

CALCULATION OF CONTINUOUS BRIDGE RC COLUMNS ACCORDING TO ELM (FEMA 440)

Miloš Čokić¹
Radomir Folić²

UDK: 624.21.042.7

DOI:10.14415/konferencijaGFS2017.005

Summary: *In order to study the behaviour of the bridge under seismic actions, the Equivalent Linearization Method (ELM) was applied. The bridge is continuous with three fields, ranging 24 + 40 + 24 m, with prestressed concrete box girder of constant height. The Nonlinear Static Pushover Analysis (NSPA) was applied. Bridge columns, exposed to horizontal seismic effect, were subject of detailed analysis, using the Type 1 Response Spectrum for category B type soil, according to EN1998: 2004. The analysis also included a 20% of traffic load. Bridge columns were designed according to EN1992, parts 1 and 2. Result of the analysis are presented with pushover curves that describe the behaviour of elements under seismic actions in longitudinal and transverse directions.*

Keywords: *Bridge columns, seismic actions, Equivalent Linearization Method, EN 1998:2004, Pushover curves*

1. INTRODUCTION

This paper presents the results of the analysis of the bridge columns, under seismic action. The calculation method applied is the *Equivalent Linearization Method (ELM)*, one of *Nonlinear Static Pushover Analysis (NSPA)* methods, described in *FEMA 440*. The most of calculations were conducted in *CSI Bridge* software package and they include the construction of pushover curves and determination of the columns bearing capacity as a final result.

Structural design based on Capacity Design Method (CDM) gives the opportunity to model and predict the behaviour of the structure exposed to seismic action, while allowing the occurrence of plastic hinges in specific parts of the structure (in columns, not in deck).

The subject of analysis is prestressed continuous concrete bridge with 24+40+24 m spans (Figure 1). The main elements of the structure are: 4 reinforced concrete (RC) columns, their foundations and prestressed concrete girder that has rectangular box cross-section of constant height. Details of design and other bridge properties are shown in the paper [1].

Three different models were formed for the analysis of bridge columns behaviour while exposed to seismic actions. The same material characteristics (for concrete, reinforcement

¹ Miloš Čokić, MSc Civil Engineering, PhD student, University of Novi Sad, Faculty of Technical Sciences, Trg Dositeja Obradovića 6, Novi Sad, e – mail: cokicmilos@gmail.com

² prof. emeritus Radomir Folić, PhD, Dr.h.c., University of Novi Sad, Faculty of Technical Sciences, Trg Dositeja Obradovića 6, Novi Sad, e – mail: folic@uns.ac.rs

steel and prestressing cables) were adopted for all three models. The quality of concrete and reinforcing steel embedded in the entire structure, is C35/45 and B400/500 (RA). The characteristics of the materials used in analysis represent a constant parameter in all three models, and for that reason they do not affect the result deviations.

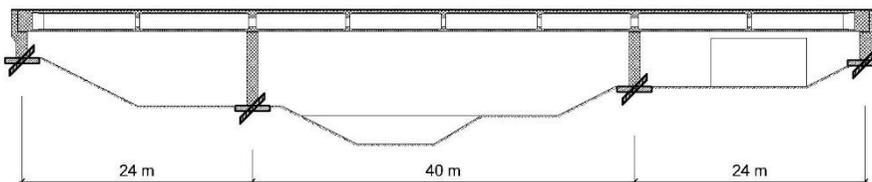


Figure 1. Bridge structural elevation according to EN1998-2 [3]

The loads included in calculations, for the purpose of seismic analysis are: dead load, prestressing load, vehicle load and seismic actions.

Unlike the calculations done in [1] according to Serbian regulations SRPS (ex JUS), the traffic load in this research is defined according to EN1991: Part 2 [2], where the Load Model 1 (LM1) was used. However, as defined in [3], 20% of equally distributed traffic load (for LM1) was included in seismic actions on the bridge.

It is presumed that the structure is located in the zone, where the effect of seismic action on the structure can be simulated using response spectrum function – EN1998-1: 2004 [3].

The spectrum type 1 was adopted. It is recommended for Eastern Europe region and the areas in which the earthquakes greater than magnitude 5.5 are expected. Ground type B was selected, which means that the structure is located in a very dense sandy or gravelly soil or clay or soil of higher strength. The damping ratio value is 0.05, according to [3]. The ratio of horizontal ground acceleration is $a_g/g \leq 0.4$, which is corresponding to IX degree of MCS scale.

According to [3], the bridge can be designed in that way to allow ductile or limited ductile (elastic in general) behaviour of structural elements under seismic actions. This behaviour is characterized by interdependence of shear forces and induced displacements. Behaviour factor q is adopted in dependence with the required behaviour of structure during the earthquake.

All three models (M1-M3) (Figure 2) were subjected to simulation and comparative analysis. Their mutual differences are reflected in the geometric characteristics of columns, as well as in the quantity of integrated reinforcement steel. Cross sections of 2 column models (M1-M2) are rectangular, while the cross-section of the columns in model M3 has circular shape. The dimensions and number of elements are shown in Table 1. The height of the left column, Column 1 (from the restraint point to the bottom of the bearing is 7 m, and the height of the right column (Column 2) is 5 m (Figure 1).

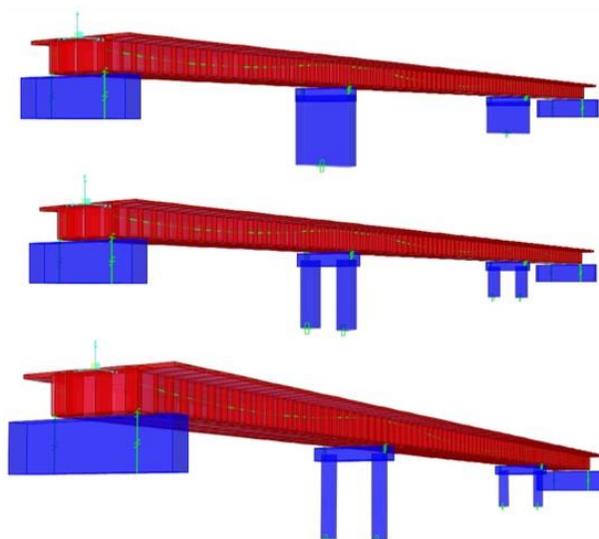


Figure 2. Bridge simulation models M1-M3

Dimensions of column's cross sections are reduced in order to comply with the required increase of ductility factor q . Models M2 and M3 are composed of two columns and the cap beam, and together they form the frame in the transverse direction. Columns with circular cross-section are adopted because they are more favourable for the acceptance of bi-axial bending moments.

Table 1. Geometric characteristics of concrete columns and reinforcement bars amount and profile type

| | M1 | | M2 | | M3 | |
|--|--------|--------|--------|--------|---------------------|--------|
| | Col. 1 | Col.2 | Col.1 | Col.2 | Col.1 | Col.2 |
| <i>Number of column elements: n_s</i> | 1 | 1 | 2 | 2 | 2 | 2 |
| <i>Column dimension in transversal direction: d [m]</i> | 6 | 6 | 1.4 | 1.4 | <i>Diameter [m]</i> | |
| <i>Column dimension in longitudinal direction: b [m]</i> | 1.2 | 1.2 | 1.2 | 1.2 | 1.2 | 1.2 |
| <i>Total rebar area: $A_{s,total}$ [cm²]</i> | 739.91 | 739.91 | 344.82 | 467.97 | 294.52 | 294.52 |
| <i>Shear reinforcement: \emptyset [mm/e]</i> | 12/10 | 12/10 | 10/10 | 12/10 | 10/10 | 10/10 |
| <i>A_s/A_c [%]</i> | 1.03 | 1.03 | 1.03 | 1.39 | 1.30 | 1.30 |

Table 1 shows geometric characteristics of concrete columns cross sections and reinforcement bars quantity and profile type. The concrete type that was used is C35/45, while the reinforcement bars are ribbed B400/500 (RA).

All columns are calculated in accordance with EN1992 - Part 1 [4] and 2 [4], using the combination for seismic effects given in EN1990: 2002 [5]:

$$\sum_{j \geq 1} G_{k,j} + P + A_{Ed} + \sum_{i \geq 1} \psi_{2,i} \cdot Q_{k,i} ; (\psi_{2,i} = 0.2) \quad (1)$$

5. PUSHOVER METHOD

Nonlinear static pushover method is a good alternative to complex nonlinear dynamic calculation method. Its application is very simple and provides an insight into the global structural behaviour as well as the identification of structural weaknesses of the structural system. [6]

Pushover curve represents a function of shear force and displacement (of the analyzed structural element or the whole structure itself), if it is reduced to a system with one degree of freedom. The general principle of pushover method is represented through gradual application of shear load to the structure, from zero value to the point of the targeted displacement or the capacity loss of the object, while the structures yield points are registered (*Figure 3*). It is necessary that a demanded displacement, caused by the earthquake and determined by using the proper Response spectrum, is less than total bearing capacity of structure.

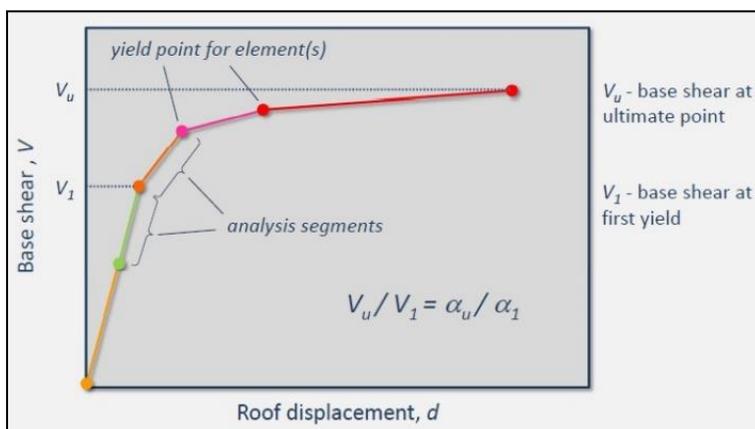


Figure 3. Pushover curve and its characteristic yield points, after [6]

The pushover curve point that represents the ultimate displacement capacity value (bridge columns in this case) can be determined by implementing one of the methods of nonlinear static pushover analysis (NSPA).

The methods differ from each other, but they have a common principle of bilinear approximation of pushover curve. Their application method is also specific – the multi-degree-of-freedom (MDOF) system is replaced by an equivalent system with one-degree-of-freedom (SDOF) system to determine the maximum displacement capacity of SDOF system, using the adequate response spectrum. Displacements of MDOF system can be obtained by applying the specific correction coefficients on the calculated SDOF displacements [7].

6. APPLICATION OF EQUIVALENT LINEARIZATION METHOD

When determining the displacement capacity of columns, the Equivalent Linearization Method (ELM) was applied, according to FEMA 440 regulations, whereby instead of adopting specific coefficients, the response spectrum function EN1998: 2004 was used for calculation, according to E1998 - Part 1 [3].

This method is called Modified Response Spectrum Method as acceleration and displacement function, also named Modified Acceleration Displacement Response Spectrum Method (MADRSM) which differ the procedures for determination of the displacement capacity: A, B and C [6].

The principles of this method are similar to the Capacity Spectrum Method (CSM) principles defined in ATC-40. According to ATC-40 CSM, to determine the maximum displacement (d_{max} - performance point), a secant oscillation period (T_{sec}) is used as an effective linear period. The values of maximum displacement are calculated by determining the point of pushover capacity curve and demand curve intersection, for the effective damping of the system. It has been shown, however, that the application of this method is not always sufficient to provide the accurate performance points [6]. Improvement of this method with the use of ELM, is shown in FEMA 440 [8].

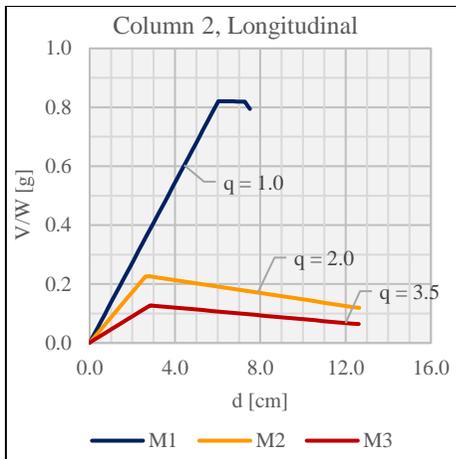
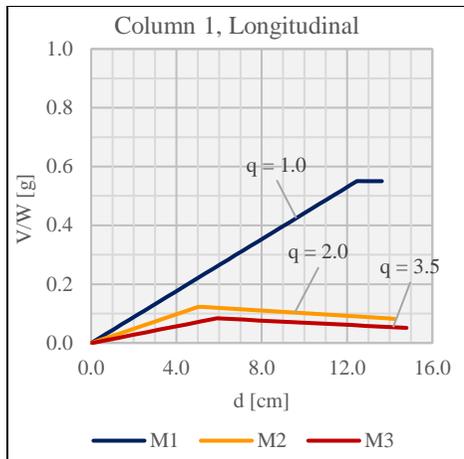
The value of maximum displacement is calculated by determining the intersection point of MADRS curve and demand curve. However, the results obtained by this procedure may be unreliable for ductility values that exceed the value of 10 to 12. [6]

7. RESULTS DISCUSSION

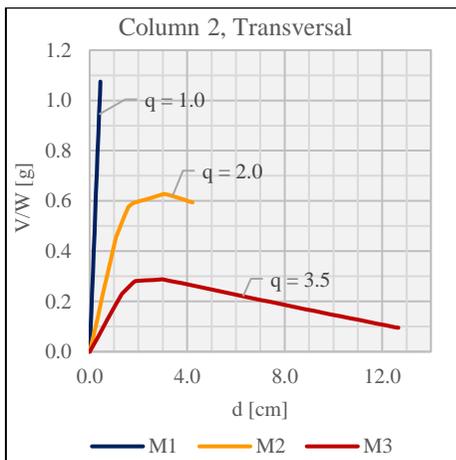
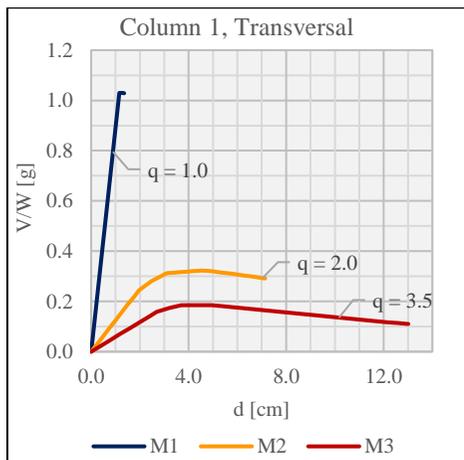
An overview of the pushover curves for both columns, for all three models, in longitudinal (*Figures 4 and 5*) and transverse (*Figures 6 and 7*) direction is displayed.

There is a notable pattern in behaviour of columns, in longitudinal and transverse direction, related to shear forces and displacement. By increasing behaviour factor q , the

capacity of column's displacements increases, i.e. the shear forces which can be received by the columns is reduced.



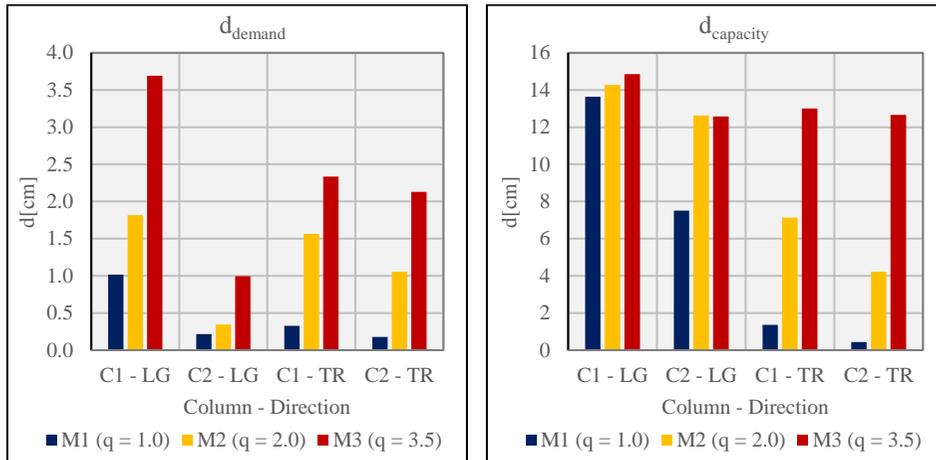
Figures 4. & 5. Pushover curves of bridge columns 1 and 2 for longitudinal seismic action



Figures 6. & 7. Pushover curves of bridge columns 1 and 2 for transversal seismic action

Columns displacements and ductility

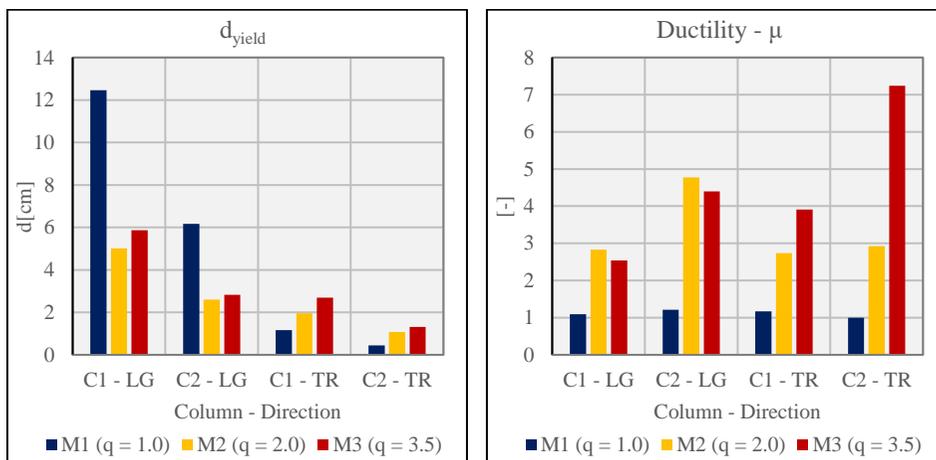
Based on the calculated values of the columns demanded displacement for seismic action (d_{demand}) (Figure 8) and calculated displacements at the yield point (d_{yield}) (Figure 10), it is possible to conclude whether the structure's joints will plastify or remain in the elastic region. It is necessary to satisfy the condition, where the demanded displacement must have a smaller value than the column maximum capacity displacement ($d_{capacity}$) (Figure 9).



Figures 8. & 9. Demanded and column capacity displacements

With the increase of behaviour factor value, $d_{capacity}$ will also have higher value (Figure 9). The same rule can be applied to the relationship between q factor and d_{demand} (Figure 8).

The difference in values of d_{demand} in transverse direction is more pronounced compared to the one's in longitudinal direction, which is a result of the adopted structural system and its characteristics (Figure 8). Displacements $d_{capacity}$ are almost equable for all models in the longitudinal direction for Column 1 and for models M2 and M3 for Column 2 (Figure 9). When comparing models in transverse direction, $d_{capacity}$ values expressed significant difference (Figure 9).



Figures 10. & 11. Columns yield point displacement and columns ductility

Ductility of columns is increasing in the transversal direction in the models M2 and M3 ($\mu_1^{TR} < \mu_2^{TR} < \mu_3^{TR}$), while the ductility of the M2 is higher than in M3 in the longitudinal direction ($\mu_1^{TR} < \mu_2^{TR}, \mu_3^{TR}; \mu_2^{TR} > \mu_3^{TR}$), which can be explained by the structural elements properties, which differ in ability of accepting seismic actions in two perpendicular directions.

Values of columns displacement at yield point (d_{yield}) are shown in Figure 10. It is noticeable that d_{yield} in M2 is less than in M3 ($d_{yield,2}^{LG,TR} < d_{yield,3}^{LG,TR}$) in both directions. Displacements at yield point in M1 are significantly higher in the longitudinal ($d_{yield,1}^{LG} > d_{yield,2}^{LG}, d_{yield,3}^{LG}$) than in the transverse direction ($d_{yield,1}^{LG} < d_{yield,2}^{LG}, d_{yield,3}^{LG}$), compared to the other two models.

The results of calculation indicate that the plastification of column joints will occur only in case of transverse seismic action on the M3 Column 2. Other bridge columns will have elastic behaviour under seismic action.

8. CONCLUSIONS

Analyzed bridge columns are designed in accordance with the methodology recommended in the FEMA 440 document, i.e. with the use of the Equivalent Linearization Method (ELM). The accuracy of the performance evaluation is less reliable, compared to the non-linear dynamic analysis (Time-History Analysis - THA), particularly if the irregularities of the bridge structure are more expressed. Considering that the analyzed bridge have a slight differences in column heights and relatively small spans, the conducted analysis is acceptable in practice, taking into the account the results of the comparative analysis of method applied in the paper and THA [10].

ACKNOWLEDGEMENTS

The work reported in this paper is a part of the investigation within the research project TR 36043 supported by the Ministry for Education and Science Republic of Serbia. This support is gratefully acknowledged (R. Folić).

REFERENCES

- [1] Čokić M., Petronijević P., Todorović M.S., Pecić N.: Analysis of the embodied energy and the CO2 emission in construction process of a bridge structure from the aspect of sustainability, *Building materials and structures*, № 2, 2015, p.p. 3-20
- [2] EN1991: Actions on structures - Part 2: Traffic loads on Bridges, CEN, September 2003
- [3] EN1998 - Part 1 & 2: Bridges, European Committee for Standardization (CEN), 2004/2005

- [4] EN1992: Design of concrete structures - Part 1-1 & Part 2 : General rules and rules for buildings; Concrete Bridges, CEN, 2004/2005
- [5] EN1990 - Basis of structural design, CEN, 2005
- [6] Lang D. H.: Seismic response estimation using non-linear static methods, *Indo-Norwegian Training Programme on Nonlinear Modelling and Seismic Response Evaluation of Structures*, December 14-16, 2014 – Continuing Education Center, IIT Roorkee
- [7] Manjula N.K., Nagarajan P., Madhavan Pillai T.M., A Comparison of Basic Pushover Methods, *International Refereed Journal of Engineering and Science (IRJES)*, ISSN (Online) 2319-183X, (Print) 2319-1821, Volume 2, Issue 5(May 2013), pp. 14-19
- [8] FEMA 440 IMPROVEMENT OF NONLINEAR STATIC SEISMIC ANALYSIS PROCEDURES, Prepared by: Applied Technology Council (ATC-55 Project), Prepared for: Department of Homeland Security, Federal Emergency Management Agency, Washington, D.C., June, 2005
- [9] Isaković T., Fischinger M.: Pojednostavnjene nelinearne metode proračuna betonskih mostova, *Građevinar* 61, 2009., pp. 625-633
- [10] Ger J., Cheng F.Y.: Seismic Design Aids for Nonlinear Pushover Analysis of Reinforced Concrete and Steel Bridges, CRC Press, Taylor & Francis Group, 2012
- [11] CSiBridge 2016, *Introduction to CSiBridge*, Computers & Structures, Inc., 1978-2015

PRORAČUN STUBOVA KONTINUALNOG AB MOSTA METODOM EKVIVALENTNE LINEARIZACIJE

Rezime: Za proučavanje ponašanja konstrukcije mosta pod seizmičkim dejstvom, primenjena je Metoda ekvivalentne linearizacije (*Equivalent Linearization Method – ELM*). Most je kontinualni sa tri polja, raspona 24+40+24 m, konstantnog sandučastog poprečnog preseka od prethodno napregnutog betona. Primenjena je Nelinearna statička pushover metoda (*Nonlinear Static Pushover Analiza – NSPA*). Detaljnije su analizirani srednji stubovi mosta, izloženi horizontalnom seizmičkom dejstvu, primenjujući spektar odgovora tipa 1, za kategoriju tla B, prema EN1998:2004. U analizu je uključeno i 20% saobraćajnog opterećenja. Stubovi su dimenzionisani prema EN1992, deo 1 i deo 2. Kao rezultat analize, formirane su pushover krive koje opisuju ponašanje elemenata pod dejstvom zemljotresa za podužni i poprečni pravac.

Ključne reči: Stubovi mosta, seizmički uticaji, Metoda ekvivalentne linearizacije, EN 1998:2004, Pushover krive