periods for sites 3, 5 and 6, located on stiff soil conditions (D1.2, Infra-NAT 2018).

| Site 3 | Site 5 | Site 6 |
|-------|-------|-------|
| RP    | Mag   | Dist  | Mag   | Dist  | Mag   | Dist  |
| 98    | 5.6   | 20.6  | 5.6   | 39.1  | 5.7   | 18.5  |
| 224   | 5.7   | 18.3  | 5.6   | 32.6  | 5.8   | 16.7  |
| 475   | 5.8   | 16.8  | 5.6   | 27.9  | 6.0   | 15.6  |
| 975   | 5.9   | 5.9   | 5.9   | 5.9   | 5.9   | 5.9   |
| 2475  | 6.1   | 15.1  | 5.6   | 21.3  | 6.2   | 14.3  |
| 4975  | 6.2   | 14.6  | 5.6   | 18.8  | 6.3   | 13.9  |
For these six locations shown in Table 1, was performed record selection used for the nonlinear analyses of the bridges. The conditional spectra were computed using the GMPE by Chiou and Youngs (2014) along with the correlation model by Baker and Jayaram (2008). Unlike Akkar and Bommer (2010), which requires few input parameters (magnitude, Joyner-Boore distance, focal mechanism and $V_{s30}$), the GMPE by Chiou and Youngs (2014) is significantly more complex and requires an accurate description of the fault rupture plane.

Beside the rake, needed to estimate the focal mechanism, it requires the dip of the fault and the depth-to-top of rupture ($Z_{TOR}$). While the rake ($\lambda=-90^\circ$) and the dip ($\delta=45^\circ$) were established from PSHA, $Z_{TOR}$ was computed from geometrical considerations as shown in Eq. 1:

$$Z_{TOR} = \max(Z_{HYP} - 0.5W\sin(\delta), upper_{sd})$$  (1)

where $Z_{HYP}$ is the hypocentral depth (fixed at 15 km from the PSHA), $upper_{sd}$ is the upper seismogenic depth (fixed at 5 km from the PSHA) and $W$ is the down-dip rupture width. $W$ was estimated through the relationship by Wells and Coppersmith (1994) for normal focal mechanisms, considering the mean magnitude from the disaggregation analysis and assuming a rupture aspect ratio equal to 1. This can be certainly true for small earthquakes but not for large ruptures, where the extent of the seismogenic layer limits the vertical propagation of the rupture. Therefore, in the case in which the calculated $W$ exceeds the permitted seismogenic layer, it is adjusted to fit it:

$$W = (lower_{sd} - Z_{TOR})/\sin(\delta)$$  (2)

where $lower_{sd}$ is the lower seismogenic depth (fixed at 25 km from the PSHA). The adopted distance metrics are the Joyner-Boore distance (i.e. the horizontal distance to the surface projection of the rupture, $R_{JB}$), the rupture distance (i.e. the slant distance to the closest point on the rupture plane, $R_{RUP}$) and the site coordinate (i.e. the horizontal distance to the surface projection of the top edge of the rupture measured perpendicular to the fault strike, $R_{X}$), which is used to quantify the hanging-wall effect. Following the equations provided in Kaklamanos et al. (2011), $R_{RUP}$ and $R_{X}$ were computed from the source geometric parameters ($W$, $Z_{TOR}$, and $\delta$), $R_{JB}$, and $\alpha$ (i.e. the source-to-site azimuth). Since $\alpha$ was not known, it was taken equal to 50°, following the recommendation by Kaklamanos et al. (2011).

The maximum scale factors (SF) and the allowed magnitudes and distances which are used for record selection are shown in Table 4. An example of record selection for Site 2 and the return period of 475 years is shown in Figure 2,
while the corresponding metadata parameters are listed in Table 5.

**Table 4** Selection parameters adopted for the selection of suitable ground motions (D1.3, Infra-NAT 2018)

| $T_r$ [years] | Max SF | Allowed Magnitudes | Allowed Distances |
|---------------|--------|--------------------|-------------------|
| 98            | 2.0    | $M_{\text{disag}} \pm 0.25$ | $D_{\text{disag}} \pm 35$ km |
| 224           | 2.5    | $M_{\text{disag}} \pm 0.25$ | $D_{\text{disag}} \pm 35$ km |
| 475           | 3.0    | $M_{\text{disag}} \pm 0.25$ | $D_{\text{disag}} \pm 50$ km |
| 975           | 3.5    | $M_{\text{disag}} \pm 0.25$ | $D_{\text{disag}} \pm 50$ km |
| 2475          | 4.0    | $M_{\text{disag}} \pm 0.50$ | $D_{\text{disag}} \pm 50$ km |
| 4975          | 4.5    | $M_{\text{disag}} \pm 0.50$ | $D_{\text{disag}} \pm 50$ km |

*Figure 2* Conditional spectrum AvgSa-based record selection performed for site 2, considering the 475-year return period. The 30 green lines are the RotD50 response spectra of selected ground motions, while their average is represented by the blue line. The red lines represent the target conditional spectrum (average and average ±2 standard deviations) (D1.3, Infra-NAT 2018).

**Table 5.** Metadata of the accelerograms selected for site 2, considering the 475 years - return period. The reference AvgSa value (for the range of periods 0.2-1.0s) is 0.54 g, the mean magnitude from disaggregation is 6.18 and the mean distance from disaggregation is 21.23 km. The recording sites are characterized by $180 \, \text{m/s} \leq V_{30} < 360 \, \text{m/s}$, corresponding to Eurocode 8 soil category C (D1.3, Infra-NAT 2018).

| #  | Record ID (NGA) | Magnitude | Distance | SF  |
|----|-----------------|-----------|----------|-----|
| 1  | 4126            | 6.0       | 7.17     | 0.44|
| 2  | 4105            | 6.0       | 14.94    | 0.70|
| 3  | 3856            | 6.2       | 41.57    | 1.51|
| 4  | 1658            | 6.05      | 24.55    | 2.70|
| 5  | 329             | 6.36      | 58.53    | 2.09|
| 6  | 4112            | 6.0       | 6.94     | 0.65|
| 7  | 4111            | 6.0       | 6.96     | 0.59|
| 8  | 4081            | 6.0       | 13.76    | 0.77|
| 9  | 629             | 5.99      | 31.11    | 1.99|
| 10 | 4102            | 6.0       | 12.17    | 0.57|
| 11 | 627             | 5.99      | 26.21    | 1.23|
To select bridges for investigation, an extensive range of national bridge inventory (NBI) data for N. Macedonia developed by Vitanova (2015) (D2.2 and D2.3, Infra-NAT 2018) was used. This method of selecting a bridge prototype is recommended in the modeling practices from existing publications (Nielson, 2005), Choi, 2002).

Fig. 3 Cumulative distributive function of max span, pier height and deck width of typical bridge structures (Vitanova, 2015) (left) and mean and standard deviation of the characteristic values for bridges (right) analyzed in the study.

A detailed review of concrete girder bridges in NBI shows that simply supported multi-span single pier bridges account
for approximately two thirds of all bride structures (Vitanova, 2015), Fig. 4.

![Bridge type distribution](image)

Fig. 4 Typical concrete bridge structural systems in N. Macedonia, (SSCGB – simply supported concrete girder bridges, CCGB – continuous concrete girder bridges, FCB – frame concrete bridges, ACB – arch concrete bridges).

![Number of columns and spans](image)

Fig. 5 Number of bridges with particular number of columns in the pier (left) and no. of spans (right), (Vitanova, 2015).

The bridges for the analytical investigation in this study were chosen according to the parameters obtained following detailed research. Three types of concrete girder bridges with 3, 4 and 5 spans were analyzed. The characteristics of the selected types are given in Tab. 6.

| Parameter                  | Type 1-3S | Type 2 - 4S | Type 3 - 5S |
|----------------------------|-----------|-------------|-------------|
| Span length [m]            | Lower: 16.7 Middle: 23.6 Upper: 30.5 | Lower: 16.7 Middle: 23.6 Upper: 30.5 | Lower: 16.7 Middle: 23.6 Upper: 30.5 |
| Deck width [m]             | 7.2       | 10.4        | 13.6        |
| Pier height [m]            | 4.9       | 7.6         | 10.3        |
| Soil types                 | A, B, C   | A, B, C     | A, B, C     |
| No. of spans               | 3         | 4           | 5           |
| Superstructure radius      | 20, 30, 45, 90, ∞ | 30, 45, 60, 120, ∞ | 30, 45, 90, 135, ∞ |
| No. of samples             | 20        | 20          | 20          |

Fig. 6 shows the plan, elevation, and member cross section of the selected bridge. The longitudinal and transverse
directions are along the global x-axis and z-axis (perpendicular to the x-axis), respectively. The radial and tangential directions correspond to the local 1-axis (along the bridge chord) and 3-axis (orthogonal to the chord), respectively. 3D computational models were created in OpenSees (McKenna et al., 2010) following the latest modeling practices published in existing studies (Seo et al., 2013; Metngalathu et al., 2018; Amjadian, M. & Agrawal, AK., 2017). The implementation of OpenSees for seismic bridge analyses is easy and practical. The modeling technique for the bridge structures that was used in this study was selected due to the acceptable accuracy reported in literature (Borzi et al., 2015). The deck and the girders were modeled by use of elastic frame elements (Fig. 7). The piers were modeled as beam with hinges elements (the flexibility-based element with internal elastic portion and two nonlinear end hinges with Gauss-Radau integration by Scott and Fenves (2006).

The deck connections were modeled as zero length elements, while for the bearing devices within super- to sub-structure connections, two node link elements were used. The connections were performed through elastomeric bearings, as standard elements of this type of bridges (Nielson and Des Roshes, 2007). The behavior of the elastomeric bearings is shown in OpenSees by use of a zero-length element with a bilinear model for material behavior in horizontal direction, defined by elastic stiffness, yielding stress, yielding displacement, and final displacement. Beam with hinges inelastic elements were used for the piers. They were discretized into fibre elements, each of which having its own stress-strain
relationship and being able to be used to model the cross-section of the column with its confined and nonconfined concrete regions as well as longitudinal steel reinforcement (Yassin, MHM.,1994; Tavares, et. al.,2012). Concrete material type Concrete01 using the Scott-Kent-Park concrete model, and the bilinear steel model Steel01 with uniaxial bilinear steel material object with kinematic hardening (OpenSees library) were used for the elements. The stress-strain law for confined and unconfined concrete was defined using the law presented by Cusson and Paultre (1995) to determine the input parameters for the concrete with a linear tension model in OpenSees. The dimensions of the column section were different from the deck width of the structure. The pier section width was 55%, while the pier height was 10% of the deck width (Fig. 6), which reflected the reasonable range observed in the bridge inventory analysis.

There was a rigid link connection between the pier or the abutment base node and the foundation base node. Due to the rigid connection between these two nodes, the inertia of the foundation base affected the structural pier. The lower point was connected through a zero length element with the equivalent node with a fixed support (Borzi, et al.,2015). The zero length element was replaced by a kinematic body constraint.

Fig. 7 Generic 3D bridge model.

**DAMAGE ESTIMATION**

Various number of damage states has been considered in different studies. HAZUS (FEMA, 2013), Mander and Basoz
(1999); Du et al. (2020), accepted five, Mackie and Stojadinovic (2007) and Franchin and Pinto (2009) accepted three and Borzi. et al. (2015) accepted two limit states, namely, damage and collapse. It is difficult to define the association between physical damage and numerical response thresholds. In this study, two levels of capacity were envisaged for verification: one concerning the functionality (damage limit state) and the other concerning the safety of the work (collapse). An exception to this rule is the fragile shear mechanism on the pile, for which only the collapse limit state is defined. The deck is considered to behave linearly during an earthquake so that it is less vulnerable and is not to be considered in a vulnerability analysis. Elements which are the most vulnerable are piers because of the exceedance of shear strength or deformation capacity due to chord rotation as well as bearings where unseating of the substructure may occur due to excessive displacement demand. If the bearings failure represents simple falling of the pier cap from the deck, the damage limit state is noticed, but if the deck loses full support, than is considered a collapse.

The displacement capacity of the bearings is deterministically known and it is gained from the geometric characteristics of the pier cap and bearing seats. The capacity thresholds of RC pier components are modeled as lognormal random variables. The medians for the flexure are obtained from the unbiased model shown in details in Biskinis and Fardis (2010 a,b), specified for assessment in Eurocode 8, Part 3 (EC8-3). For flexure-controlled failure, the ultimate chord rotation is obtained from

$$\theta_u = \theta_y + (\varphi_u - \varphi_y) L_p \left(1 - \frac{L_p}{2L_s}\right)$$

(3)

where$(\varphi_u - \varphi_y)$ is plastic component of the ultimate curvature, $L_p$ is plastic hinge length, $L_s$ is shear span ($M/V$ ratio).

For cyclic loading and proper detailing for earthquake resistance, the plastic hinge length is calculated from

$$L_p = 0.1L_v + 0.17h + 0.24\frac{d_vr_y}{f_c}$$

(4)

Where $h$ is the depth of the cross section, $d_v$ is the (mean) diameter of the tensile reinforcement. Such expressions of capacity are evaluated separately for the longitudinal and transverse direction of the base segment of each pier.

For the chord rotation, two threshold levels are considered: yielding $\theta_y$ and ultimate $\theta_u$. Yield and ultimate curvatures are defined from a bilinear fit of a section moment-curvature analysis dealing with general cross-section shapes and reinforcement layouts.

For the bridge piers, a brittle shear failure is assumed (Maekawa & An, 2000), i.e., when the shear capacity of an RC
section is exceeded by seismic demand, an abrupt failure takes place. Hence, only the collapse limit state is defined for the shear capacity of the piers. Shear capacity of piers represents the sum of concrete shear strength, axial force and transverse steel that is computed according to the model published in Biskinis and Fardis (2010 a,b).

To explain the bi-directional response of the bridge excited by a multi-component seismic input, the risk indicator (RI)-ratio between the demand and the capacity for each time step is defined within the meaning of one-directional ratio as a combination of SRSS (elliptical shape of resultant) for the piers and maximum displacement (rectangular shape) for the bearings. The experimental evidence of detecting failure in bending/shear is presented in Biskinis and Fardis (2010 a,b).

The flexural deformation of a bridge pier in a collapse limit state is presented within the meaning of demands and capacities in two directions (longitudinal and transversal), concerning the component local reference:

\[
RI = \sqrt{\left(\frac{\theta_{DL}}{\theta_{CL}}\right)^2 + \left(\frac{\theta_{DT}}{\theta_{CT}}\right)^2} \quad (5)
\]

Calculation of the risk indicators for each limit state is verified for each component of a structure subject to the risk of damage or collapse. Each risk indicator depends on five clues: (1) return period of seismic action, (2) analyzed signal, (3) signal time step, (4) structural element (pier or bearing) and (5) direction. Directional combination of risk indicators is calculated according to two rules:

for piers \[
RI_{1,2,3,4} = \sqrt{RI_{1,2,3,4,x}^2 + RI_{1,2,3,4,y}^2} \quad (6)
\]

for bearings \[
RI_{1,2,3,4} = \max\left\{\left|RI_{1,2,3,4,x}\right|, \left|RI_{1,2,3,4,y}\right|\right\} \quad (7)
\]

The maximum risk indicator from all component indicators for every limit state is separately adopted as a representative risk indicator for the structure, where the maximum is calculated by taking the absolute maximum among various components for various mechanisms, checking every step of each single simulation.

This methodology was applied for defining the seismic risk of the road systems in the framework of the WP4 and WP5 in three partner countries (Italy, N. Macedonia and Israel) in the framework of the Infra-NAT project 783298 - UCPM-2017-PP-AG (www.infra-nat.eu), founded by the European Union, Civil Protection. The same methodology can be
RESULTS AND DISCUSSION

Inelastic time history analyses were performed for three, four and five span curved bridges located on A, B and C soil type conditions (according to the Eurocode). A total number of 300 bridge structures covering each number of spans and having different geometrical characteristics were analyzed. Hysteretic behaviour of nonlinear two node link bearing elements and nonlinearity of beam with hinges pier elements controlling the flexural deformation and shear strength were used to compute the risk indicator in each case of seismic motion by use of damage models.

Figs. 8 and 9 show the mean values of RI for representative models of all three types of structures analyzed in this study. These refer to the relationships of structural elements as follows: 1) for the piers: a total chord rotation that represents a ductile mechanism and shear deformation that represents a fragile mechanism; 2) for the decks: the loss of support mechanism towards the supports or the pier cap, controlled by the relative excursion of the support devices.

Fig. 8 Chord rotation of piers ($\rho$), shear deformation (V), unseating of bearings ($u$) and overall risk indicators for damage limit state (DLD) and collapse limit state (CLS) for 3 (left) and 4 (right) span bridges.
Fig. 9 Chord rotation of piers (ρ), shear deformation (V), unseating of bearings (u) and overall risk indicators for damage limit state (DLD) and collapse limit state (CLS) for 5-span bridges

From the presented above, it can be concluded that, in both limit states, the greatest effect is exerted by the unseating of the bearings. The chord rotation of the piers has a considerably lower effect upon the behavior of bridges regardless the number of spans and type of soil upon which they are founded. The piers are not critical elements in this type of bridges since they are sufficiently flexible and do not cause damage, i.e. failure of these structures. The great effect of curvature radius on these bridges, particularly those founded on soil of weaker bearing characteristics, is evident. On the other hand, the number of spans has a considerable influence upon RI when chord rotation of the piers is considered, particularly in the case of soil type A. There is not much difference in the effect of curvature radius in the case of bridges founded in soil of a better quality, particularly bridges with a small number of spans. For bridges with 3 spans, the considered coefficient has a lower value in the case of soil type C. For multi-span bridges founded on lower quality soil, it shows a considerable difference in the value of RI, meaning that multi-span bridges exhibit considerably less favorable behavior during an earthquake if founded on a weaker soil. The shear deformation of the piers has the greatest effect upon the behavior of bridges with smaller number of spans. In bridges with a greater number of spans, the shear deformation effect is negligible. The curvature radius has an exceptional effect on RI particularly when multi-span bridges founded on weaker soil are at stake.

The results from the study show that the soil conditions, curvature radius, total length of the structure, pier height and deck width have a significant influence on the risk indicator value, especially for the collapse limit state. The mean values for damage and collapse limit states (DLS and CLS, respectively) are presented in the subsequent text. The dependence between these parameters for DLS and CLS and bridge type 1 founded on soil type A are given in Fig. 10. The left diagram shows that, no matter what curvature radius the bridge has and how long the bridge is, RI has an approximate value particularly for straight bridges and bridges with the biggest curvature radius. This is shown by the almost horizontal trend of change of direction of the indicator in respect to the total bridge length. On the other hand, with the increase of the bridge length, particularly in the case of bridges with smaller curvature, this parameter acquires a lower value. This means that the damage to the considered 3-span bridges funded on soil type A does not depend on the respective curvature radii.
The situation is slightly different in the case of the collapse limit state. (Fig. 10, right). For this limit state, bridges of the same length, but different curvature radius, show very similar value of RI. The length of the structure directly influences the risk indicator, particularly in the case of straight bridges, where the risk indicator acquires a higher value much faster with the increase of spans. It can also be concluded that, the lesser total length of bridges, the more they exhibit similar behaviour under an earthquake. Bridges of smaller length show similar behaviour under an earthquake, i.e., the increase in length leads to more unfavourable behaviour.

Fig. 10 Mean risk indicator for the damage limit state (left) and collapse limit state (right) in function of the total length of the three span bridge structures founded on soil type A with trend directions

The same conclusions can be drawn for the mean values for the damage limit state for soil types B and C, so that they are not shown herein. Fig. 11 and Fig. 12 show only the mean risk indicator for the collapse limit state in function of the total length of three span bridge structures funded on soil type B and C, for bridge type 2, and A and C for bridge type 3. From these figures, it can be seen that the mean values for the risk indicators for soil type B and C are similar for bridge type 1 for the shortest bridges. The tendency of the mean risk indicator for bridges founded on soil type B is similar to that for soil type A, but different for soil type C. Higher values for the mean risk indicator for collapse occur in longer curved bridges with smaller radius of curvature.

Having in mind the above, the following conclusions can be drawn for bridge type 1:

- Straight bridges have the highest mean values of RI for both limit states. For the damage limit state, these values differ more than those for the collapse limit state. In this paper, only the DLS for soil type A is presented.
- Regardless the curvature radius, the risk indicator for the damage limit states does not show significant variation with length deviation.
- For the worst soil conditions (soil type C), the length of the structure has a considerable influence on the risk indicator change trend, so that for longer bridges, the risk indicator has a higher value.
Fig. 1 Mean risk indicator for the collapse limit state in function of the total length of three span bridge structures founded on soil type B (left) and C (right), with trend directions.

In the case of bridge type 2, the situation with the risk indicator for the collapse limit state is different (Fig. 12). The values for the risk indicator are higher than those for bridge type 1. Straight bridges have higher values, whose variation is greater in the case of weaker soil types. Longer bridges with the largest curvature radius are characterized by lower values of the risk indicator, meaning that they exhibit a less favorable behavior under an earthquake.

Fig. 12 Mean risk indicator for the collapse limit state in function of the total length of four span bridge structures founded on soil type A (left) and C (right), with trend directions.

Fig. 13 shows the mean risk indicator for collapse in function of the total length of bridge type 3 (five spans, 5S) founded in different soil conditions. These figures show that the radius of curvature has a very high influence on the risk indicator especially in good soil conditions. With the increase of the length of a structure, the risk indicator gets a higher value for all curvature radii of the superstructure. The highest values are characteristic for straight bridges regardless the soil conditions. A greater difference among RI is noticed in bridges founded in good soil conditions, meaning that, in better soil conditions, the difference in behaviour of bridges with different radius is greater. In that sense, straight bridges and bridges with a greater radius behave less favourably than bridges of a smaller radius.
Fig. 13 Mean risk indicator for the damage limit state (a) and collapse limit state (b) in function of the total length of four span bridge structures founded on soil type B, with trend directions.

The mean value for the collapse limit states for bridges founded on soil type B is a little bit lower compared to that of bridges founded on soil type A (Fig. 14) for all curvature radii, whereat regardless the curvature radius, the behaviour of the bridges changes with the increase of the total bridge length.

In accordance with the above stated, the following conclusions can be drawn:

- A change of the mean RI with the change of length of beam bridges is observed, but it does not have greater effect on RI for DLS;
- Straight bridges behave differently under earthquakes compared to bridges in curvature, particularly those with more spans;
- The difference in behavior of bridges with a larger curvature radius is much greater when the soil conditions are better;
- The behavior of bridges of smaller length and any curvature radius is less different than the behavior of bridges of a greater length. This particularly refers to multi-span bridges founded on soil of better characteristics.

The height of the central piers considerably affects the behavior of bridges particularly if founded on weaker soil. Fig. 14 shows the RI relationships for DLS of the height of the central piers of bridges type 3. From the presented diagrams, it can be concluded that bridges with greater pier height and curvature radius exhibit less favorable seismic behavior than bridges with smaller curvature radius founded on better soil. In the case of straight bridges, with the increase of the pier height, the mean value of RI tends to decrease, i.e., the bridges tend to behave more favorably. For bridges in curvature, the greater height of the piers contributes to their less favorable behavior.
Mean risk indicator for the damage limit state for bridge type 3 founded in soil type A (a) and C (b) in function of the pier height.

The same can also be concluded for the effect of the deck width (Fig. 15). With the decrease of the bearing characteristics of the soil, higher values of RI are observed, meaning that bridges founded on weaker soil exhibit less favorable seismic behavior regardless the curvature radius and the deck width. For any curvature radius and any soil characteristics, the behavior of the bridge becomes more favorable with the increase of the deck width.

Mean risk indicator for the damage limit state for bridge type 3 founded on soil type A (a) and C (b) in function of the deck width.

Fragility curves, defined as cumulative probability distributions that allow estimation of the probability of reaching or exceeding a given level of damage for a given severity of ground shaking, for each boundary state, are given below. For the calculation of the fragility curves, six return periods of the reference input motion were adopted and, for each of them, 30 simulations were carried out.

Fig. 16 shows the vulnerability curves for DLS for bridges type 1 and different soil conditions. Fig. 16 a), b) и c) show comparative diagrams of vulnerability of each type of soil taken separately. From these, one can observe a great
difference in the probability of reaching a certain level of damage for a certain level of PGA (g) for bridges with different radii. Particularly great difference is observed regarding the probability for soil type C where, for 0.4 g, the probability for damage to straight beam bridges is 2%, while for bridges with curvature radius of 90°, it is 60%. Under the same PGA, for a bridge founded on soil type B, the probability for damage is 38%, whereas for soil type A, it is only 10%. If direct comparison of the fragility functions for the same curvature radius and different soil conditions is made, (Fig. 16 d), e), g)), it can be seen that the difference between the vulnerability of structures founded on soil type B and C is less significant than that for soil type A.

![Fragility curves for bridge type 1, for DLS, different soil conditions and curvature radius](image)

Fig. 16 Fragility curves for bridge type 1, for DLS, different soil conditions and curvature radius

Fig. 17 shows the fragility curves for CLS for bridge type 1. Fig 17 a), b) and c) show the part of the fragility function for the collapse limit state for various soil types and different curvature radii. The diagrams show that straight bridges founded on soil type B and C are significantly less vulnerable than those founded on soil type A. The same situation holds also for the other bridge types, so that they are not presented in this paper.
Fig. 17 Fragility curves for bridge type 1 for CLS, different soil conditions and curvature radii.

Fig 18 and Fig. 19 show the fragility curves for DLS for bridge type 2 and bridge type 3, respectively. From all the presented diagrams, it can be concluded that the difference in behavior of bridges is greater in the case of bridges with a larger curvature radius and bridges founded on soil of less favorable characteristics.

Fig.18 Fragility curves for bridge type 2 for DLS, different soil conditions and curvature radius.
The difference in the behavior of straight bridges is greater than that of bridges in curvature, particularly in the case of bridges founded in less favorable soil conditions. Straight bridges exhibit the most favorable behavior, with the least probability for damage and failure.

![Fig. 19 Fragility curves for bridge type 2 for DLS, different soil conditions and curvature radius.](image)

**CONCLUSIONS**

Presented in this paper is a research of the effect of span length, deck width, pier height, local soil conditions and radius of horizontal curvature on seismic vulnerability of multi-span beam bridges. To perform the research, a typical bridge structures existing in R.N. Macedonia – a multi-span RC beam bridge was selected. The selection was done based on an extensive range of national bridge inventory (NBI) data. The span length, the height of the central piers and the width of the structures were obtained by statistical analysis and these were varied between their mean value plus minus one standard deviation. All bridges were analyzed in different soil conditions and with different curvature radius. To define the seismic behavior of the structure, the risk indicator (RI) representing the demand and bridge capacity ratio was defined and calculated for two limit states (LS), namely the damage limit state and the collapse limit state, through which the behavior of the structures was defined.
The variations in the response of the structures depended on the span length, the horizontal curvature and the local soil conditions. From the results obtained from the comparative fragility curves for different curvature radii, length and type of soil and the values obtained for RI, the following can be concluded:

1. With the increase of the number of spans and curvature radius of the superstructure, bridges become more vulnerable, particularly if founded on soil of weaker characteristics. Components that mainly affect the vulnerability of structures are the bearings with their unseating regardless the number of bridge spans, span length and soil conditions in which the structures are founded. The largest displacements occur in bridges with a greater number of spans and a greater curvature radius founded in weaker soils.

2. In the considered type of beam bridges, regardless the number of spans and soil type in which they are founded, the piers are not critical elements and do not affect their seismic vulnerability. The chord rotation that occurs under the earthquake effect is not significant and enables these bridges to be sufficiently flexible so as not to suffer damage, while shear deformation that occurs is small and does not cause failure.

3. The number of spans considerably affects the ductility of the structures, particularly when these have a greater number of spans and are founded in soil of weaker characteristics. The flexibility of bridges with a smaller number of spans is not much different regardless the curvature radius.

4. The shear deformations of the piers considerably depend on the number of bridge spans, but not so much on the soil type in which the structure is founded and its curvature radius. Larger shear deformations occur in multi-span bridges.

5. Structures with smaller curvature radius are characterized by a lesser difference in seismic behavior irrespective of their length. The greatest variation of behavior is observed in bridges with a larger substructure radius. Structures with a greater curvature radius and greater length are considerably more vulnerable particularly if founded in unfavorable soil conditions.

6. The behavior of structures with a smaller number of spans, regardless the local soil conditions, the total length and the curvature radius, is less different than that of multi-span structures.

7. For the worst soil conditions, (soil type C), the length of the structure has a considerable effect upon the risk indicator change trend. Hence, in the case of longer bridges, the risk indicator has a higher value, meaning that the structures exhibit less favorable behavior.
8. Bridges of a greater pier height and greater curvature radius are characterized by considerably more unfavorable seismic behavior than bridges of a smaller curvature radius founded in better soil. In the case of straight bridges, with the increase of the pier height, the mean value of RI tends to decrease, i.e., the bridges tend to behave more favorably. For bridges in curvature, the greater pier height contributes to their more unfavorable behavior.

9. Regardless the superstructure curvature and the quality of soil on which a bridge is founded, its behavior becomes more favorable with the increase of the deck width.

The developed fragility curves presented in this paper point out the need for consideration of the effect of the local soil conditions, the bridge structure length, the horizontal curvature radius and the number of spans of different types of bridges. This paper shows realistic fragility curves for the selected bridge configurations that can be used for loss estimation for the selected type of bridge structures. However, further research is necessary to develop fragility curves for other bridge configurations that would take into account the same and extended parameters in the analysis.

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