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THE VARIATION OF INTERNAL FORCES OF THE BRIDGE SUPERSTRUCTURE UNDER INFLUENCE OF THE VERTICAL COMPONENT OF THE SEISMIC ACTION

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Abstract: *In this paper the authors have investigated the variation of internal forces magnitude of the pre-stressed box-girder bridge superstructure under influence of the vertical component of the seismic action. This influence is particularly important for the structures located near earthquake epicenters, where they have to sustain high vertical seismic loads due to intensive vertical vibrations. The calculation is based on the Eurocode standard, according to the clause that the effects of the vertical seismic component acting in the upward direction on pre-stressed concrete decks should be always taken into account. This static and dynamic analysis is performed using the computer software package Robot Millennium on 3D model of the three span box-girder bridge (41+55+41m). Each of the three bridge spans consists of eight bar elements, while the columns are formed with three bar elements. The bridge superstructure is based on the abutments and columns on bearings, two for each support. A response spectrum analysis is conducted according to the EN 1998-2 standard. Design ground spectrum for the elastic analysis has been designed according to the above regulations and for the parameters of the soil category B and design ground acceleration of 0,2g. Extreme values of the forces are determined using the SRSS rule. A dynamic analysis is conducted for the 10 modal shapes. The impact of seismic activity on the bridge is analyzed using the CQC method. Both the seismic load and traffic load are applied aiming to determine the relationships between internal forces magnitude with and without the influence of the seismic action vertical component.*

1. INTRODUCTION

Earthquakes that occurred in the past 30 years attracted much of attention on the effects of vertical ground motion i.e. vertical acceleration. Table 1 shows only a few of near-field catastrophic earthquakes which had significantly higher vertical acceleration than its horizontal counterparts. These differences are especially notable in the response spectra recorded near the causative fault. Furthermore, the vertical component of motion may be more severe than horizontal component at short periods, especially for most structural systems that have short vertical natural periods. Near-source earthquake recordings show that V/H ratio of PGA (peak ground acceleration) could exceed value of 1. Besides, the V/H ratio value of 1 is exceeded in the high-frequency range of the response spectra. This fact gives cause for concern since common engineering practice considerably underestimates the spectral ratio, especially in the near-field region [1] (usually, V/H ratio is assumed to be two-thirds or even less).

Year	Earthquake	M	Station	PGA (g)			V/H	Distance* (km)	Damage & Casualties
				H0	H90	Up			
1979	Imperial Valley-06	6,53	El Centro Array #6	0,41	0,44	1,66	3,77	1,35	$30 million in damage, 91 injured
		6,53	Agrarias	0,37	0,22	0,83	2,25	0,65	
1984	Morgan Hill	6,19	Hollister Diff Array #3	0,08	0,08	0,24	3,02	26,43	$7 million in damage, 27 injured
1985	Nahanni, Canada	6,76	Site 1	0,98	1,10	2,09	1,90	9,60	No serious damage – sparse population, wood-frame or log structures
1986	N. Palm Springs	6,06	Morongo Valley	0,22	0,20	0,40	1,81	12,07	$4,5 million in damage, 29 injured
1987	Superstition Hills-02	6,54	Wildlife Liquef. Array	0,18	0,21	0,41	1,97	23,85	$3 million in damage
1989	Loma Prieta	6,93	LGPC	0,56	0,61	0,89	1,47	3,88	$7 billion in damage, 63 dead, 3757 injured
		6,93	Capitola	0,53	0,44	0,54	1,02	15,23	
1990	Manjil-Rudbar, Iran	7,37	Abbar	0,51	0,50	0,54	1,05	12,56	$7 million in damage, 50,000 dead, 60,000 injured
1992	Landers	7,28	Lucerne	0,73	0,79	0,82	1,04	2,20	$100 million, 3 dead, 400 injured
1994	Northridge-01	6,69	Rinaldi Receiving Sta	0,84	0,47	0,85	1,02	6,50	$20 billion in damage, 72 dead, 9000 injured
		6,69	Pacoima Dam (upper left)	1,58	1,29	1,23	0,78	7,01	
		6,69	Arleta	0,34	0,31	0,55	1,61	8,66	
1995	Kobe, Japan	6,90	Port Island (0m)	0,31	0,28	0,56	1,79	3,30	$60 billion in damage, 5000 dead, 26000 injured
1999	Chi-Chi, Taiwan	7,62	TCU 118	0,11	0,09	0,12	1,03	26,8	$4 billion in damage, 2400 dead, 8000 injured
		7,62	CHY092	0,08	0,11	0,12	1,07	22,7	
1999	Chi-Chi, Taiwan-06	6,30	TCU 075	0,06	0,11	0,17	1,56	26,31	
		6,30	TCU 076	0,11	0,12	0,26	2,07	25,85	

*closest distance to the fault

Table 1 - Near-field earthquakes with high vertical PGA

1.1 Why is the vertical component not considered?

According to Table 1, earthquakes that had higher vertical acceleration than its horizontal counterpart caused greater damage to structures and resulted in more victims. In spite of these recently recorded destructive earthquakes, vertical component is still failed to be taken into account. It is obvious that an earthquake is a natural phenomenon that generates spatial seismic waves. According to the previous statement, an earthquake creates complex movements in three-translational and three-rotational directions that vary from one location to another. In modern seismology, it is a common practice to record only translational components in three orthogonal directions, but almost always only horizontal components have been involved in spectral response computations. Vertical component of the ground motion is often neglected partly due to the common practice of considering only far-fault earthquake recordings. These earthquake recordings implicate rotational components of motion that are so small that they do not significantly influence horizontal and vertical seismic loads. Obviously, all structures are primarily designed for a downward force G called gravity load equal to total mass M (comprised of self weight and imposed loads) times gravitational acceleration *g* in the vertical downward direction. Vertical acceleration during ground shaking either adds to or subtracts from gravitational acceleration. Since safety factors are included in the structural design, it is believed that most structures have sufficient over-strength against vertical component and they do not need any additional strengthening. This is another cause because of which engineers tend to neglect vertical component of ground motion in an earthquake-resistant structural design. Furthermore, engineers often neglect the fact that a peek in vertical ground motion is not compatible with a peek in the horizontal ground motion [2, 3]. The mentioned misconception arise from the often-quoted engineering "rule of thumb", a principle with a broad application which is not intended to be strictly accurate or reliable in every situation. Analyzing earthquake disasters which occurred in the past few years, experts started to consider the possibility that most of the resulting damage may be attributed to vertical acceleration. This leads to the conclusion that the existing structural codes may not be sufficiently adequate.

2 COMPARISONS OF SOME MAJOR CODES

2.1 UBC (Uniform Building Code) and IBC (International Building Code)

As referred in the UBC section 1620.3.2 and the corresponding IBC section 1630.11, horizontal cantilever and horizontal pre-stressed elements are required to be verified against vertical acceleration i.e. vertical component of earthquake ground motion [4].

2.2 NZS (New Zealand Standards)

According to the NZS sections 5.5.1 and 6.4.1, structures or parts of structures which are sensitive to vertical accelerations, such as the previously mentioned horizontal cantilevers or equipment items have to be analyzed using a vertical component of the earthquake ground motion record simultaneously with their two counterpart horizontal components [5, 6].

2.3 Eurocode 8 (EN 1998-1-1 & EN 1998-2)

According to the EN 1998-1, section 4.3.3.5.2, the vertical component should not be neglected if a_{vg} (vertical ground acceleration) is greater than 0,25g i.e. 2,5 m/s^2. This section of the Eurocode refers to horizontal or nearly horizontal structural members spanning 20 m or more; to horizontal or nearly horizontal cantilever elements longer than 5 m; to horizontal or nearly horizontal pre-stressed elements; to beams supporting columns; and base-isolated structures [7]. According to the EN 1998-2, section 3.2.2.3 (1)P, near source effects have to be used in analysis when the site is located within 10 km horizontally of a known active seismotectonic fault. The section 4.1.7 (2)P of the same code states that the effects of the vertical component acting in the upward direction on pre-stressed concrete decks have to be considered [8].

3 BRIDGE MODEL

One teacher said: "A young engineer will have best understanding of the problem if she or he learns from a solved problem." Students who are taught by the authors of this paper are familiar with the problem regarding vertical component in a structural analysis. A simple example of the bridge shown given below is presented to them as a part of the related course.

3.1 Bar model

While carrying out preliminary studies bar models are widely used in analyzing the behavior of structures, as well as of bridges subjected to earthquakes. A three span (41 + 55 + 41 m) straight bridge in both vertical and horizontal plane is assumed to be located in the seismic zone VIII of Croatia on the B soil [9].

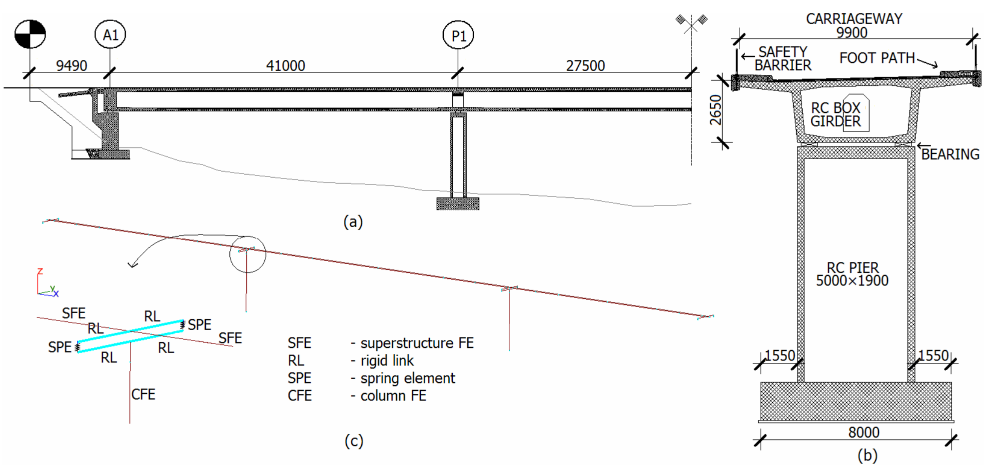

Figure 1 – (a) elevation, (b) cross section of bridge (c) bar model

The superstructure is a box-girder whose every span is divided into eight finite elements. According to Chen and Duan, a minimum suggested division for a superstructure is five finite elements per span [10]. The carriageway consists of two lanes – 7,10 m in clear width. On the left and right side of the carriageway there is a footpath which is 1,50 m wide. Since not possible to model a diaphragm in a bar model, the mass of each diaphragm is lumped at appropriate locations. The bridge superstructure is based on the abutments and columns on bearings, two for each support. The bearings are modeled by using spring elements. They are symmetrically arranged relative to the longitudinal axis of the bridge, positioned 2 m on its left and right side and connected to the superstructure and columns by rigid links. The columns are modeled by frame elements and divided into the three finite elements as suggested by Chen and Duan [10]. Fixed boundary conditions were specified at the base of the columns. The abutments are massive so they needed no modeling. The entire modeling was made using the software package Robot Millennium [11].

3.2 Applied loads

Self weight of the bridge, along with finishing layers of the carriageway, foot paths and the safety barriers make permanent load of the superstructure. Traffic load is represented by the Load Model 1 which is, according to the European standards, defined as live load. The design response spectrum for the horizontal components of the seismic action is defined according to the expressions given in the EN 1998-1-1, section 4.2.4 [12] and by Radić [9]. The design response spectrum for the vertical components of the seismic action is defined using the expressions for the horizontal component and multiplied by reduction factors as follows [9]:

- reduction factor is equal to 0,70 for T < 0,15s
- reduction factor is equal to 0,50 for T > 0,50s
- reduction factor is to be interpolated between values of 0,50-0,70 for 0,15s < T < 0,50s

T(s)	$S_{d,H}(T)$	$S_{d,V}(T)$
0,00	0,200	0,140
0,10	0,289	0,280
0,15	0,333	0,350
0,20	0,333	0,335
0,30	0,333	0,305
0,40	0,333	0,280
0,60	0,333	0,250
0,80	0,275	0,206
1,00	0,237	0,178
1,50	0,181	0,136
2,00	0,149	0,112
2,50	0,128	0,097
3,00	0,114	0,085
4,00	0,071	0,053
5,00	0,049	0,036
6,00	0,036	0,027
8,00	0,022	0,017
10,00	0,015	0,011

(a)

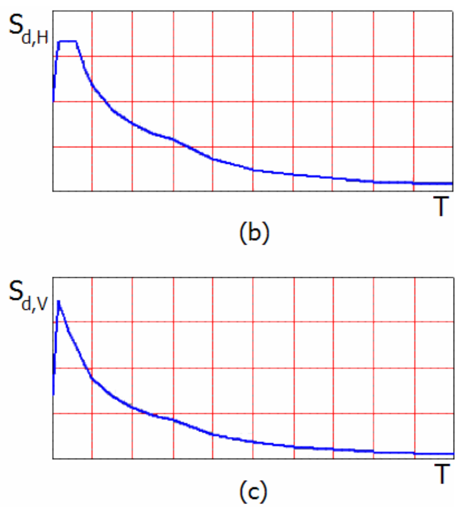

Figure 2 – (a) Response spectrum (RS) values, (b) Horizontal RS and (c) Vertical RS

Extreme values of the forces are calculated using the SRSS rule. The dynamic analysis is conducted for the 10 modal shapes. The effect of seismic activity on the bridge is analyzed by the CQC method.

4 RESULTS AND CONCLUSION

The bridge model was calculated and four load combinations were made for the given loads. The three load combinations included the effects of the seismic action and the forth was the basic load combination. The results are compared for the two combination relations, as described below. First, the internal forces obtained from the load combination with seismic load acting in all three orthogonal directions are compared with their counterparts which were obtained from the load combination with the seismic load acting in only two orthogonal horizontal directions:

$$\boxed{(G\,"+"\,\psi_{21}Q\,"+"\,S_{xyz})/(G\,"+"\,\psi_{21}Q\,"+"\,S_{xy})} \qquad (1)$$

where:

G	is unfactored dead load
$\psi_{21} = 0{,}20$	is combination factor for seismic load combination
Q	is unfactored live load
S_{xyz}	is unfactored seismic load - both horizontal components and vertical component
S_{xy}	is unfactored seismic load - both horizontal components without vertical component

and “+” stands for the symbol of combination.

These ratios are represented in figure 4 with red color columns.

Second, the expression (2) shows the ratio of the internal forces obtained from the load combination with the seismic load acting only in the vertical direction against the internal forces resulting from the basic load combination:

$$\boxed{(G\,"+"\,\psi_{21}Q\,"+"\,S_{z})/(\gamma_G G\,"+"\,\gamma_Q Q)} \qquad (2)$$

where:

S_z	is unfactored seismic load - vertical component
$\gamma_G = 1{,}35$	is safety factor for dead load
$\gamma_Q = 1{,}50$	is safety factor for live load
$\psi_{21} = 0{,}20$	is combination factor for seismic load combination

This unusual comparison is made as a reply to the question: *“Why?” – “Why not? If it is possible to combine basic loads with only two horizontal components of the seismic action, why cannot we combine basic load with only the vertical component of the seismic action?”* These ratios are represented in figure 4 with blue color columns.

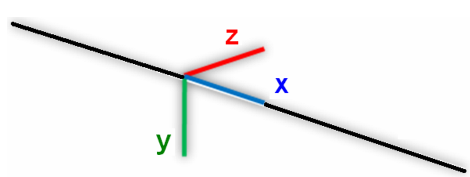

Figure 3 – local coordinate system of superstructure frame elements

In this example, as expected, there is no variation of the magnitude of internal forces, such as axial force (F_x), transversal force in the local z direction (F_z), torsion moment ($M_{T,x\text{-}x}$), and bending moment around the local y axis ($M_{B,y\text{-}y}$), under influence of the vertical component combined with both horizontal components of the seismic action. The magnitude variation of the internal transversal forces in the local y direction (even up to 135%) and the internal bending moment about the local z axis (up to 115%) indicates the need to combine the vertical component of the seismic action with both horizontal components. Especially when designing pre-stressed bridge superstructures since prestressing cables tend to counteract bending moments caused by dead weight of a superstructure, live load, etc. This relatively simple example confirms the importance of including the vertical component of the seismic action in the bridge design, even if it is not a near-fault site. It is expected that these effects are even larger for irregular structures. The combination comprised of basic loads and only the vertical component is not relevant and it can be ignored during the analysis because the internal forces are reduced even to 15% of its reference value.

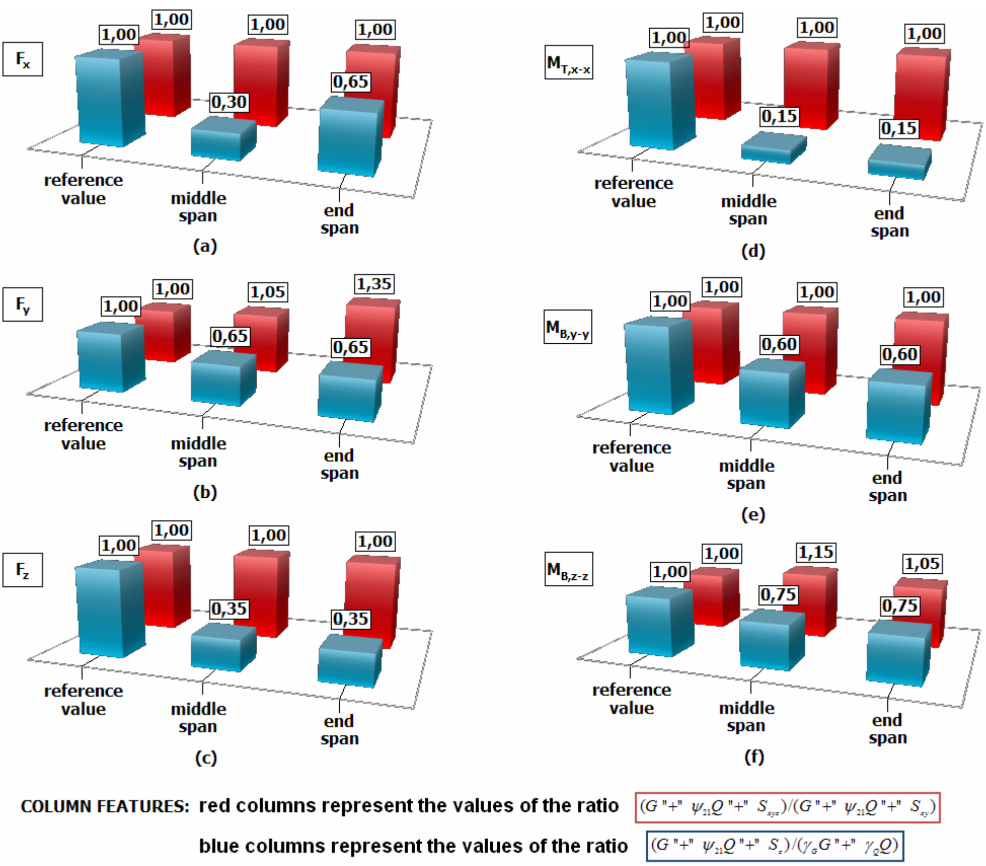

Figure 4 – (a) axial force, (b) transversal force y, (c) transversal force z, (d) moment of torsion, (e) bending moment y-y, (f) bending moment z-z

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