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EFFECTS OF THE SOIL-STRUCTURE-INTERACTION ON THE REGULAR SEISMIC BEHAVIOUR OF BRIDGES

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ABSTRACT: Eighteen types of idealized four span bridges are investigated in this paper in order to determine the effects of soil-structure-interaction (SSI) on the regular dynamic behaviour of bridges. The simplified dynamic analyses in the transverse direction are performed for fixed-based structures and for structures where the soil-structure interaction is taken into account. Three different types of soil, representing soil classes B, C and D, in accordance with EC8-1, are considered. Different criteria for regular dynamic behaviour of bridges are applied and influence of the SSI on the regularity of bridges is discussed.

KEY WORDS: Bridge; Regular behaviour; Soil-structure-interaction; EC8-2.

1 INTRODUCTION

In recent years earthquakes have caused unexpected collapse of the affected bridges. In many cases the dynamic response was influenced by soil conditions. Numerical investigations have shown that the additional base flexibility introduced by the soil-foundation system could play an important role in altering the overall response of the bridge system [1], [2]. Whether the SSI will have beneficial or detrimental effect on the bridge behaviour depends on the characteristics of the structure and ground motion due to earthquake [3].

The present paper is an attempt to clarify the effect of SSI on dynamic behaviour of bridges and the selection of appropriate method for dynamic analysis. In this paper, bridge is considered as regular if in the linear dynamic analysis the fundamental mode dominates in dynamic behaviour of structure and the equivalent static analysis can be applied in accordance with EC8-2 [4]. In a case of bridges with ductile behaviour, ρ [4] is used as the determining factor to esteem a regular behaviour of bridges.

In this paper the influence of SSI on the regularity of eighteen types of idealized four span bridges is analyzed. The regularity of these bridges was first studied by Isakovic, Fischinger and Kante [5]. They investigated the influence of the relevant parameters on the dynamic transverse response of these bridges with fixed-base columns. The responses of single-degree-of-freedom (SDOF) models and multi-degree-of-freedom (MDOF) models for elastic and inelastic analysis were compared and in many cases significantly different results were obtained. Based on the differences in the responses between SDOF and MDOF models they proposed the regularity index, as a new quantitative measurement of bridge regularity.

In our study influence of SSI is taken into account through the use of equivalent springs and dashpots, i.e. through the dynamic stiffness of foundations. The regularity of each bridge model was checked using several criteria proposed in EC8-2 and by authors.

Conclusions about the influence of SSI on dynamic behaviour of bridges are carried out.

2 BRIDGE MODELS AND SEISMIC ACTION

2.1 Bridge models

A parametric study of the dynamic response in transverse direction was carried out for eighteen different types of bridges [5]. All bridges are 4-span R/C structures 180 m long. The deck rests on three single column piers. The heights of the piers vary from 7 m, to 2x7=14 m and 3x7=21 m resulting in 18 different combinations. Each particular combination is defined by *Vijk*, where *i*, *j* and *k* denote the multipliers of the unit height of 7 m, for the first, second and third piers, respectively, Fig.1. The abutments are pinned in the transverse direction. In the longitudinal direction a fixed support at the left abutment and a roller support at the right abutment are assumed.

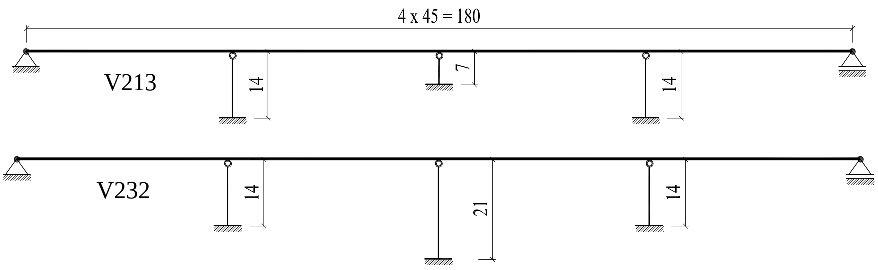

Figure 1. Layout of bridges V213 and V232

The deck is continuous, prestressed concrete box girder. The cross section of a deck and piers are presented in Fig. 2. Their material properties are given in Table 1.

Table 1. Cross-section properties

	A	A_s	I_x	I_y	I_z
deck	6.97 m^2	4.025 m^2	-	5.37 m^4	88.45 m^4
piers	4.16 m^2	-	7.3899 m^4	2.2059 m^4	-

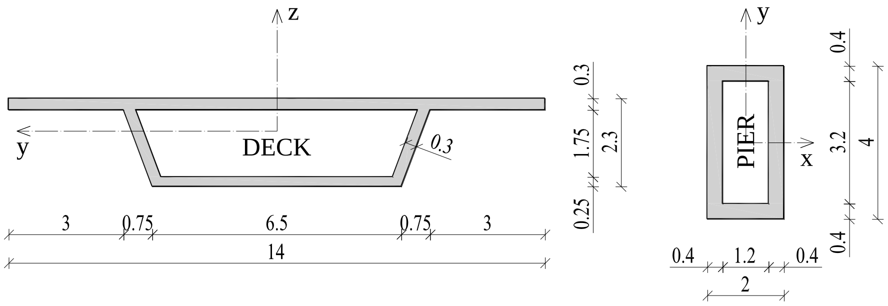

Figure 2. Cross-sections of deck and piers

2.2 Seismic action

The peak ground acceleration (PGA) at the bridge site is $a_g = 0.35g$, which represents a seismic event with a 475-year return period. The design response spectrums type I for the soil class B, C and D in accordance with the EC8-1 [6] are used. The behaviour factor is assumed to be q=3.5.

3 SOIL AND FOUNDATION

3.1 Soil

The subsoil is assumed to be a half space. In order to analyze the influence of the soil properties on the regularity of bridges, three soil types B, C and D [6], are applied. The soil characteristics are given in Table 2. Shear modulus G and shear wave velocity v_s are reduced, according to ATC (1978) [7], to the values $G_0=0.385G$ and $v_{s0}=0.625v_s$, that correspond to the effective soil acceleration $a_g=0.35g$.

Table 2. Soil characteristics

Soil class	v_s (m/sec)	ν	γ (kN/m^3)	ρ (t/m^3)	G (kPa)	G_0 (kPa)	v_{s0} (m/sec)
B	600	0.33	20.0	2.04	734400	282744	375.0
C	300	0.33	19.0	1.94	174600	67221	187.5
D	160	0.33	18.0	1.84	47104	18135	100.0

3.2 Dynamic stiffness of foundation

The influence of the soil and foundations on the dynamic response of bridges is taken into account through the use of equivalent springs and dashpots at the base of each column. The foundations are prismatic, with rectangular base dimensions l_xxl_y =11x8m and height equal to 2m, Fig. 3.

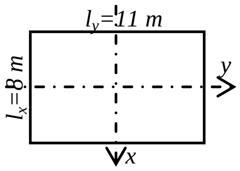

Figure 3 Foundation

The dynamic impedances of prismatic foundations resting on a half-space are calculated taking the solutions for arbitrary shaped foundation given by Dobry and Gazetas [8]. The dynamic stiffness K_i in the $i=x,y,z,\varphi$ direction is presented in the following form:

$$K_i = K_{i,stat}\ k_i + i a_o c_i\ = \mathrm{K}_i + i\mathrm{C}_i \quad , i=x,y,z,\varphi \tag{1}$$

where: $K_{i,stat}$ is static stiffness,

a_o is dimensionless frequency ($a_o = \omega B/c_s$) ,

$k_{i.}\ c_i$ are dimensionless stiffness and damping coefficients [1].

Dynamic impedance of the foundation is a complex, frequency-dependent quantity. The real part presents spring stiffness K_i while imaginary part presents dashpot damping C_i. Although dynamic impedances of the foundation are in general the frequency-dependent quantities, their low frequency values do not fluctuate appreciably with frequency and can be replaced with frequency independent springs and dashpots. The stiffness of spring K_i and damping of dashpot C_i, for soils type B, C, and D, were calculated for driving frequency equal to the first frequency of the structure. For one of the cases they are given in Table 3. The damping in the soil is assumed to be 5%.

Table 3. Spring and dashpots characteristics

	Stiffness [MN/m], [MNm/rad]			Damping [MNs/m]		
	B	C	D	B	C	D
K_z	9.033	2.148	0.579	0.115	0.055	0.0276
K_x	7.436	1.768	0.477	0.093	0.044	0.022
K_y	7.224	1.718	0.463	0.106	0.050	0.025
$K_{\varphi x}$	284.938	67.743	18.276	0	0	0

4 NUMERICAL ANALYSIS

In order to check the applicability of the fundamental mode method in the linear dynamic analysis of bridges, two type of analysis proposed in EC8-2 were performed: the fundamental mode method (FMM) and the response spectrum method (RSM). The dynamic analysis was carried out for two cases: (1) fixed-base columns, and (2) elasticaly supported columns (SSI).

The FMM is the equivalent static analysis which can be applied if the dynamic behaviour of the structure can be sufficiently approximated using SDOF model. Whether the FMM is adequate for linear dynamic analysis or not were investigated using three criteria defined in EC8-2: (1) eccentricity e_o between the centre of mass and centre of stiffness, (2) mass participation factor of the first vibration mode, (3) ρ - parameter for ductile bridges, and one additional criterion based on a relative displacement of the centre of mass for a fixed-base structure and structure with SSI.

The dynamic analysis was carried out using SAP2000 [9]. In all models, the damping of the bridge superstructure was approximated with the Rayleigh damping, by assuming a 5% modal damping ratio in the first and the second mode. The masses of the piers were concentrated at the top and the bottom of each pier. The stiffness of the piers was reduced to 50% of uncracked section. The soil influence was taken into account through the use of equivalent springs and dashpots.

4.1 Fixed-base structures

The analysis of behaviour of eighteen bridges with fixed-base columns was carried out using the fundamental mode method (FMM) and the response spectrum method (RSM). The regularity of bridges is considered using three criteria:

- Theoretical eccentricity e_o between the centre of mass and the centre of stiffness, relative to the bridge length L

$$e_o = \; x_m - x_s \; / L \quad , \tag{2}$$

where x_m is coordinate of the centre of mass, x_s is coordinate of the centre of the substructure stiffness and L is the bridge length.

- Relative difference D between the areas bounded by the deck displacement diagrams obtained by RSM and FMM methods, Fig. 4:

$$D = \frac{\sum \left| (d_{i,RSM} - d_{i,FMM}) \right| \Delta x_i}{\sum \left| d_{i,RSM} \right| \Delta x_i} \cdot 100 \tag{3}$$

where: $d_{i,RSM}$ is displacement in point i obtained by RSM method,
$d_{i,.FMM}$ is displacement in point i obtained by FMM,
Δx_i is the length between two points of girder.

- Modal mass participation factor.

According to 4.2.2.2 of EC8-2, the simplified fundamental mode method can be used if theoretical eccentricity e_o is less than 5%. Theoretical eccentricity e_o is equal to zero for all symmetric bridges, which means that FMM is applicable in dynamic analysis. For all non-symmetric bridges e_o is calculated and presented in Table 4. The e_o is higher than 5% in all cases, except for the bridge type V213 where e_o is equal to 1.9%. It means that all non-symmetric bridges, except V213, behave as irregular and the FFM is not acceptable for dynamic analysis.

Table 4. Theoretical eccentricity *e* [%]

Model	V112	V122	V132	V113	V123	V133	V213	V223	V233
e_o [%]	10,3	17,5	18,8	11,8	20,7	22,4	1,9	7,7	11

The fundamental mode method for flexible deck model (4.2.2.4 of EC8-2) is presented in Fig. 4. Starting with the inertial forces at the top of the column $F_{0,i}$, the displacements of deck $d_{0,i}$ are obtained and fundamental period T_k is calculated by the Rayleigh method. The deck displacements $d_{1,i}$ is calculated by applying the seismic forces $F_{1,i}$ obtained from the spectral acceleration of the design spectrum $S_d(T)$ (3.2.2.5 of EC8-1 [6]), corresponding to the fundamental period T_k of the bridge. If the displacements $d_{1,i}$ differ from the displacements $d_{0,i}$ the procedure has to be repeated starting with displacements $d_{1,i}$.

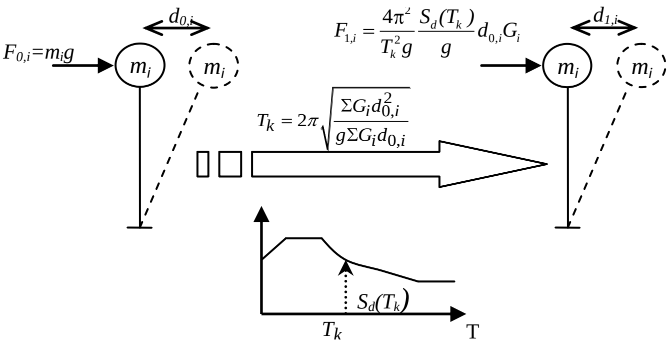

Figure 4. Fundamental mode method, flexible deck

The normalized diagrams of displacement along the deck were calculated for all types of bridges using the FMM and the RSM. In the RMS method 90% of the mass participation is taken into account. Typical diagrams are presented in Fig. 5.

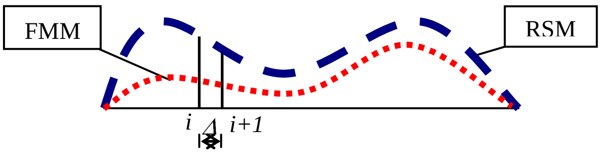

Figure 5. Displacement diagrams

Relative differences D between the areas bounded by the normalized displacement diagrams obtained by RSM and FMM are calculated using equation (3).

Period of vibration T_i, modal participation factor r_i and Σr_i, for first 3 modes are presented in Table 5, as well as the relative difference D.

Table 5. Periods T_i, modal participation factors r_i, Σr_i and D

	Mode	Ti (sec)	r_i	sum (r_i)	D(%)
V111	**1**	**0.193**	**0.97**	**0.97**	
	2	0.178	0.00	0.97	3.5
	3	0.146	0.03	1.00	
V222	**1**	**0.508**	**0.97**	**0.97**	
	2	0.343	0.00	0.97	0.6
	3	0.207	0.03	1.00	
V333	**1**	**0.862**	**0.97**	**0.97**	
	2	0.417	0.00	0.97	0.2
	3	0.223	0.03	1.00	
V121	1	0.317	0.69	0.69	
	2	0.178	0.00	0.69	11.8
	3	**0.157**	**0.31**	**1.00**	
V131	1	0.366	0.67	0.67	
	2	0.178	0.00	0.67	6.7
	3	**0.158**	**0.33**	**1.00**	
V232	**1**	**0.632**	**0.95**	**0.95**	
	2	0.343	0.00	0.95	0.3
	3	0.214	0.05	1.00	
V212	1	0.343	0.00	0.00	
	2	**0.317**	**0.94**	**0.94**	3.1
	3	0.157	0.06	1.00	
V313	1	0.417	0.00	0.00	
	2	**0.366**	**0.93**	**0.93**	1.1
	3	0.158	0.07	1.00	
V323	**1**	**0.632**	**0.99**	**0.99**	
	2	0.417	0.00	0.99	0.5
	3	0.214	0.01	1.00	
V213	1	0.396	0.22	0.22	
	2	**0.325**	**0.71**	**0.93**	55.7
	3	0.157	0.07	1.00	
V112	1	0.329	0.47	0.47	
	2	**0.187**	**0.53**	**1.00**	38.2
	3	0.150	0.00	1.00	
V122	1	0.443	0.76	0.76	
	2	0.245	0.04	0.81	7.9
	3	**0.164**	**0.19**	**1.00**	
V132	**1**	0.515	0.78	0.78	
	2	0.255	0.01	0.79	9.1
	3	**0.165**	**0.21**	**1.00**	
V113	1	0.388	0.46	0.46	
	2	**0.187**	**0.54**	**1.00**	37.8
	3	0.150	0.00	1.00	
V123	1	0.532	0.73	0.73	
	2	0.254	0.09	0.81	9.4
	3	**0.164**	**0.19**	**1.00**	
V133	1	0.629	0.76	0.76	
	2	0.268	0.04	0.80	3.8
	3	**0.165**	**0.20**	**1.00**	
V223	**1**	**0.579**	**0.94**	**0.94**	
	2	0.371	0.04	0.98	2.7
	3	0.210	0.02	1.00	
V233	**1**	**0.745**	**0.94**	**0.94**	
	2	0.373	0.02	0.96	0.8
	3	0.218	0.04	1.00	

Comparing the eccentricity, the relative mass participation factors r_i and the relative difference *D*, the following can be concluded:

- Eccentricity is not a relevant factor for regularity of bridges. The structure V213 whose eccentricity e_o is just 1.9% has a very high difference, *D*=55.7%, between the areas of displacements obtained by FMM and RSM, due to the influence of the second mode. So, the application of FMM in dynamic analysis of this bridge is not appropriate;
- The symmetric bridges with stiff end columns, V121 and V131, have *D*=11.8% and *D*=6.7%, respectively, which means that they behave as irregular ones, due to the influence of higher modes;
- All structures with modal mass participation factor in the first mode greater than 90% have *D*<3.5%, which ranks them in the class of regular bridges. This group includes all symmetric bridges except V121 and V131, and two non-symmetric bridges V223 and V233;
- The nonsymetric bridges V223 and V233 have theoretical eccentricity grater than 5% (e_o>5%), but the relative difference between RSM and FMM less than 5% (*D*<5%), which means that application of FMM is adequate, i.e. they behave as regular bridges.

This analysis shows that the theoretical eccentricity e_o could not be quite a relevant indicator for regular dynamic behaviour of bridges in the transverse direction. The FMM can be used for dynamic analysis of all symmetric bridges, except in the case of bridges with stiff end columns. Although some non-symetric bridges, (bridges with flexible substructure V223 and V233), have regular behaviour, the RSM is recommended for all non-symetric bridges, since the influence of higher mode can be detected only by modal analysis.

4.2 Soil-structure interaction

The influence of soil stiffness and damping on the regular behaviour of bridges with influence of SSI is analyzed for three different soil classes B, C and D (EC8-1). The case of fixed-base structure is treated as the fourth limiting case, G=∞. The dynamic impedances of the foundations are taken according to Section 3.2. The springs and dashpots are used for modeling the soil influence in the SAP2000. The link elements are connected with the superstructure by short rigid elements of one-meter length. The foundation masses are taken into account. The effects of wave propagation are neglected.

The part of results obtained for three regular bridges: V111, V232 and V333 and three irregular ones: V131, V123 and V213 is presented in the following.

The influence of soil prolongs the period of vibrations. This effect is more pronounced for weaker soils. The fundamental period of vibration T_1 and the relative differences between the fundamental periods of structures with fixed-base and structures where the SSI is included are presented in Table 6. The biggest change of the fundamental period of vibration occurs in the case of symmetric bridge with the shortest columns, V111, for all soil classes. The lowest influence of a soil class on the fundamental period of vibration is occurred in the case of bridge type V232.

Table 6. Fundamental periods of vibration and ΔT for different soil classes

	T [s]	T [s]	T [s]	T [s]	ΔT (%)	ΔT (%)	ΔT (%)
type	fixed	B	C	D	B	C	D
V111	0.242	0.265	0.329	0.489	9.5	36.0	102.1
V232	0.774	0.793	0.846	0.836	2.5	9.3	8.0
V333	1.031	1.045	1.071	1.198	1.4	3.9	16.2
V131	0.413	0.434	0.497	0.666	5.1	20.3	61.3
V123	0.609	0.622	0.697	0.817	2.1	14.4	34.2
V213	0.423	0.434	0.481	0.64	2.6	13.7	51.3

The SSI has an influence on the modal mass participation factor. The 90% of the modal mass participation is obtained for the structures with SSI when 4-6 modes of vibrations are included, while for the fixed-base structures 1-3 modes are sufficient to obtained this value. It means that higher number of modes have to be included in the response spectrum analysis in order to obtain 90% of mass participation in the case when SSI is included.

Table 7. Periods T_i, modal participation factors r_i and Σr_i for bridge V213

	Mode	T_i (sec)	r_i	sum (r_i)
V213	1	0.423	0.38	0.38
	2	**0.372**	**0.59**	**0.98**
	3	0.177	0.02	1.00
V213-B	**1**	0.434	0.36	0.36
	2	0.385	0.27	0.64
	3	0.183	0.01	0.65
	4	0.055	0.13	0.78
	5	**0.053**	**0.12**	**0.90**
V213-C	**1**	0.481	0.61	0.61
	2	0.405	0.07	0.68
	3	0.197	0.00	0.68
	4	0.112	0.13	0.81
	5	**0.107**	**0.12**	**0.93**
V213-D	**1**	0.640	0.71	0.71
	2	0.425	0.01	0.72
	3	0.218	0.01	0.73
	4	0.213	0.11	0.84
	5	**0.200**	**0.11**	**0.95**

The influence of SSI on modal response is presented in Table 7 for one of the cases - bridge V213. For the fixed-base structure 90% of modal mass is obtained when two vibration modes are included, while for structures elastically supported (soil class B, C and D), five modes have to be included to achieve this value. Almost the same results are obtained for other types of bridges.

The 4.1.4.2 of EC8-2 proposed that the SSI should be accounted for at piers where under the action of a unit horizontal load at the top, the flexibilities of the soil contribute more than 20% of the total top displacement. In this paper, the relative displacement Δ_d in the centre of mass due to earthquake is used as a measure of the SSI effect:

$$\Delta_d = \frac{\left| d_{flex} - d_{fix} \right|}{d_{flex}} \cdot 100\% \ . \quad (4)$$

In Eq. (4) d_{flex} is the displacement in the centre of mass when SSI is included, while d_{fix} is the displacement in the same point for fixed-base structure. If relative displacement Δ_d is less than 20% the influence of SSI is regarded as negligible.

The relative displacements were calculated for all cases using RSM and are presented in Table 8. For the soil class B, Δ_d is less than 20% for all cases, which means that SSI is not important. For the soil class C, Δ_d is higher than 20% for the bridges V111 and V213. For the soil class D the relative displacement Δ_d is higher than 20% for all bridges, except for symmetric bridges V232 and V333.

Table 8. Δ_d

Δ_d [%]			
type	B	C	D
V111	9.3	20	41.3
V232	2.3	7.6	11.3
V333	1.2	3.6	14.3
V131	4.7	15.9	31
V123	2.4	14.2	38.9
V213	11.4	25.8	37.5

The design seismic displacements d_E were calculated in accordance with 2.3.6.1 of EC8-2, as follows:

$$d_E = \pm \eta \mu_d d_{Ee} \ . \quad (5)$$

In Eq. (5) d_{Ee} is the displacements determined from the linear seismic analysis (RSM) based on the design spectrum in accordance with 3.2.2.5 of EC8-1 [6]. The damping correction factor is η=1 for ξ=5%. The displacement ductility μ_d is detrminated as follows:

- if $T \geq T_o = 1.25 T_c$, then $\mu_d = q$;
- if $T < T_o$, then $\mu_d = (q-1)\frac{T_o}{T} + 1 \leq 5q - 4$.

where T is the fundamental period, T_c is defined in accordance with 3.2.2.2 of EC8-1, and $q=3.5$ is the value of the behaviour factor.

Four diagrams of transversal deck displacements for symmetric bridges V111 and V313 and non-symmetric bridges V123 and V213, with fixed and elastically supported columns (soil class D), are presented in Figs. 6 and 7, respectively. The flexibility of the soil caused larger displacements of the deck. The displacements are larger as the soil is weaker. It should be pointed out that one part of deck displacement is occurred due to the rigid body rotation of piers, caused by soil deformation at the base.

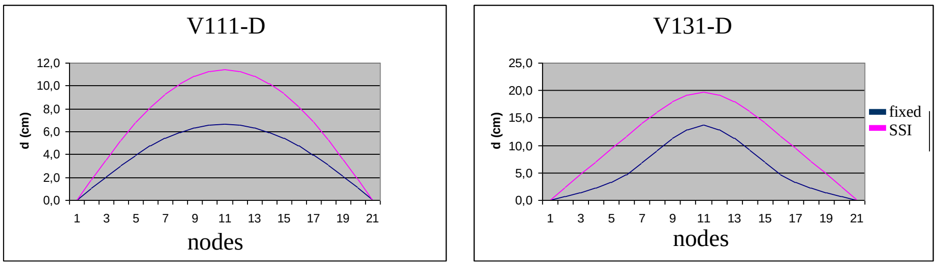

Figure 6 Diagrams of deck displacements, symmetric bridges, soil class D

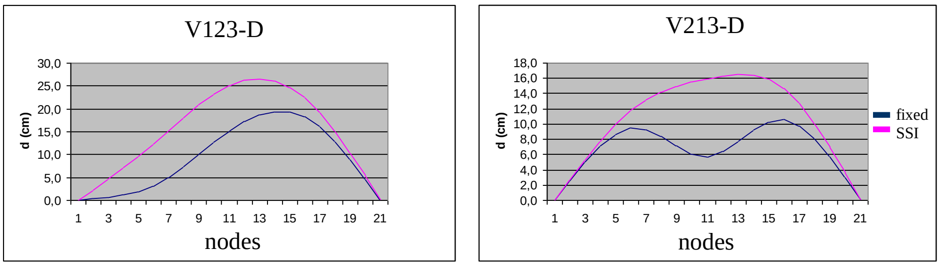

Figure 7 Diagrams of deck displacements, nonsymmetrical bridges, soil class D

In the case of bridges with irregular arrangement of the columns the soil flexibility has beneficial effect on the deck displacements, smoothing the diagram of displacements, as in the case V213-D, Fig.7.

4.3 Ductile bridges

In the case of ductile structures the regular behaviour of bridges was checked comparing the maximum and minimum of local force reduction factor calculated for each pier in accordance with 4.1.8 of EC8-2:

$$\rho = \frac{r_{max}}{r_{min}} \leq \rho_o \ . \qquad (6)$$

If the ratio of r_{max} to r_{min} differs for a factor larger than 2, the bridge is considered to have irregular behaviour. This requirement provides the optimum post-elastic seismic behaviour, which is achieved if plastic hinges develop simultaneously in as many piers as possible [4].

The local force reduction factor associated with member i under the specific seismic action is equal to

$$r_i = q\frac{M_{Ed,i}}{M_{Rd,i}} \quad . \tag{7}$$

where $M_{Ed,i}$ is value of the design moment at the plastic hinge, and $M_{Rd,i}$ is the design flexural resistance.

All piers have the same cross section, presented in Fig.2. The longitudinal reinforcement ratio is 1%. The behaviour factor q is assumed to be 3.5. The effective moment of inertia was calculated applying the expression specified in the EC8-2, Annex C, in the following form:

$$I_{eff} = 0{,}08 I_{un} + I_{cr} \ , \tag{8}$$

where I_{un} is the moment of inertia of the cross-section of uncracked pear, and I_{cr} is the moment of inertia of the cracked section at the yield point of the tensile reinforcement. The moment of inertia I_{cr} was calculated from the expression:

$$I_{cr} = M_y / (E_c \Phi_y) \ , \tag{9}$$

where M_y and Φ_y are the yield moment and the curvature of cross-section, respectively and E_c is the elastic modulus of concrete. The yield moment was estimated as 75% of ultimate moment M_u. The yield moment and the curvature were calculated using the program RESPONSE 2000 [8]. The obtained values are presented in Table 9.

Table 9. The yield moment M_y, curvature κ_y and I_{eff}

M_u, M_y, κ_y		
Ultimate moment M_u	51276,8	(kNm)
Yield moment M_y	38457,6	(kNm)
Curvature κ_y	0,00091	(1/rad)
$E_c I_{cr} = M_y/\kappa_y$	42261,1	(kNm^2)
$I_{eff}=0{,}08 I_{un}+I_{cr}$	1.933	(m^4)
I_{eff} / I_{un}	0.262	

As the design flexural resistance $M_{Rd,i}$ is the same for all piers, the ratio between maximum and minimum reduction factor of member i can be expressed as:

$$\rho = \frac{max M_{Ed,i}}{min M_{Ed,i}} \ , \tag{10}$$

where $_{max}M_{Ed,i}$ is maximum value of the design moment at the plastic hinge, and $_{min}M_{Ed,i}$ is minimum value of the design moment at the plastic hinge.

The parameters ρ were calculated for bridges with fixed base and bridges where SSI is included, and are given in Table 10.

Table 10. The parameter ρ

ρ						
	B		C		D	
type	fixed	SSI	fixed	SSI	fixed	SSI
V111	1.4	1.4	1.4	1.4	1.4	1.4
V232	1.4	1.5	1.4	1.4	1.4	1.3
V333	1.4	1.4	1.4	1.4	1.4	1.4
V131	2.9	2.9	2.8	2.7	2.8	2.1
V123	1.9	1.9	1.8	1.9	1.8	2.0
V213	4.4	4.3	4.4	4.2	4.4	3.8

The parameter ρ is less than 2 for all symmetric bridges, except for the bridge with stiff ends columns - V131, which means that all symmetric bridges except bridge V131 have regular behaviour. The most irregular bridge appears to be V213 – the structure with stiff central column and small eccentricity (e=1.9%). The soil flexibility tends to diminish irregular behaviour of this structure, increasing the flexibility of the central part, but the ρ remains still high, far from the proposed values. According to EC8-2, such a bridge can be designed using either a reduced q-value or the non-linear time history analysis.

5 CONCLUSIONS

A parametric study was undertaken in order to determine the effects of SSI on the regular seismic behaviour of bridges in linear dynamic response, as well as on the regular behaviour of ductile bridges, with different pier heights.

Presented results demonstrate that an eccentricity-based criterion [4] for selection whether the FFM is adequate or not for linear dynamic analysis is not always applicable. This criterion is adequate for almost all symmetric and some nonsymetric bridges. If the substructure is with stiff end (or central) column and relatively small theoretical eccentricity, neglecting higher mode of vibrations may lead to unsatisfactory results. In such cases it is better to apply the modal mass participation criteria and then, if necessary, perform the RSM or the time history analysis. These types of bridges perform the nonregular seismic behaviour.

In comparison with fixed-base structure, SSI caused larger deck displacements, equalizing displacements and smoothing the curvature of the displacement diagram. The SSI does not affect the regular bridge structures, where FMM is adequate. The SSI effect is more pronounced in the case of irregular bridges, where its influence is more beneficial than detrimental.

The soil flexibility diminishes the ρ factor, but not as significantly as to improve the dynamic behaviour of irregular ductile bridges.

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REFERENCES

[1] Sextos A, Kappos A, Pitilakis K, "Inelastic dynamic analysis of RC bridges accounting for spatial variability of ground motion, site effects and soil-structure phenomena. Part 2: Parametric study", Earthquake Eng. and Struct. Dyn, Vol **32**, pp. 629-652, 2003

[2] Jeremic B, Kunnath S, F. Xiong: "Influence of soil–foundation–structure interaction on seismic response of the I-880 viaduct, Engineering Structures", Vol **26**, pp 391–402, 2004

[3] Mylonakis G, Gazetas G, "Seismic soil–structure interaction: beneficial or detrimental?" Journal of Earthquake Engineering, Vol **4**, No 3, pp 277–301, 2000

[4] prEN 1998-2:2004, Eurocode 8: *Design of structures for earthquake resistance*, *Part 2: Bridges*, DRAFT No 5, European Committee for Standardization, June 2004

[5] Isaković T, M. Fischinger, P. Kante, "Bridges: when is single mode seismic analysis adequate?", Structures & Buildings, Vol **156**, Issue 2, pp. 165-173, 2003

[6] EN1998-1:2004, Eurocode 8: *Design of structures for earthquake resistance, Part 1: General rules, seismic actions and rules for buildings*, European Committee for Standardization, November 2004

[7] ATC, 1978, Tentative *Provisions for the Development of Seismic Regulations for Buildings*, Applied Technology Council, ATC-03 Report, Redwood City, California.

[8] Dobry R, Gazetas G: "Dynamic Response of Arbitrary Shaped Foundations", JGED, ASCE, Vol **112**, No 2, pp 109-135, 1986

[9] SAP2000 *Analysis Reference Manual*, Computers and Structures, Inc., Berkeley, California, USA

[10] RESPONSE 2000 www.ecf.utoronto.ca

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