COMPDYN 2013
4th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, V. Papadopoulos, V. Plevris (eds.)
Kos Island, Greece, 12–14 June 2013

SPECTRUM RESPONSE AND CODES EVOLUTION CASE STUDY – POST-COMPRESSED GIRDER BRIDGE

Carmen BUCUR[1], Victor Mircea BUCUR[2]

[1] Technical University of Civil Engineering
Member of IABSE - WG7
Bucharest, Romania
e-mail: vmb51@yahoo.com

[2] External Collaborator -Technical University of Civil Engineering
Bucharest, Romania
e-mail: vmb51@yahoo.com

Keywords: Earthquake Engineering, Technical Regulations, Spectral response, Evolution, Reinforced concrete

Abstract. *The object of this study is a bridge structure with sectioned, prefabricated, post-compressed, reinforced concrete girders, to be used as a foot-walk. The bridge has two 32.25 m spans. The utilised 33.0 m girders are the longest in the Romanian Prefab Girders Catalogue. Later on, the bridge function was extended, it being used for the road vehicles traffic as well. The bridge, still in use, was checked two times, according to different technical codes. The article is built up on three plans, namely: (i) a brief presentation of Vrancea seismic epicentre, Romania, (ii) a brief presentation of the evolution of the Romanian technical codes for designing the building - and bridge type structures under the seismic action, (iii) application of these codes on this existing bridge type structure and comparing its spectrum response under the seismic action, modelled according to various codes. The analysis of the bridge was carried out taking into consideration the hypothesis of two ductility classes: limited ductility and ductile.*

.

1 INTRODUCTION

During 1988 – 1989 a foot-walk was built to cross-over a hydro-power station off-take. The bridge having two 32.25 m-long spans has sectioned, prefabricated, and post-compressed, simply supported reinforced concrete girders. The utilised 33.0 m long girders are the longest in the Romanian Prefab Girders Catalogue.

In 1995 – 1996 they thought of opening this bridge for the road vehicles traffic as well. Consequently, a general bridge overhaul was carried out, also implying a spectrum analysis according to a new technical code ("Norm for seismic design of the road and railway bridges and of the road over-or under-crossing passages made out of reinforced and pre-stressed concrete") being under technical debate at that time.

Presently, the bridge is to be rehabilitated and shall be checked for the earthquake resistance, being modelled complying with the Romanian codes in force (i) *Design of structures for earthquake resistance. Part 2: Bridges* – Romanian Standards Association, SR EN 1998-2:2006; (ii) *Design of structures for earthquake resistance. Part 2: Bridges. National Annexe* - Romanian Standards Association, SR EN 1998-2:2006/NA:2010; (iii) *Seismic Design Code, Part 1 - P100-1/2006, Earthquake Resistant Design of buildings* - Ministry of Transports, Construction and Tourism, 2007.

From seismic point of view, the area where the bridge is built on is affected by the earthquakes in Vrancea Region, but with reduced exposure (figures 3 and 4). No earth accelerations were registered, and, consequently no “time history” study was drawn up.

The article is structured on three plans, namely: (i) a brief presentation of Vrancea seismic epicenter, Romania, (ii) a brief presentation of the evolution of the Romanian technical codes for designing the building and bridge type structures under the seismic action, and (iii) the application of these regulations on this existing bridge type structure and comparing its spectrum response under the seismic action, modelled according to various codes. The analysis of the bridge was carried out taking into consideration the hypothesis of two ductility classes: limited ductility and ductile.

2 BRIEF PRESENTATION OF VRANCEA SEISMIC EPICENTRE

Romania is highly vulnerable to earthquakes and flood. It is also one of the most seismically active countries in Europe [7]

According to the number of people lost in earthquakes during the XXth century (1574 people as well as those lost in the earthquake on March 4, 1977, 1424 in Bucharest), Romania can be ranked as the 3rd country in Europe, after Italy and Turkey. Romania is followed by the former Yugoslavia and by Greece.

The Vrancea region, located where the Carpathians Mountains Arch bends, at about 135 ± 35 km epicentre distance from Bucharest, is a source of sub-crustal seismic activity, affecting the Romania’s territory and an important part of the territories of Republic of Moldova, Bulgaria and Ukraine. The seismic areas cover about 65% of the territory of Romania, fig. 1, [4].

Vrancea region, which is Romania’s main seismogenic zone, is characterized by the following main remarkable features [6]: (i) Its shallow seismic activity is located mainly in the lower crust (h>15km), and consists of small and moderate magnitude earthquakes; (ii) The sub-crustal seismicity represents the main feature of Vrancea region; (iii) The recurrence times estimated from the available catalogues are the following: 10 years for Mw > 6.5, 25 years for Mw > 7.0 and 50 years for Mw > 7.4; (iv) The large instrumentally recorded earthquakes show a remarkably similar fault plane solution. They typically strike SW-NE (220°), dip 60° to 70° to the NW, and the slip angle is roughly 80° to 90°.

Figure 1: Vrancea sub-custral (60-180 km) seismic source in the Carpathian Mountains of Romania [4]

3 BRIEF PRESENTATION AND EVOLUTION OF THE ROMANIAN EARTHQUAKE - CODES

- Seismic Design Code, Part 1 - P100-1/2006-Earthquake Resistant Design of buildings,
- SR EN 1998-2:2006-Design of structures for earthquake resistance. Part 2: Bridges
- SR EN 1998-2:2006/NA: 2010- Design of structures for earthquake resistance. Part 2: Bridges. National Annexe

3.1 Historical evolution

In 1941 the first national seismic code (after the 1940 earthquake) named "Draft Instructions" took effect. It's followed in 1945 by "Instructions for preventing the damage of buildings due to earthquakes". Between 1963 – 1977 there are two successive versions of the code with the indicative P13.

In figure 2 it is presented the evolution of the values of the normalized acceleration spectrum (β) for T_C=1.5 sec, [3].

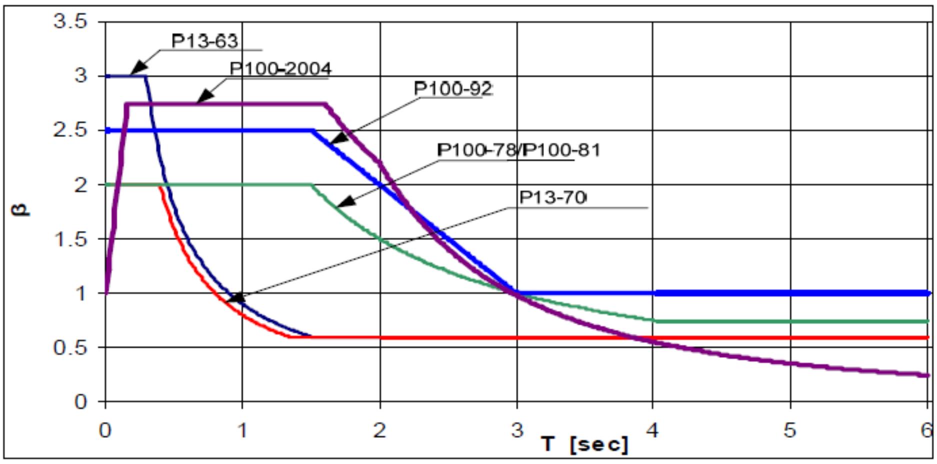


Figure 2: Evolution of the normalized acceleration spectrum values (β) for T_C=1.5 sec [3]

From 1977, after the great earthquake, a series of versions for the code with the indicative

P 100 took effect. They were permanently tweaked by the last researches. The actual version of the code "P100-1:2006 Code for earthquake resistance of buildings and structure" is in accord with EUROCODE 8.

The first code for bridges took effect in 1978. In Table 1 there is a synthesis of the evolution of the earthquake resistance codes for bridges in Romania.

STAS 10101 (1978)	Actions on structures – Classification and combination of Actions for Railway Bridges and Viaducts - *National Committee for Science and Technology – National Institute for Standardization*
PD 197 (1980)	„Design Provisions for Earthquake Resistance of Transportation and Telecommunications Constructions" - *National Committee for Science and Technology – National Institute for Standardization*
DRAFT (1997) Inspired by EUROCODE 8	„Design Provisions for Earthquake Resistance of Road, Railway Bridges and Overpasses Made of Reinforced and Prestressed Concrete" - *Ministry of Public Works and Territorial Administration*
Actual - EUROCODE 8	1. EUROCODE 8 – Design of structures for earthquake resistance. Part 2: Bridges 2. National Annex – may 2010 *Ministry of Transports, Construction and Tourism*

Table 1: The evolution of the codes for earthquake resistance of bridges in Romania

3.2 The presentation of actual norms

In Romania the value of P_{NCR} (probability – within the next 50 years - to exceed the reference seismic action for no-collapse requirement - life safety) is 39% in 50 years, and T_{NCR} (reference return period of the reference seismic action for no-collapse requirement - life safety) is 100 years. The value of P_{DLR} (in 10 years, the probability of exceeding the seismic action for damage limitation requirement) is 28% in 10 years, and T_{DLR} (reference return period of the seismic action for damage limitation requirement) is of 30 years. The ground classification scheme (A, B, C, D, E, S1 and S2) is not applicable in present. For design, the local site conditions are classified in 3 zones/sites, Z_1, Z_2, Z_3, based on accelerograms recorded during 1977, 1986 and 1990 Vrancea earthquakes, zones characterized by the values of the control period of the response spectra, T_C, fig. 3, [10].

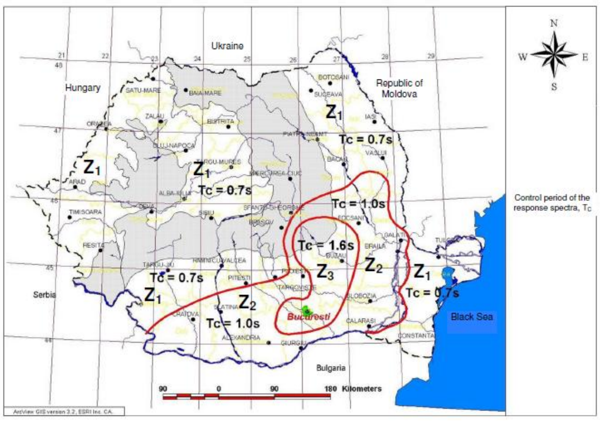


Figure 3: Zone/site in the Romanian territory in terms of control period T_C, [10]

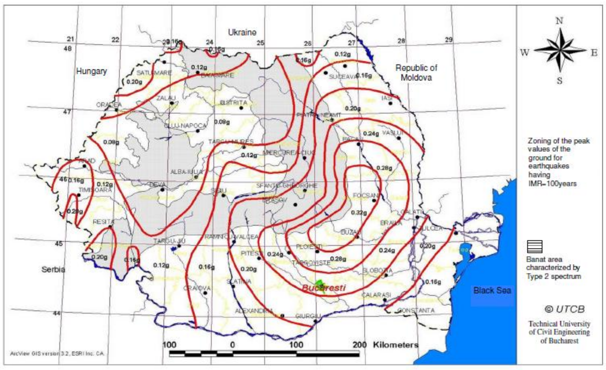


Figure 4: Zoning map of the Romanian territory of the peak values of the ground, [10]

The zoning design ground acceleration a_{gR} in Romania, for seismic events having the mean recurrence interval for the no-collapse requirement – life safety (of the magnitude) T_{NCR} = 100 years, is indicated in the figure 4 and is used for the design of buildings in the ultimate limit state for all zones/sites (country territory), [10].

The procedures for low seismicity are not applicable in Romania.

On Romanian territory three importance classes for bridges are defined, table 2, [9].

Importance classes	Description
III	Road and railway bridges of critical importance for maintaining communications, especially in the immediate post-earthquake period (the bridges supporting the emergency traffic after a major seismic event).
II	Road and railway bridges located on lines of communications of average importance.
I	All the bridges, except those of III and II class as well as road or railway bridges located on small lines of communications. For example, temporary bridges sited on industrial lines of communication - but only with beneficiary agreement and the bridges do not serve important economical or social objectives where they have to maintain communications after a major seismically event.

Table 2: The importance classes for bridges on Romanian territory, [9]

The regulations provide that both the ductile structures and the limited ductile ones shall be designed based on the linear analysis using a modified spectrum response, called designed spectrum.

The limit value of the displacement along the bridge of the abutment rigidly connected to the deck depends on the bridge importance class, namely for Class III - d_{lim}=60 mm; and for Class I – there is no limitation.

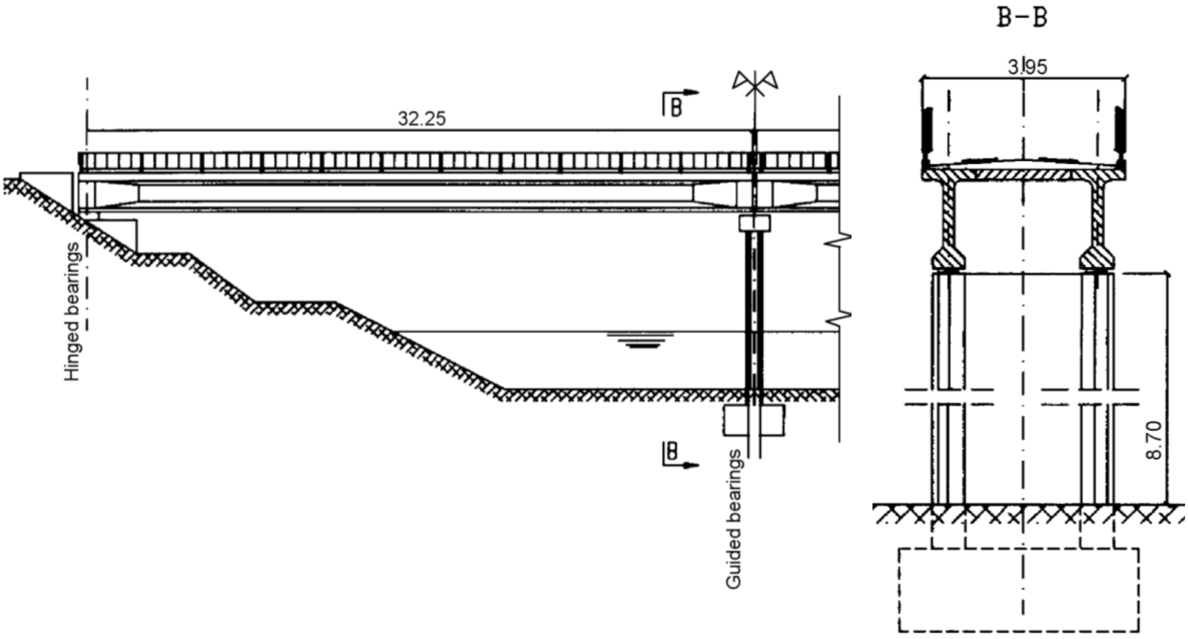
In case of ultimate limit state, after appearing the design seismic action, the bridge shall preserve its structural and residual integrity, although damages may appear in certain areas of the bridge. So, it is allow some plastic areas to appear in the piers due to their bending. These plastic areas (hinges) act as energy dissipation primary elements. The bridge deck shall remain in the elastic domain.

4 THE ANALYSIS OF THE BRIDGE

4.1 Structure description

The bridge was built in 1988 – 1989 to be a foot-walk having two 32.25 m long spans, simply supported girders, fig. 5.a, [2].

The abutments are massive. The reinforced concrete pier has a lamellar elevation and a foundation is made out of a marl-embedded concrete block, fig.5.b. The deck is made up of two sectioned, prefab, reinforced concrete girders, post-compressed on site, being 33.0 m long and 1.80 m high. This prefab 33.0 m girder is the longest in the Romanian Catalogue for road bridges girders. The girders have 2.70 m inter-axe distance, with a 1.95 m wide and 0.18 m thick monolith plate (equal with the thickness of the girders upper plate). The link between the two girders is ensured by three cross-bars (two marginal and a central one). The girders bearings are of hinged-type out of neoprene on the abutments and simple ones on the pier.



a. Elevation b: Cross section

Figer 5: The structure

The bridge belongs to the Importance Class II.

4.2 Design model

This structure is space modelled with finite elements. The abutments and the pier foundation were eliminated out of the structure. The axes system was chosen in the following way**:** the Ox axis is along the bridge, the Oy axis is transversal on the bridge, the Oz axis is vertical; the xOy plan is at the level of the supports of the superstructure girders.
The model, fig. 6, is made up in the following way:

Superstructure:

- In one of the spans, the concrete girders were modelled using one-dimensional finite elements with equivalent geometrical characteristics. The cross-girders were also modelled with one-dimensional elements. The surface-concreting plate was modelled with bi-dimensional elements. This span will be called FRAME in the article.
- In the second span, all the elements making up the superstructure were modelled using bi-dimensional elements having combined the behaviours of the membrane and the plate. This span will be called SHELL.

Pier:

- The finite elements used for the pier elevation are the three-dimensional ones. The pier is considered to be fixed into the foundation.

Girders bearings:

- On the abutments, the hinge type neoprene bearings were modelled using three simple translation-type connections on the three axes of the coordinates, as follows: vertical and transversal on the bridge with rigid ties; longitudinally with a constant elastic spring, equivalent to the used neoprene bearing.
- On the pier, the simple type neoprene bearings were modelled as follows: vertically and transversally on the bridge identical displacement conditions were imposed between the girders and the pier; longitudinally one-dimensional finite elements (double-hinged bar) of known

length were introduced, having geometric and elastic characteristics equivalent to the used bearing type, linking the girders to the pier.
- The rotations are free.

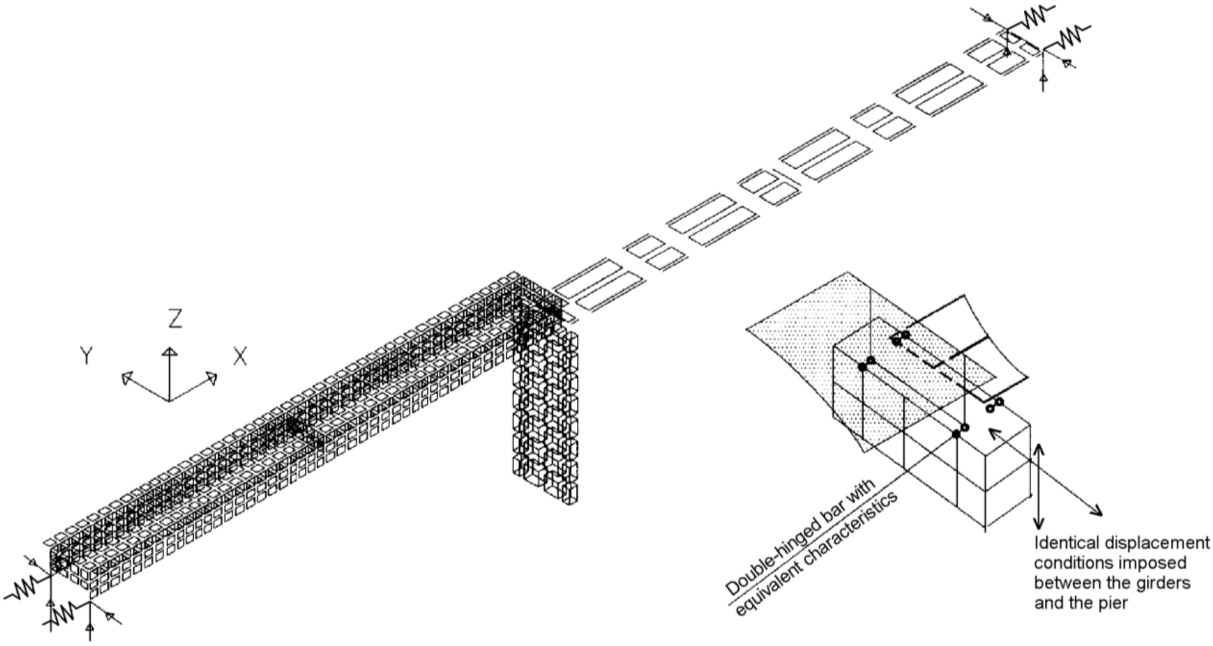


Figure 6: Design model

For design, they took into consideration the masses of the resistance structure for every sub-assembly, as well as the masses of non-structural elements and the masses of the live load, concentrated on the girders. The masses of the live load are considered to be 20%. The live load taken into consideration was the maximum allowed one, as found out when transforming the foot walk into a bridge.

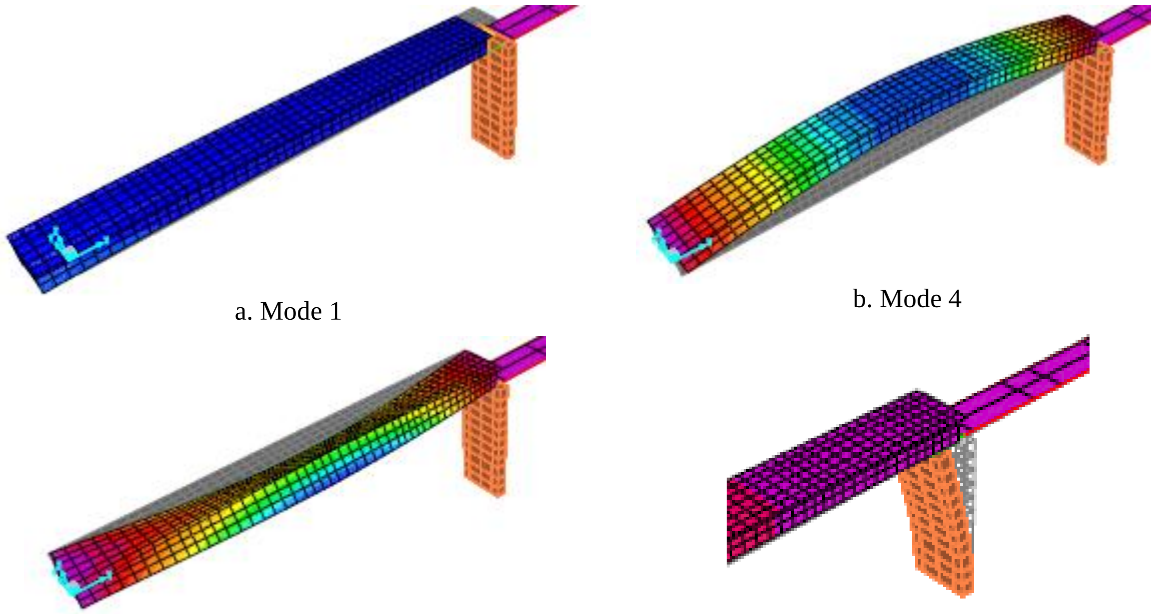
4.3 Proper dynamic response

Results

21 proper vibration modes were considered whose summed up effective modal mass represents about 97% in the longitudinal direction, 90% in the transversal direction and 87% on the vertical one.

- *Modes 1 and 2* (T1=0.558s, T2=0.548s) are vibrations along the bridge, first those of the discrete deck with bi-dimensional elements (SHELL), fig. 7a, and then the discrete one with one-dimensional elements (FRAME). The value of the fundamental period shows that the structure places itself at the upper part of the rigid behaviour. The form of the proper vectors of the modes 1 and 2 presents a rigid body movement (translation) of the two decks along the bridge.
- *Modes 3 and 4* (T3=0.348 and T4=0.323s) are vertical vibrations, bending with a single half-wave, first of the FRAME deck and then of the SHELL deck. The pier has also a slight vibration along the bridge, as if it would be mobilized by the vibrating deck, fig. 7b.
- *Mode* 5 (T5=0.317s) is the torsion vibration mode of SHELL deck, fig. 7c.
- *Mode 6* (T6=0.267) is the bending vibration mode of the pier compared to the bridge transversal axis, fig. 7d.
- *Mode 7* (T7=0.247s) is the FRAME deck torsion in rated by the bridge longitudinal axis – corresponding to mode 5 of SHELL deck.

- In *mode 8* (T8=0.208s) the two decks have different vibrations. The SHELL deck has torsion while the FRAME deck has a lateral vibration. The pier has also a lateral vibration.



a. Mode 1

b. Mode 4

c. Mode 5

d. Mode 6

Figure 7: Particular vibration modes

Comments

1. The value of the fundamental periods places the structure at the limit between the rigid and half-rigid behaviour.

2. The minor differences of the vibrations periods (longitudinal – 1.8% and bending – 7.7%), obtained for modelled SHELL and FRAME decks show that the modelling based on equivalence with FRAME elements is acceptable for these types of vibrations.

3. The first pier vibration mode intercalates between the torsion type vibrations of the two decks.

4. The high difference between the torsion vibration periods – 28.3%, of the two decks, shows that for this type of vibration the FRAME modelling is not advisable. The vertical development of the SHELL modelled deck (which is lost at FRAME deck) allows it to simulate a more complete dynamic response.

5. For the span modelled with elements bi-dimensional (SHELL) the torsion accompanies more or less the majority of the vibrations.

4.4 Spectrum response. Comparisons

The spectrum analysis was carried out according to the recommendations in several technical codes grouped as follows:

(1) (i) Norm for the design of the earthquake-resistant road and railway bridges and of the road under-and over-crossing passages out of reinforced and pre-stressed concrete – under technical debate in 1997;
(ii) R 11100/1-1993 - Macro-seismic zoning of Romania's territory;

(2) (i) Design of structures for earthquake resistance. Part 2: Bridges;
(ii) Design of structures for earthquake resistance. Part 2: Bridges. National Annex;
(iii) Seismic Design Code, Part 1 - P100-1/2006, Earthquake Resistant Design of buildings.

In the article, the study situation according to regulations (1) is named **Code 1997** and that for the study situation according to regulations (2) is named **Code 2006.**

The bridge location in the context of the Romania's seismic map leads, for the case study **Code 1997** to ratio $a_g/g=0.1$ and $T_c \leq 0.8s$ and for the case study **Code 2006** at $a_g=0.12g$ and $T_C=0.7$ respectively. For **Code 2006,** beside the situation of the seismic action modelled under the form of the elastic spectrum - called **Code 2006-Se**, there are also considered two other situations of the design spectrum referring to the structural ductility, namely a limited ductile structure ($q_{duct.limit}=1.5$) - called **Code 2006-Sd(q=1.5)** and ductile structure ($q_{duct.}=3.5$) - called **Code 2006-Sd(q=3.5)**.

In figure 8 acceleration spectra for all study situations are presented graphically.

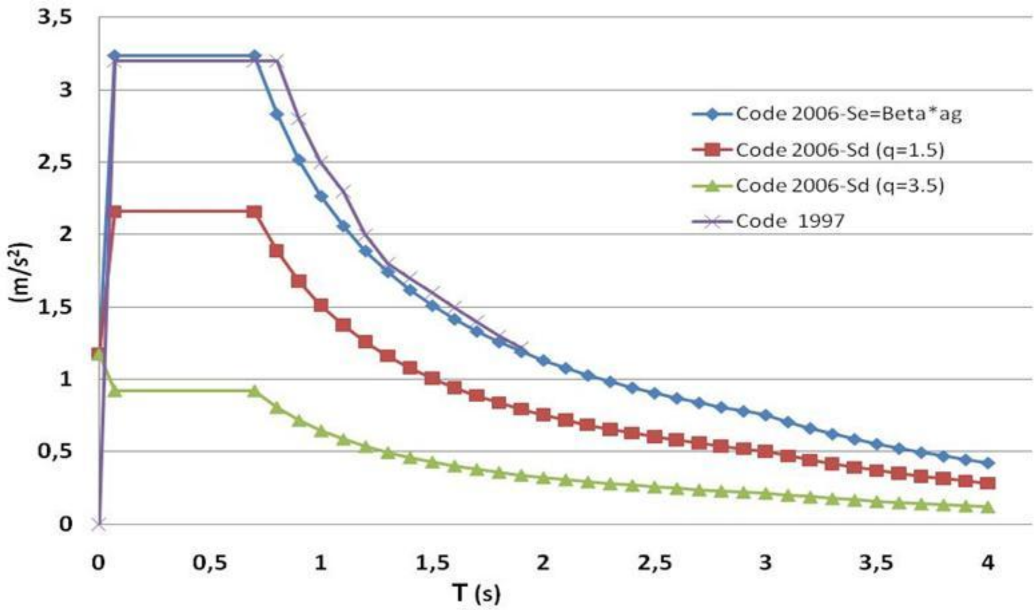


Figure 8: Acceleration spectra for all study situations

Studying the spectrum values one can find out that in the case **Code 1997** starts from the zero value but for the horizontal zone the coordinates are practically equal with those for the situation **Code 2006-Se**. For all cases **Code 2006 (Se, Sd)** the first coordinate has the same value. The other spectrum coordinates differ one from the other in inverse ratio to the considered ductility coefficient – 1.5 and 3.5 respectively.

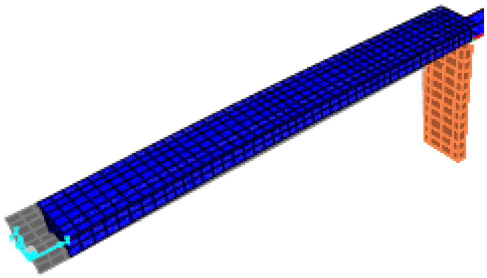
The cases of spectrum loading – twelve in number – are as follows: **Code 1997** with three directions of spectrum action, x-longitudinal, y-transversal and z-vertical; **Code 2006-Se, Code 2006-Sd(q1.5), Code 2006-Sd(q=3.5)** each with three directions.

The horizontal spectrum action is described by two orthogonal components considered independent between them and represented by the same response spectrum. For the vertical component the same response spectrum may be considered, but with reduced coordinates depending on the magnitude of the bridge vibration periods. The fraction from the critical amortization was considered 5%. The total maximum effects are defined based on the maximum values produced by the spectral action applied separately on the three directions according to rule SRSS.

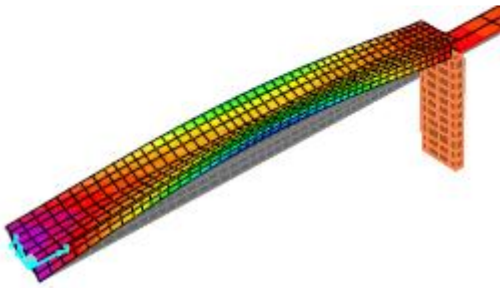
Spectrum displacements. Results, comments

Spectrum displacements follow the bridge „sensible” vibration forms. The maximum value is that in the bridge longitudinal direction, followed by the transversal and the vertical ones. The examples are given in figure 9. The figure 10 presents a synthesis of the spectrum displacements maximum values in the 12 loading situations.

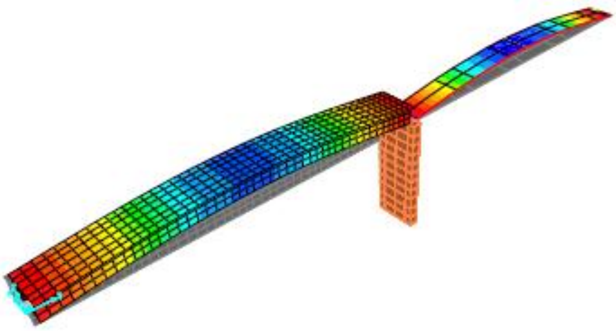
The obtained displacements for the spectrum loading **Code 1997** and **Code 2006-Se** (elastic spectrum) are practically equal, namely 0.0301 m and 0.0305 m respectively. Decrease of the displacements values for the cases considering the limited ductile structure and ductile as compared the cases considering the elastic structure - 1.5 and 3.46 respectively - is practically proportional with the ductility coefficient.



Spectrum displacements for loading
Code 1997 –longitudinal direction



Spectrum displacements for loading
Code 2006-Se – transversal direction



Spectrum displacements for loading
Code 2006-Sd (q=1.5) –vertical direction

Figure 9: Spectrum displacements

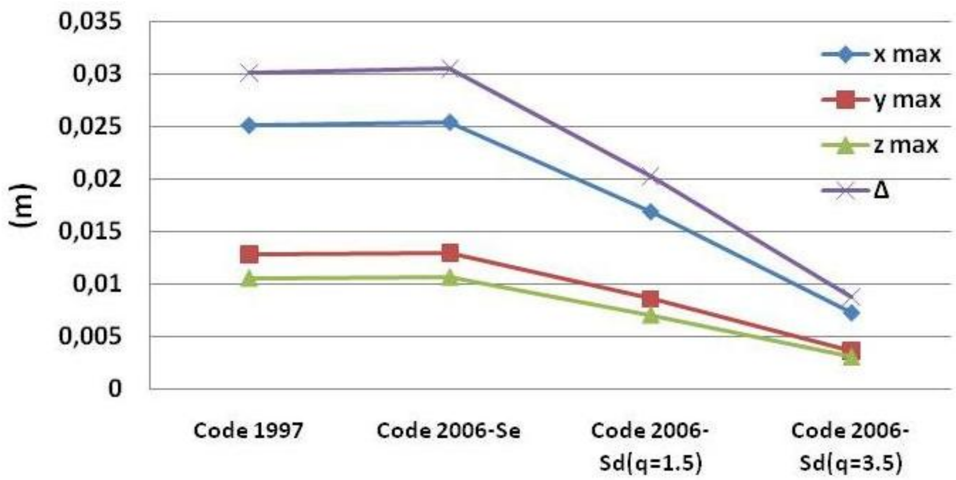


Figure 10: Systematization of spectrum displacements values

It is found out that the influence of the first spectrum coordinate (that does not comply with the right proportion rule) is much reduced. It happens because in the acceleration spectrum the first zone – between 0 and T_B=0.07s – is narrow and then the influence of this zone is reduced.

The displacements along the bridge for the spectrum loading **Code 1997** and **Code 2006-Se** are 0.0251 m and 0.0254 m respectively. These values represent about 42% from the limit displacement at the ultimate limit state for class II where the bridge belongs from (d_{lim}=60 mm)

The displacements values on the bridge transversal direction resulted from the spectrum loading in the same direction, (transversal), represents about 50% from the values of the longitudinal displacements for the longitudinal spectrum loading.

The displacements values on the bridge vertical direction resulted from the spectrum loading in the same direction, (vertical), represents about 42% from the values of the longitudinal displacements for the longitudinal spectrum loading.

<u>Spectrum efforts. Results, comments</u>

Studying the obtained values one can find out the same variation ratio as for the spectrum displacements, namely:

(1) Values of the spectrum efforts obtained for the loading situation in **Code 1997** are practically equal with those in the loading situation in **Code 2006-Se** – Elastic spectrum.

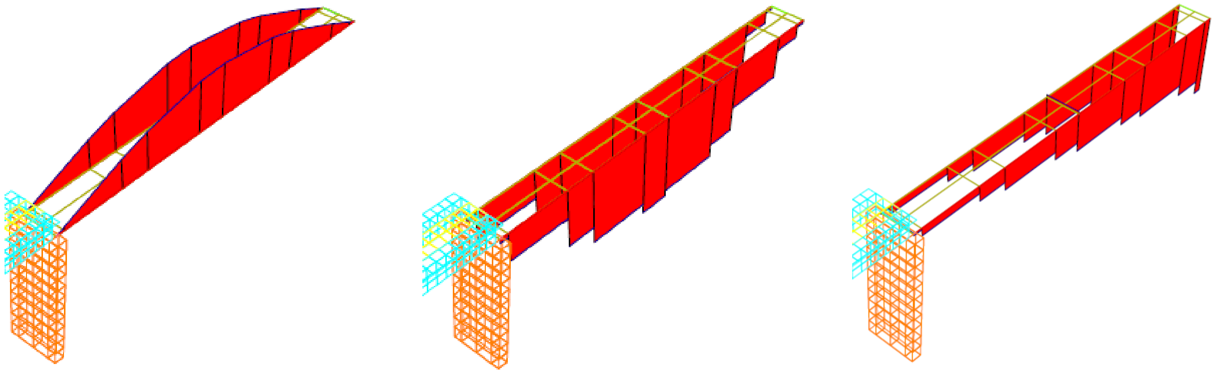
(2) Values of the spectrum efforts obtained for the loading situation with the design spectrum are practically divided by the ductility coefficients that are 1.5 and 3.5, with some exceptions – stresses in the bi-dimensional type elements – where the values have decreased with 1.6 and 3.7 respectively.

Consequently, bellow, we'll comment the values obtained for the loading situation in **Code 2006-Se** – Elastic spectrum.

* *In the deck modelled with one-dimensional elements*, fig. 11:
- the maximum dynamic moment develops in the field at the spectrum action on the vertical direction (M=1109 kNm), fig. 11.a;
- the maximum dynamic axial force develops in the field from the transversal spectrum action (N=725 kN), fig. 11.b;
- at the longitudinal seismic action the area where the deck rests on the abutment is stressed where the axial force represents about 30% from the maximum one, fig.11.c.

* *In the deck modelled with bi-dimensional elements*, fig. 12:
- force in the plan of bi-dimensional elements, on the bridge longitudinal direction has the maximum value (1526 kN(/m)) of the seismic action on vertical direction; position on the deck is in the center and on the flange, fig. 12.a;
- from the seismic action on the bridge transversal direction one can obtain: (i) the maximum value for the force on the transversal direction (607 kN(/m)) located in the area where it rests on the pier, (ii) for the moment after the longitudinal axis, fig. 12.b, the maximum value is in the area where it rests on the abutment (157 kNm(/m)), (iii) for the moment on the direction of the transversal axis, the maximum value (87 kNm(/m)) is also in the area where it rests on the abutment;
- values of the moments resulted from the seismic action on longitudinal and vertical directions are not significant.

* *In the pier*, fig. 13:
- from the dynamic action on the longitudinal direction, the maximum values are obtained for the stresses on longitudinal (758 kN/m^2) and vertical (2951 kN/m^2) directions, fig. 13.a;

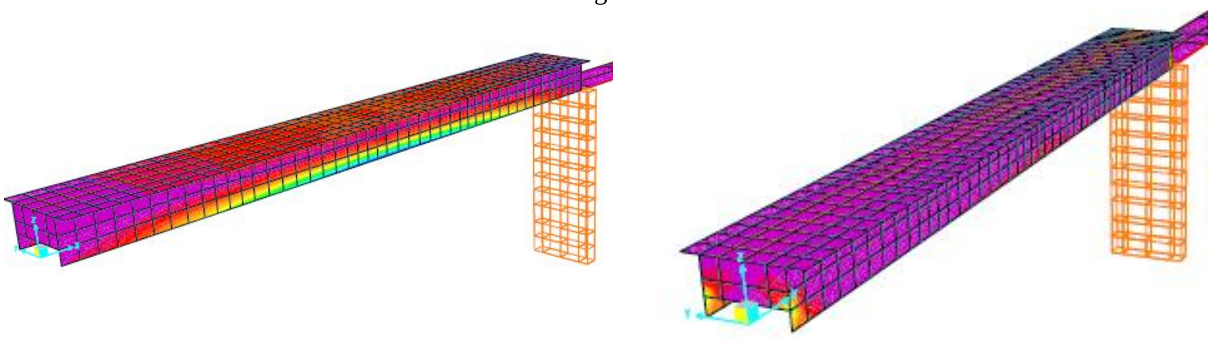


a: Maximum moment in the seismic action on the vertical direction

.b: Maximum axial force in the seismic action on the transversal direction

c: Maximum axial force in the seismic action on the longitudinal direction

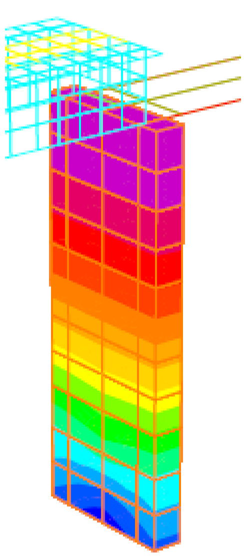
Figure 11



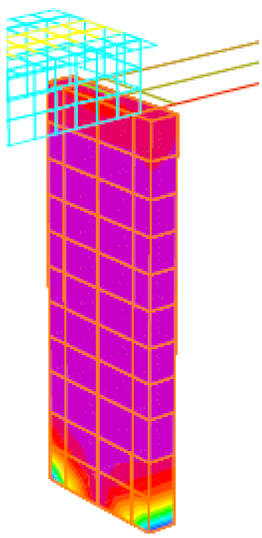
a: Force on the bridge longitudinal direction from the seismic action on the vertical direction

b: Moment on the bridge longitudinal axis resulted from the seismic action on the transversal direction.

Figure 12



a: Stresses on the vertical direction resulted from the seismic action on the longitudinal direction



b: Stresses on the transversal direction from the dynamic action on the transversal direction.

Figure 13

- the maximum value of the stress on transversal direction is obtained from the seismic action on the same direction (2660 kN/m^2), fig. 13.b;
- the lowest stress on the three directions results out of the seismic action on the vertical direction.

5. FINALE COMMENTS

1. Modelling the two decks (SHELL and FRAME) with different finite elements leads to the following observations: (i) it is find out a decoupling of the proper dynamic response of the two decks, they entering the vibration by turns – SHELL span first for the longitudinal and torsion type vibrations, FRAME span first for the vertical and transversal type vibrations; (ii) the periods differences vary from 2% for the longitudinal vibration and 28% for the torsion vibration; (iii) in case of the bi-dimensional span modelled with SHELL elements torsion accompanies more or less the majority of the vibrations.
In conclusion, to obtain a dynamic response as complete as possible the deck design model shall preserve its development along is three directions.

2. From the spectrum loading **Code 1997** and **Code 2006-Se** one can obtain practically the same dynamic response. The values of the dynamic response for the design spectra, **Code 2006-Sd(q1.5)** and **Code 2006-Sd(q=3.5)** are lower proportionally with the ductility coefficients - 1.5 and 3.5 respectively. This is due to the fact that in the acceleration spectrum, the first zone – between 0 and T_B=0.07s where, generally, the coordinates do not comply with the proportional rule with the ductility coefficients – is narrow and its influence reduced.

3. From the spectrum action on the bridge longitudinal direction one can find out that: (i) the dynamic displacement has the maximum value; (ii) the decks have the displacement of a rigid body so that the dynamic efforts appear only in the supporting areas; (iii) the stress on the pier is maximum.

4. In case of the spectrum action on the transversal direction one can find out the following: (i) the displacements on the transversal direction are 50% from the longitudinal displacement for the case of the spectrum action on the longitudinal direction; (ii) the maximum dynamic efforts are in the center of the decks.

5. From the spectrum action on the vertical direction one can find out that: (i) the displacements on the represent about 42% from the longitudinal displacement for the case of the spectrum action in the longitudinal direction; (ii) the maximum dynamic efforts are in the center of the deck.

REFERENCES

[1] Avram C., Bob C., Friedrich R., Stoian V. - *Reinforced Concrete Structures. Finite Element Method. Theory of Equivalences*, Ed. Academiei Republicii Socialiste România, 1984

[2] C. Bucur, A. Rus, A. Ghiţă - *Răspuns dinamic - studiu de caz. Pasarela peste canalul de fugă de la Săcădat* -A1 X-lea Congres Naţional de drumuri şi poduri, Iaşi 1998 Referate şi comunicări Vol. III pag.314-321

[3] Creţu D., Demetriu S. - *Metode pentru calculul răspunsului seismic în codurile româneşti de proiectare. Comparaţii şi comentarii* - Revista AICPS, Nr. 3/2006

[4] Lungu D., Văcăreanu R., Arion C., Aldea A. - *Building inventory data for seismic risk mitigation* – NERO European Building Inventory Workshop, Pavia 2011

[5] Negoescu E., Fierbinþeanu V. - *Proiectarea şi protecţia antiseismică a podurilor şi construcţiilor pentru transporturi*, Editura Tehnicã, 1985

[6] Moldoveanu C.L., Radulian M., Mărmureanu Gh., Panza G.F. – *Outlines of seismic Microzoning of Bucharest, Romania* - United Nations Educational Scientific and Cultural Organization and International Atomic Energy Agency, The ABDUS SALAM International Centre for Theoretical Physics, MIRAMARE – TRIESTE March 2002

[7] *** United Nations, International Strategy for Disaster Reduction (The World Bank) – *South eastern Europe Disaster Risk mitigation and adaptation Initiative* - Geneva 2002

[8] *** SR EN 1998-1/NA November 2008 - Eurocode 8: Design of structures for earthquake resistance – *Part 1: General rules, seismic actions and rules for building, National Annex*

[9] *** SR EN 1998 2/NA May 2010 - Eurocode 8: Design of structures for earthquake resistance - *Part 2: Bridges, National Annex*

[10] *** Seismic Design Code, *Part 1 - P100-1/2006, Earthquake Resistant Design of buildings*

[11] *** SR EN 1998-2/ 2006, *Eurocode 8: design of structure for earthquake resistance. Part 2: bridges*

[12] *** Technical University of Civil Engineering, Bucharest, Building Research Institute, International Institute for Seismology and Earthquake Engineering (Japan) – *Study on early Earthquake Damage evaluation of existing Buildings in Bucharest, Romania* – march 2006

[13] *** *Norm for the anti-seismic designing of the road, railway bridges as well as of the under path of reinforced and prestressed concrete*, 1997 (draft - in Romanian)