

NONLINEAR DYNAMIC ANALYSIS OF GIRDER BRIDGE DESIGNED IN ACCORDANCE WITH EN 1998-2 EXPOSED TO EARTHQUAKE

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Summary: *Methods used for the design of structures in seismically active areas are based on structural behaviour assessment. Basic elements of such analysis are seismic hazard and seismic demand assessment of the structure. For the design of reinforced concrete structures, linear elastic models combined with response spectra (EN 1998-2) are applied. These methods provide a good estimate of seismic forces, but are insufficient in determining the displacements and deformations, which are crucial for the post-elastic behaviour that depends on the provided balance between stiffness, bearing capacity and ductility. In this paper, nonlinear dynamic analysis for the assessment of seismic behaviour of reinforced concrete girder bridge designed in accordance with EN 1998-2 is presented. Seismic excitation is introduced by applying accelerograms corresponding to the designed seismic action.*

Keywords: *Nonlinear dynamic analysis, RC girder bridge, seismic response*

1. INTRODUCTION

Primary objective of designing structures in seismically active areas is to ensure the appropriate safety and performance level of the structure during and after earthquakes. [1] When it comes to city bridges, as well as bridges on vital roads, aseismic design should provide adequate functionality after earthquakes. The level of functionality primarily depends on the strength of the earthquake, philosophy and design quality at all stages of development and performance criteria. It is common, based on their importance, for bridges to be classified in specific categories, which would determine the risk factors depending on the return period of the designed earthquakes.

Basic method for determining the seismic action in structural design is through the application of response spectra. The alternative methods for assessing the seismic performance of structures include the implementation of artificial and/or real

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accelerograms. Basic rules for time-history representation of seismic actions, prescribed in EN 1998-2 [2], include the usage of at least three pairs of horizontal ground motion components, from recordings consistent with the designed seismic action. If such recordings are not available, modified recordings or simulated accelerograms may replace the missing recorded motions, by scaling the amplitude of motions from the known recordings. For each component of horizontal motion, the SRSS spectrum is determined by taking the square root of sum of squares of the 5 % damped spectrum. The ensemble spectrum is afterwards formed by taking the average value of the SRSS spectra of the analyzed earthquakes, and then scaled, taking into consideration that it is not lower than 1.3 times the 5 % damped elastic response spectrum of the design seismic action, in the period range between 0.2 and 1.5 times the natural period of the fundamental mode of the structure in case of a ductile bridge.

2. NUMERICAL EXAMPLE

A continuous three span reinforced concrete (RC) bridge structure (55 m in total length) is analyzed (Fig. 1). Concrete class C30/37 is adopted, as well as reinforcement S500 (class C). Torsion rigidity of the deck is reduced to 50 % of the homogenous cross section, considering the development of diagonal cracks resulting from the main tension stresses. While the deck is supposed to remain in elastic response range, the main elements resisting seismic forces are columns. They are of different height (14 m and 7 m), with a single solid circular cross section of diameter 2.0 m. Column cross section consists of the concrete cover to the reinforcement, modelled as the unconfined concrete [1], while the part of the section surrounded by the transverse reinforcement is modelled as the confined concrete. [2]

A ductile seismic behaviour of columns is expected, which is introduced through the adopted behaviour factor $q = 3.5$.

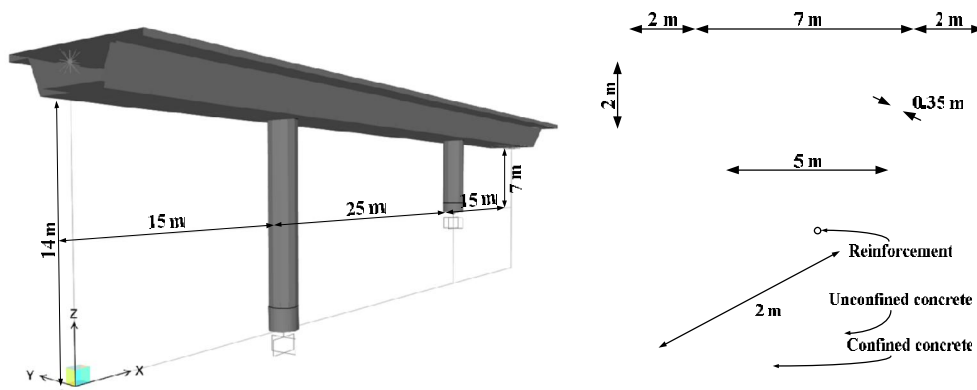


Figure 1. Structural model of RC girder bridge and cross-sections of the deck and columns

Structural analysis was conducted using the FEM software SAP2000 v15.2.1. [3] Self weight is generated by the software, based on the cross-section dimensions and the density of the concrete material. The additional permanent load on the deck is assumed as $\Delta g = 20 \text{ kN/m}$. Dynamic model of the deck consists of line FE, 2.5 m in length. Bridge mass is concentrated in the nodes in proportion to the length of the segments.

The same reinforcement was adopted for both columns, in accordance with EN 1998-2. The longitudinal reinforcement $64\text{Ø}25$ (reinforcement ratio $\rho = 1.0 \%$), determined for the shorter column (greater stiffness), is also adopted for the longer column, while the axial force of the two columns varies insignificantly. The transverse reinforcement is one spiral $\text{Ø}16/70$ (reinforcement ratio $\rho = 1.2 \%$). Force-based seismic design for ductility, implicitly assumes that members which develop inelastic deformations and ductility, have a nearly bilinear monotonic force-deformation behaviour, close to elastic-perfectly-plastic. The elastic stiffness used in the analysis should correspond to the stiffness of the elastic branch of a bilinear force-deformation response of a ductile member. When the actual monotonic force-deformation curve of a member expected to yield under the design seismic action is approximated as bilinear, the analysis should use as elastic stiffness the secant-to-yield point flexural stiffness, which applies in particular to columns in bridges designed for „ductile behaviour“ (i.e. $q > 1.5$).

The effective moment of inertia I_{eff} of a column of constant cross section is estimated as:

$$I_{eff} = 0.08 \cdot I_{un} + I_{cr} \quad (1)$$

where I_{un} is the moment of inertia of the gross cross section of the uncracked column and I_{cr} is the moment of inertia of the cracked section at the yield point of the tensile reinforcement, which may be determined from the expression:

$$I_{cr} = M_y / (E_c \cdot \Phi_y) \quad (2)$$

in which M_y and Φ_y are the yield moment and curvature of the section respectively and E_c is the elastic modulus of concrete. According to expressions (1)-(2), the effective stiffness of the columns is reduced to 37.1 % and 36.8 %, for longer and shorter column, respectively. Modal analysis is conducted for the first five modes, which in the sum of effective modal masses capture more than 90 % of the total mass. The first two fundamental periods are $T_1 = 0.712 \text{ s}$ and $T_2 = 0.205 \text{ s}$.

3. NONLINEAR DYNAMIC ANALYSIS

The assessment of the structural behaviour in the post-elastic region, for the events of strong ground motion, is possible only through nonlinear analysis. [4] Seismic action for nonlinear dynamic analysis of RC girder bridge is introduced through seven pairs of artificially generated accelerograms corresponding to the design seismic action. Each pair of records represents a different earthquake. Vertical component of ground acceleration is neglected in analyses. The ensemble spectrum of pseudo-acceleration for all the applied accelerograms is given in Fig. 2.

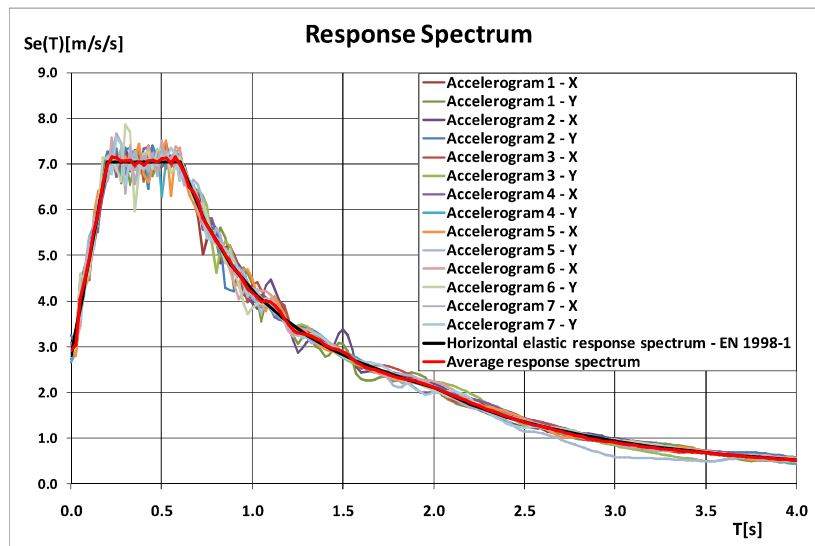


Figure 2. Response spectrum

Material nonlinearity in the plastic hinge region is introduced through the fiber model of the cross-section, consisting of the confined part of the section (core of the column, EN 1998-2), unconfined part (protective concrete cover to the reinforcement, EN 1992-1-1) and the adopted longitudinal reinforcement (see Fig. 1). Stress-strain relations for confined and unconfined concrete parts of the cross-section are presented in Fig. 3.

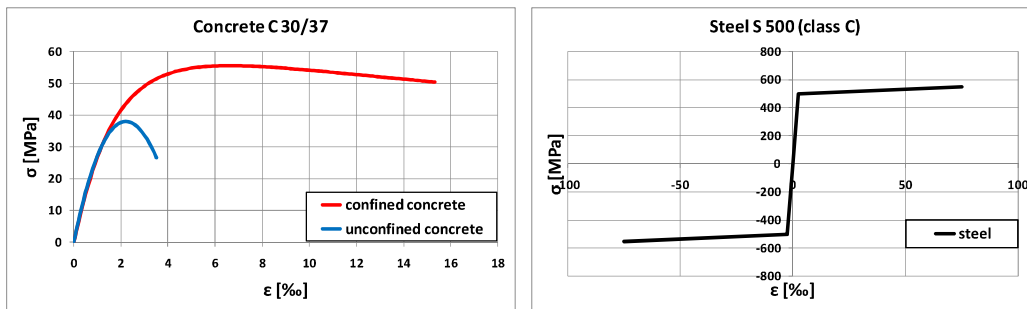


Figure 3. Stress-strain relations for concrete C30/37 and steel S500

4. RESULTS

Achieved local ductility μ_d in the critical cross-sections of the bridge columns can be estimated according to the rotation of the plastic hinge at the yield point θ_y and maximum achieved rotation θ_d for a given earthquake. Based on the results of nonlinear dynamic analysis, it can be concluded that inelastic deformations occurred only in the critical sections of the shorter column. Table 1 presents results of the achieved local ductility in the critical cross-sections of the shorter column of the bridge for the estimated rotation at the yield point $\theta_y = 0.002123$ rad.

Table 1 – Achieved local ductility in the plastic hinge of the shorter column of the bridge

	Longitudinal direction		Lateral direction	
	θ_d [rad]	$\mu_d = \theta_d/\theta_y$	θ_d [rad]	$\mu_d = \theta_d/\theta_y$
Maximum value	0.006208	2.92	0.000514	< 1
Minimum value	0.002087	< 1	0.000200	< 1
Average value	0.004444	2.09	0.000393	< 1

Characteristic moment-rotation relation of the plastic hinge in the shorter column for longitudinal direction is presented in Fig. 4.

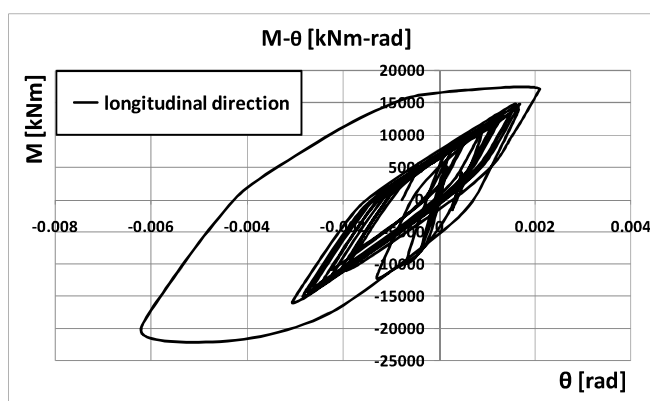


Figure 4. Moment-rotation relation (fiber model – shorter column – longitudinal direction)

5. CONCLUSIONS

According to EN 1998-2, the structure can be designed in a manner that limited level of damage caused by bending in pre-determined regions is allowed. Such design method is also known as the capacity design method. Basic concept of this method is to allow damage (i.e. plastic deformations) of the cross-sections due to bending in pre-defined critical regions, in order to dissipate the induced seismic energy. Damaging in RC girder bridges, i.e. plastification of the cross-sections is usually permitted in the bridge columns, while the bridge deck has to remain undamaged after the seismic action, in order to maintain the basic functions of the bridge (pedestrian traffic and transportation of the vehicles belonging to special services). The foundation structure and the abutments should remain undamaged because of the long and difficult repair.

Based on the results of nonlinear dynamic analysis, it can be concluded that the damage has occurred only in the critical regions of the shorter column of the bridge. Maximum value of the achieved local ductility in longitudinal direction is 2.92, which is less than the adopted behaviour factor $q = 3.5$. Maximum value of the achieved local ductility in lateral direction is less than 1.0, regardless of the adopted behaviour factor $q = 3.5$.

Based on the previous results, it can be concluded that the failure mechanism did not occur for any of the earthquakes analyzed.

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НЕЛИНЕАРНА ДИНАМИЧКА АНАЛИЗА ГРЕДНОГ МОСТА ДИМЕНЗИОНИСАНОГ ПРЕМА EN 1998-2 ИЗЛОЖЕНОГ ДЕЈСТВУ ЗЕМЉОТРЕСА

Резиме: Поступци који се користе за пројектовање грађевинских конструкција у сеизмички активним подручјима се заснивају на процени понашања конструкције. Основни елементи оваквих анализа су сеизмички хазард и процена сеизмичког захтева за конструкцију. За димензионисање армиранобетонских конструкција се примењују линеарно-еластични модели у комбинацији са спектрима одговора (EN 1998-2). Овим методама анализе могу се добро проценити сеизмичке силе али не и померања и деформације које су од кључног значаја за анализу пост-еластичног понашања које зависи од обезбеђеног баланса између крутости, носивости и дуктилности. У овом раду се за процену сеизмичког понашања гредног АБ моста димензионисаног према EN 1998-2 примењује нелинеарна динамичка анализа. Сеизмичка побуца се уводи у прорачун применом акцелерограма који одговарају пројектном сеизмичком дејству.

Кључне речи: Нелинеарна динамичка анализа, гредни АБ мост, сеизмички одговор