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COMPUTING A 'REASONABLE' SPATIALLY VARIABLE EARTHQUAKE INPUT FOR EXTENDED BRIDGE STRUCTURES

Anastasios SEXTOS[1], Andreas KAPPOS[2] and Basil KOLIAS[3]

SUMMARY

During strong ground motion, it is expected that a bridge will be subjected to excitation that is non-uniform along the structure, in terms of amplitude, frequency content and arrival time, a fact primarily attributed to the wave arrival delay, the loss of coherency, and the effect of local site conditions. Although considerable research has been carried out over the last twenty years in all the aforementioned directions, the knowledge gained has only partially been reflected on modern seismic code provisions. Currently, it is only Eurocode 8 - Part 2 that has adopted quantitative provisions for tackling this complex phenomenon of asynchronous motion. As a result, the goal of this paper is to assess these current provisions by focusing on some typical bridge structures. Using a special purpose computer program, the simplified approach proposed by EC8 is evaluated against the results of more refined analysis; the latter involves multiple support excitation of the bridges using pier-dependent artificial accelerograms that account for the aforementioned three main sources of spatial variability of ground motion. The results indicate that the new EC8 provisions are easy to apply and provide a good qualitative prediction of the asynchronous motion effects on the bridge. However, as expected, their application is subject to limitations and has to be performed by exercising engineering judgement.

1. INTRODUCTION

In current practice it is customary to assume that, during an earthquake event all of the bridge supports experience identical ground motion time histories, even in the case of multi-span bridges of considerable overall and/or span length. This assumption of identical support ground motion is also implicitly made when performing an equivalent static or a response spectrum analysis. However, reality is far more complex, since extensive scientific research has shown that earthquake ground motion may significantly differ among the support points, especially for long bridges, in terms of amplitude, frequency content and arrival time, thus inducing, under certain circumstances, significant forces and deformations. These spatial and temporal variations of seismic motion can be primarily attributed to: travelling of the waves at a finite velocity, loss of their coherency in terms of statistical dependence (due to multiple reflections, refractions and superposition of the incident seismic waves propagation), effect of local soil conditions, as well as attenuation of motion due to geometrical spreading of the wave front and loss of kinetic energy. In addition to the above, seismic motion is further modified by the foundation, depending on its relative flexibility with respect to the soil, since the foundation is not always able to vibrate according to the displacement field that is imposed to it by the incoming waves. The first pioneering studies on the effect of non-synchronism of ground motion on bridge response date back to the ’60s, though it is only since the ’90s that this phenomenon has been seen from a more practical perspective. Having set up the fundamental constitutive framework of the spatially variable ground motion, the effort was gradually extended to applications on simple structures, while analytically derived solutions for identifying loss of coherency patterns [Luco and Wong, 1986, and Zendagui et al., 1999, among others] and generating spatially variable seismic motions were developed [Deodatis, 1996, Hao, 1989, Harichandran et al, 1990, Zerva, 1990].

[1] Lecturer, Department of Civil Engineering, Aristotle University of Thessaloniki, Email : asextos@civil.auth.gr
[2] Professor, Department of Civil Engineering, Aristotle University of Thessaloniki, Email : ajkap@civil.auth.gr
[3] Dr. Civil Engineer, DENCO Consulting Engineers Ltd., Athens, Email : bkolias@denco.gr

More realistic bridge configurations were also studied by various researchers such as Der Kiureghian [1996], Simeonov et al. [1997] and Romanelli et al. [2004], among others, implementing correspondingly refined analysis approaches and establishing the fundamental framework to consider the potential role played by multiple support excitation on the dynamic response of the structure itself. Recently, the effect of asynchronous motion on the inelastic dynamic behaviour of bridges has also been examined involving specific code-prescribed bridge configurations [Lou and Zerva, 2005], a set of parametrically modified realistic bridge structures [Deodatis & Shinozuka, 2000, Lupoi et al., 2005] or experimental results [Norman et al., 2006]. Extension of the proposed methodologies to account for the coupling effect of spatial variability of ground motion (SVGM), site effects (SE) and soil-structure interaction (SSI) within a comprehensive framework has been performed by Sextos et al. [2003b] and Sextos et al. [2005], while the effects of asynchronous excitation on curved [Sextos et al., 2004, Allam & Data, 2004] and isolated [Ates et al., 2006] bridges have also been studied. All the aforementioned efforts have a practice-oriented aim to provide a statistical basis for detecting systematic trends and quantifying the relative importance of the various phenomena involved in the seismic response of bridges. Inevitably, though, since current seismic design philosophy typically relies on energy dissipation through non-linear behaviour, the only available tool for a meaningful study of the problem is the generation of spatially distributed motions to be used in non-linear time-history analysis, a fact that is recognised in a forthcoming *fib* state-of-the-art document [*fib*, 2006].

Despite the major practical interest in generating such motions and the considerable research carried out over the recent years, the multi-parametric character and the complexity of the problem have not yet allowed the development of detailed guidelines in modern codes. As a result, the potential effect of asynchronous excitation is only partially considered. In particular, most modern codes deal with the problem solely, and rather indirectly, on the basis of seating length provisions, such as the US Standard Specifications for Highways and Transportation Bridges [AASHTO, 1996], [ATC, 1996], and the Japanese Design Specifications for Highway Bridges [2002]. According to AASHTO in particular, the required seismic design displacements are determined through any seismic analysis of the bridge, provided an acceptable analysis method is used. More specifically, a minimum bearing support length N is prescribed for the expansion ends of all girders, which is a function of the length of the deck L (in meters), the height H of the column or pier (in m), and the skew angle S of the support (in degrees), based on the following dual relationship and the Seismic Performance Categories (SPCs, in particular A, B, C and D as defined in AASHTO):

$$N(mm)=\begin{cases}(203+1.67L+6.66\mathrm{H})\cdot(1+0.000125S^2) & for: SPC_{A\,and\,B}\\(305+2.50L+10.0\mathrm{H})\cdot(1+0.000125S^2) & for: SPC_{C\,and\,D}\end{cases} \tag{1}$$

In case the displacements resulting from elastic analysis exceed the above minimum values, the values resulting from the analysis should be used in design. The Japanese Code, on the other hand, specifies the seat length S_E of a girder at the support as follows:

$$S_E(cm)=u_R+u_G\geq 70+\frac{L}{2} \tag{2}$$

where u_R is the differential displacement between the superstructure and substructure (in cm), u_G is the relative displacement of the ground occurring due to ground deformation between piers (in cm), and L is the clear span length (in m). An effort to relate the expected relative displacements δ_a of a multiply excited bridge system to the overall length L, has been made through a statistically derived amplification factor R_D proposed by Sextos et al. [2003b]:

$$\delta_a = R_D\,\delta_s = (0.8\ln(L) - 2.8)\,\delta_s \tag{3}$$

where δ_s are the relative displacements that would result from 'standard' synchronous motion analysis and L is the overall length (in m). Furthermore, a model to compute the differential displacements of points on the ground and on the top of an SDOF linear elastic system has also been proposed by Nuti & Vanzi [2005], while Kawashima and Sato [1996] suggested an alternative approach based on the use of a 'relative displacement spectrum'. The aforementioned research has not yet culminated into seismic code provisions due to the significant complexity of the particular problem. It is worth noting, though, that this lack of guidance has not prevented the use of more refined approaches (accounting for spatial variability of ground motion) for the design of important structures, such as the Metsovitikos bridge [Oldham et al., 2002] and the Rion-Antirion bridge [Combault et al., 2000] in Greece. On the other hand, the recent US guidelines for seismic design of bridges [ATC/MCEER, 2003] do not provide detailed guidance regarding the consideration of spatial variation of ground motion effects, despite the clear statement that is made about the significance of the particular issue. A suggestion is only made to refer to a publication by the Caltrans Seismic Advisory Board Adhoc Committee on Soil-Foundation-Structure Interaction [CSABAC, 1999] for the development of spatially variable acceleration time histories.

In the light of the foregoing remarks, Part 2 of Eurocode 8 for Bridges [CEN, 2004], seems to be the only seismic code worldwide that provides such a clear and detailed framework for considering the effect of spatial variability of ground motion in bridge design, through both a simplified and an analytical approach, the latter being included as an 'informative' annex. This represents a step forward with respect to the previous (1995) provisions, which were almost completely revised. Therefore, the key objective of this paper is to assess, based on the experience gained from previous research by the authors, the accuracy and range of applicability of the new EC8 provisions, with emphasis on the simplified procedure proposed in its main body (the one expected to be used for practical design). Based on the comparison of the results from the application of the simplified EC8 procedure with those of a comprehensive spatial variability analysis, both applied to three bridges of different structural configuration, the applicability of the new EC8 seismic design framework is evaluated. The description of the new design framework for spatial variability is first provided in the following section.

2. OVERVIEW OF THE EUROCODE 8 PROVISIONS REGARDING SPATIAL VARIABILITY

2.1 Overview

Part 2 of Eurocode 8, in its final version, clearly recognises that since spatial variability of seismic action is a situation wherein the ground motion at different supports of the bridge differs, the seismic action cannot be based on the characterisation of the motion at a single point. Moreover, EC8 prescribes that during design an adequate (albeit simplified) model should be implemented in order to account for the propagatory character of the seismic waves, as well as for the progressive loss of correlation between motions at different locations that arises from both propagation and potential differences in the mechanical properties of the (non-uniform) soil media. In order to address the previous requirements, a simplified approach is prescribed for the estimation of the pseudo-static effects, involving sets of appropriate displacements that are imposed statically at the supports of the bridge deck. According to EC8, spatial variability shall be considered for bridges of continuous deck when one or both of the following conditions apply:

- Soil properties along the bridge vary in such a way that the soil at the various supports corresponds to more than one category (as specified in Eurocode 8 – Part 1).
- Soil properties along the bridge are approximately uniform, but the length of the continuous deck exceeds a specified limit, L_{lim}; the recommended value of L_{lim} is $L_g/1.5$, where the length L_g is the distance beyond which motions can be considered as completely uncorrelated and is given in Table 1 as a function of soil category.

For the general case, the potential maximum values of the considered seismic action effect can be estimated through a simplified procedure described below. This method should be followed, unless a more accurate analysis is carried out. Additionally, a more detailed procedure for the assessment of the asynchronous motion effects in the frequency domain according to Der Kiureghian [1996] is also included as an informative annex while guidance of the generation of artificial spatially variable ground motions is also provided.

2.2 The simplified approach

The idea (put forward by the third author, while serving on the EC8-2 drafting panel) is simple and practical: since motion is different between support points, the various bridge supports are subjected to different values of (location-dependent) earthquake accelerations, which are partially correlated; as a result, pseudo-static internal forces develop. From the numerous combinations of relative support vibration, two cases are identified as the most critical: a) all piers are subjected to ground displacements of the same sign (but not the same magnitude) and b) the two piers in each pair of two successive piers are displaced in opposite directions. According to these two deformation cases (called Set A and Set B displacements), the structure is subjected to pseudo-static forces, whose effects are then combined with those that result from a typical uniform excitation analysis using the SRSS (square root of the sum of squares) rule. It is also noted that the simplified method is based on the conventional data defining the design seismic action according to EC8-1, i.e. the design spectrum (estimated on the basis of seismicity PGA and soil class). The basic steps of the procedure are:

Step 1: The inertia response of the bridge is calculated using a single input seismic action for the entire structure. In case that a response spectrum analysis is performed, a single response spectrum is used, whereas a uniform accelerogram is applied when the structure is analysed in the time domain. According to EC8, in both cases, either the spectrum or the corresponding accelerogram should correspond to the most severe ground type underneath the bridge supports. It has to be noted herein, that this provision is already an upper bound of the level of the seismic forces since, especially for long bridges (of long fundamental period) the adoption of a uniform soft soil profile (i.e. of category C or softer, which may often be found at the centre of the bridge) may result to significant and unrealistic increase in the seismic demand.

Step 2: Set A consists of relative displacements d_{ri} (where i is the support identification number) that depend on the design ground displacement d_g, which in turn corresponds to the ground type of support i, in accordance with Eurocode 8 - Part 1; L_i is the distance (projection on the horizontal plane) of support i from a reference support i = 0 (i.e. most commonly selected as one of the two abutments). The displacements d_{ri} are calculated from the following relationship:

$$d_{ri} = \varepsilon_r L_i \leq d_g \sqrt{2} \text{ where } \varepsilon_r = d_g \sqrt{2} / L_g \text{ and } d_g = 0.025 \cdot a_g \cdot S \cdot T_C \cdot T_D \quad (4)$$

based on the definition of the well-known parameters of the elastic spectrum, given in Eurocode 8 – Part 1. It is clear that the d_{ri} displacements do not necessarily increase proportionally, since a limit value of $d_g \sqrt{2}$ applies (Fig. 1). It is also noted that since the design ground displacement d_g is soil type dependent, hence support dependent, the coefficient ε_r should be preferably denoted as ε_{ri}.

Step 3: The displacements d_{ri} that correspond to Set A, are then applied simultaneously with the same sign (+ or –) to all supports of the bridge (1 to n) in the horizontal direction considered as shown in Figure 1 and the corresponding (static) action effects are derived. Set A displacements are applied separately, in each horizontal direction of the analysis, on the relevant support foundations or on the soil end of the relevant spring representing the soil stiffness in the analytical model.

Step 4: Set B represents the influence of ground displacements occurring in opposite directions at adjacent piers. This is accounted for by assuming displacements Δd_i of any intermediate support i (>1) relative to its adjacent supports i-1 and i+1 considered undisplaced (see Figure 2).

$$\Delta d_i = \pm \beta_r \varepsilon_r L_{av,i} \quad (5)$$

where $L_{\alpha v,i}$ is the average of the distances $L_{i-1,i}$ and $L_{i,i+1}$ of intermediate support i from its adjacent supports i-1 and i+1 respectively. For the end supports (0 and n) $L_{\alpha v,0} = L_{01}$ and $L_{\alpha v,n} = L_{n-1,n}$ and β_r is a factor accounting for the magnitude of ground displacements occurring in opposite direction at adjacent supports. The recommended values for β_r are:

$\beta_r = 0.5$ when all three supports rest on the same ground type
$\beta_r = 1.0$ if the ground type at one of the supports is different from that at the other two

while ε_r is as defined for set A above. If a change of ground type appears between two supports, the maximum value of ε_r should be used. As also shown in Fig. 1, Set B displacements are eventually of opposite sign between all adjacent supports i and i+1, for i = 0 to n-1. The desired Set B displacements d_i are then calculated on the basis of the already derived displacements Δd_i as follows:

$$d_i = \pm \Delta d_i / 2 \quad \text{and} \quad d_{i+1} = \pm \Delta d_{i+1} / 2 \quad (6)$$

Step 5: As previously, the calculated Set B displacements are statically imposed at the bridge supports and the subsequent corresponding (static) action effects are obtained.

Step 6: In each horizontal direction (i.e. longitudinal or transverse) the most severe effects resulting from the pseudo static analyses of Steps (3) and (5) shall be combined with the relevant effects of the inertia response of Step (1), using the SSRS rule. The result of this combination constitutes the effects of the analysis in the direction considered. For the combination of the effects of the different components of seismic action (including vertical motion), the rules prescribed in Eurocode 8 Parts 1 and 2 are applicable and the probable maximum action effect E, due to the simultaneous occurrence of the components of the seismic action along the horizontal axes X, Y and the vertical axis Z, may be estimated as:

$$E = \sqrt{E_x^2 + E_y^2 + E_z^2} \quad (7)$$

where E_x, E_y, E_z correspond to the independent seismic action along each axis. It has to be noted here, that it is not clearly stated in the code whether, for the case of inelastic analysis, both inertial and pseudo-static (due to Set A and Set B) action effects should be reduced by the same behaviour factor q (as one would normally expect).

Table 1: Limiting length to consider spatial variability effects, as a function of soil category

Ground Type	A	B	C	D	E
L_g (m)	600	500	400	300	500

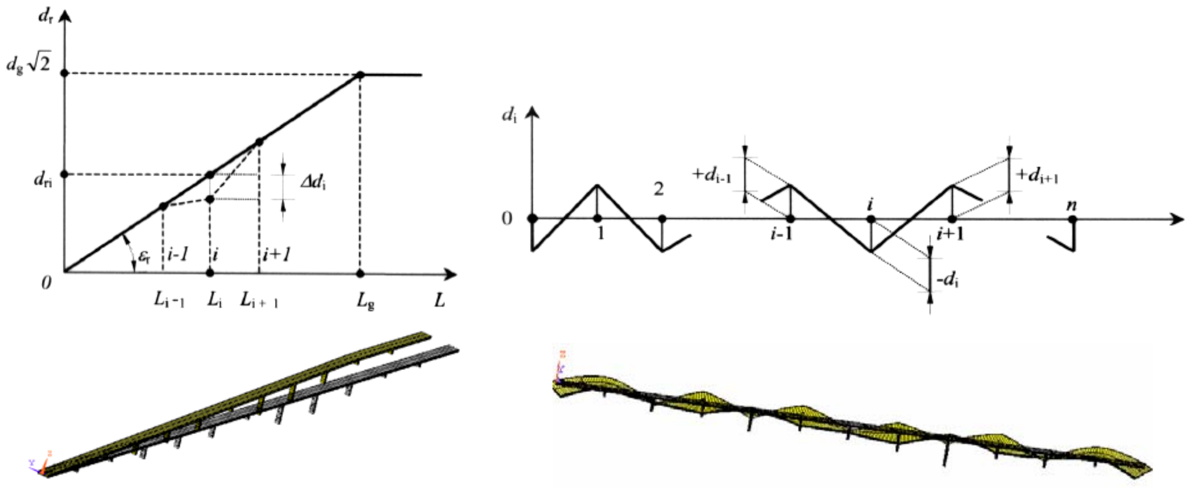


Figure 1: Imposed displacement patterns and resulting deformed shapes (indicative) according to Eurocode 8 simplifying approach for Set A (left) and Set B (right) pseudo-static displacements

3. COMPARATIVE SIMPLIFIED AND REFINED ANALYSES

3.1 Overview

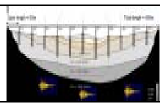
Having focused on the EC8 provisions for performing simplified spatial variability analyses of bridges, a computer tool was developed to automatically derive the required displacement sets and impose them on any FE model build in ANSYS [Sextos, 2005]. This import facility is easy to use and unaffected by the complexity of the FE model. Moreover, the analysis is also automatically performed, leading to the (user defined) monitored response components that result from the static Set A and Set B analysis (typically transverse or longitudinal pier top displacements, or bending moments along the specified axis). By returning to the EC8-SSVAB code, the deformed shape of the FE model is then loaded, and the monitored action effects derived by the ANSYS engine can then be superimposed on the effects of the uniform excitation analysis described in Step 1. In order to assess the range of applicability of these simplified provisions, three previously studied bridges were selected (namely Bridge A, Bridge B and Bridge C), having different structural configuration in terms of overall length (i.e. 200m, 600m, and 478m, respectively) span length, pier-deck connection, and curvature in plan (Table 2). All structures have been extensively studied under both synchronous and asynchronous excitation. The results from the application of the EC8 simplified procedure are compared with those derived by comprehensive spatial variability analysis in the following. It is noted in the framework of this analysis the elastic EC8 response spectrum is used (q=1). In case that the design spectrum is used instead, the resulting action effects will de consequently reduced by the behavior factor q, hence the relative importance of the (Part A and Part B) pseudo-static displacements would have been higher.

The simplified method is *directly* applied *primarily* for the estimation of the increased distress at the piers and not for the calculation of the resulting displacements. The reason is that if the latter was also of interest, both directions of excitation should be used. In particular, equation (4) has to be applied twice, starting from either reference abutment each time, while the minimum (of the two cases) resulting displacement increase has to be considered in order to avoid disproportionately increased displacements at the piers that are close to the opposite abutment (Fig. 2). Instead, EC8 correctly prescribes increased levels of minimum overlap lengths (§ 6.6.4) to ensure the function of the support under extreme relative displacements that may result due to spatial variability of seismic motion. It is also noted that the relevant supports are estimated with an additional safety margin equal to $\sqrt{2}$ compared to the stresses induced at the bridge system.

3.2 Application to a straight, 200m bridge

Bridge A is a straight, asymmetric, four-span bridge of 200m total length, supported on hollow section piers of height from 7 to 21m. Its main characteristics are summarized in Table 2 while a more detailed description of the bridge under consideration can be found in Sextos et al. [2003b]. Despite the fact that the particular bridge is not long, spatial variability issues shall be considered in order to investigate whether the assumption of an abrupt change in soil conditions along its length would result to modified dynamic response despite the short length.

Table 2: Summary of bridges assessed

Structural configuration and analysis				
Bridge ID #		Bridge A	Bridge B	Bridge C
Bridge geometry	Number of spans	4	12	12
	Span length	50m	50m	44 - 55m
	Total length	200m	600m	638m
	Pier-deck connection	Monolithic	Monolithic	Monolithic / Bearings
	Curvature	No	No	In plan
	Foundation	Pile Group	Pile Group	Pile Group
Spatial variability parameters	Loss of coherence	Yes	Yes	Yes
	Wave-passage	Yes	Yes	Yes
	Site response	No*	No*	No*
	SSI	No*	No*	No*
References (structural configuration)		Sextos et al., [2003b]	Sextos et al., [2003b]	Sextos et al., [2004]
* In this particular set of analyses. Comprehensive SVGM/SSI analyses can be found in the references given above				

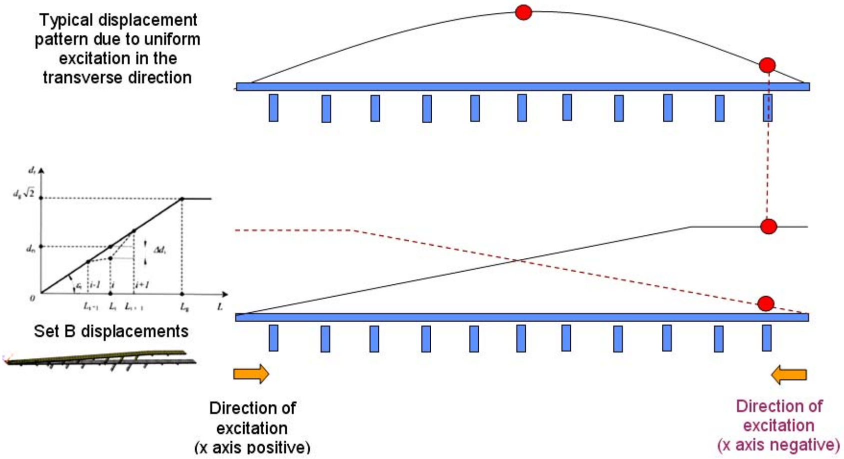


Figure 2: Superposition of the displacement patterns in order to extend the EC8 simplified approach to the estimation of the displacement increase due to (pseudo-static) spatial variation of ground motion

After defining Set A and Set B displacements for the particular bridge, static analysis of the structure is performed and the resulting pseudostatic bending moments at the base of all piers are derived. Using the SRSS rule, the latter are then superimposed with the corresponding action effects that were obtained from the dynamic elastic analysis of the bridge that was performed using a synthetic accelerogram compatible with the EC8 elastic response spectrum. The complete EC8 simplified analysis results are given in Sextos and Kappos [2005]. It is found (Table 3) that for the case of uniform soil conditions, the effect of SVGM is negligible (i.e. lower than 1%) when using the simplified (EC8) approach. Even a comprehensive multiple support excitation analysis in the time domain, utilizing motions that are generated using the computer code ASING [Sextos et al., 2003a] to match the EC8 response spectrum, the arrival time delay effect and the (adopted) Luco and Wong [1986] coherency loss pattern, essentially verifies that for the particular short bridge, the effect of asynchronous excitation is not detrimental and to a certain degree, it is even beneficial (i.e. a reduction of 1% to 6% in the resulting bending moments is observed). The latter is an observation in agreement with the findings of previous studies [Zerva, 1990] as well as with results from the comparative analysis of a number of short bridges [Sextos et al., 2003b]. As a result, the overall length limit that the new EC8 provisions set for considering SVGM effects (i.e. 300m for Soil Class C profiles) is deemed realistic. On the other hand, the suggestion that spatial variation of ground motion issues have to be accounted for, independently of the bridge length, 'when the soil at the various supports corresponds to more than one categories' may initially seem too conservative since it essentially refers to a large number of practical cases. Nevertheless, it has to be recalled that independently of the seismic code framework used, the dynamic response of a bridge under multiple support excitation is inevitably dependent on the frequency content of the selected ground motion (response spectrum), hence in cases of significantly different soil profiles along the bridge, the modification of the bridge response can be substantial even for short bridges [Lou and Zerva, 2005]. As a result, the EC8 provisions for considering SVGM effects in short bridges are deemed reasonable and physically justified.

3.3 Application to a straight, 600m, twelve span bridge

In order to assess the validity of the above observations for the case of a longer bridge without completely changing the basic structural configuration, a straight, 600m, twelve span bridge is also analysed. Its main characteristics are summarized in Table 2 while a more detailed description of the bridge under consideration can be found in Sextos et al., 2003b. As previously, the two sets of horizontal displacements (i.e. Set A and Set B) are defined and imposed at the foundation level of all piers and abutments, while the analysis results are superimposed with those arising from the 'standard' time history analysis (i.e. for uniform excitation). The ratios of the resulting bending moments using both methods are summarized in Table 3. For the case of uniform soil profile assumption, the EC8 spatial variability simplified approach leads to a negligible increase in pier base bending moments that does not exceed 4%; the latter is contrasted with the *extreme* 39% and 52% demand increase at Piers 3 and 5 resulting from the refined time history analysis scheme. However, the average distress increase among all piers is 1% and 2% among the two methods. For the case that the non-uniform soil conditions are accounted for, by modifying the coefficients ε_r and β_r through the EC8 approach and considering the complete soil stratification at each support through the comprehensive multiple support excitation scheme, it is seen that the EC8-predicted seismic demand is increased (*at maximum*) by 13% compared to a much larger (*maximum*) increase (at a ratio of 3.1) that is observed using the refined method. Moreover, the location of the most affected piers cannot be captured by the EC8 simplification and the significant relative site amplification at the vicinity of the central Piers 5 and 6 that are founded on much softer uppermost layers is completely missed. However, the average distress increase among all piers is not equally diverging (2% and 35% between the simplified and the refined method). In general, it can be stated that the simplified EC8 approach is not able to capture the extent of the dynamic effects of spatial variation of ground motion on the bridge response nor the subsequent demand distribution, a fact that is attributed to the pseudo-static character of the particular approach and to the corresponding neglect of higher modes modified participation that is typically observed in multiply excited structures [Sextos et al., 2005]. Nevertheless, the reluctance of EC8 to propose a more substantial distress increase due to SVGM effects is reasonable and justified if one considers that any 'refined' procedure, although indeed comprehensive and more accurate, is inevitably case dependent on the assumptions (and related uncertainty) regarding the frequency content of the incoming waves, the exact soil conditions and the coherency loss pattern adopted. Along these lines, further research is needed especially regarding the validation of the existing methods with measurements of both the asynchronous seismic ground motion and the structural response.

3.4 Application to the (curved, 638m long) Krystallopigi bridge

The investigation presented above was also extended for the case of curved bridges. This decision is further stimulated by the fact that EC8 defines L_i as the 'distance (projection on the horizontal plane) of support *i*,' thus hinting to the use of the simplified approach to curved bridges. For this purpose, a real, already constructed bridge with significant curvature in plan is selected (Bridge C); the Krystallopigi bridge is a twelve-span structure of 638m total length, curvature radius equal to 488m and piers of height that varies between 11 and 27m. Its main characteristics are summarized in Table 2 while a more detailed description of the bridge under consideration and its inelastic dynamic behaviour under multiple support excitation is given in Sextos et al. [2004]. By utilizing the procedure followed for Bridges A and B, the corresponding Set A and Set B displacements are calculated for Bridge C and are then imposed at the support points. The resulting ratio is depicted in Table 4 and is compared to the corresponding ratio derived using the asynchronous dynamic excitation of the structure for two different target frequency contents (i.e. EC8 and Kozani earthquake elastic spectrum) and six different angles of incoming shear waves incidence. Again, the 13 support-dependent accelerograms were generated using the method and assumptions described above. The results indicate that as a general trend, it can be stated that the EC8 approach is found to be in agreement with previous studies [Sextos et al., 2004] with respect to the importance of the pseudo-static component of the structure. However, it is also noted that, despite the fact that the maximum increase ratio in the EC8-induced bending moments (ratio 2.5) at the base of the piers (i.e. Pier 2) is comparable to the maximum increase observed when the comprehensive multiple excitation analysis is performed (ratio 2.1 and 2.2 for the two reference earthquake motions), the seismic demand distribution along the length is significantly different between the simplified and the refined approach. This can be again attributed to the fact that the dynamic response of Bridge C under asynchronous excitation is even more complicated, since the interplay between the pseudo-static and the dynamic component of motion depends also on the selected reference earthquake and the angle of wave incidence. As a result, it is considered that for such complex structures as bridges with significant curvature in plan, the EC8 simplified approach predicts well the expected maximum distress increase but it cannot yield a reliable estimate of the distribution of the (increased due to SVGM effects) seismic demand. For such complex cases, a more refined analysis scheme should be used (as EC8 also clearly and correctly suggests), preferably within the framework of a parametric scheme.

Table 3: Comparison of Pier Top Displacements of Bridge B (600m) using the simplified (in blue) and the comprehensive approach (in red).

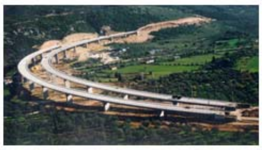
Case A: Uniform soil conditions

Pier Base Bending Moment (kNm)	PIER1	PIER2	PIER3	PIER4	PIER5	PIER6	PIER7	PIER8	PIER9	PIER10	PIER11	
UNIFORM EXCITATION	21600.0	23860.0	68070.0	105600.0	70230.0	25430.0	46090.0	61990.0	49120.0	10390.0	22310.0	
SET A - EC8	78.8	45.3	44.0	3.1	53.2	23.5	32.3	153.6	1344.1	1455.6	886.6	
SET B - EC8	2626.2	2582.7	3895.5	2921.7	3461.2	1194.6	2669.3	2136.8	3294.1	2631.0	2642.4	
SRSS SIMPLIFIED EC8	21759.2	23999.4	68181.4	105640.4	70315.3	25458.1	46167.2	62027.0	49248.7	10816.3	22483.4	
RATIO Masynchronous / Msynchronous												AVERAGE
EC8 Approach (based on Elastic Spectrum)	1.007	1.006	1.002	1.000	1.001	1.001	1.002	1.001	1.003	1.041	1.008	1.006
Refined: Wave Passage Effect	0.943	1.049	1.346	0.796	1.160	0.951	0.805	0.756	0.811	0.758	0.890	0.933
Refined: Wave Passage + Coherency Loss	1.093	0.976	1.395	0.875	1.520	1.072	0.819	1.005	0.955	0.827	0.733	1.024

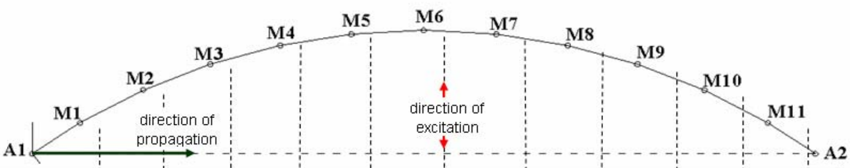
Case B: Soil conditions change considerably

Pier Base Bending Moment (kNm)	PIER1	PIER2	PIER3	PIER4	PIER5	PIER6	PIER7	PIER8	PIER9	PIER10	PIER11	
UNIFORM EXCITATION	21600.0	23860.0	68070.0	105600.0	70230.0	25430.0	46090.0	61990.0	49120.0	10390.0	22310.0	
SET A - EC8	78.8	45.3	44.0	3.1	53.2	23.5	32.3	153.6	1344.1	1455.6	886.6	
SET B - EC8	5252.4	5165.4	7791	5843.4	6922.4	2389.2	5338.6	4273.6	6588.2	5262	5284.8	
SRSS SIMPLIFIED EC8	22229.6	24412.8	68514.4	105761.5	70570.4	25542.0	46398.2	62137.3	49578.1	11737.1	22944.5	
RATIO Masynchronous / Msynchronous												AVERAGE
EC8 Approach (based on Elastic Spectrum)	1.029	1.023	1.007	1.002	1.005	1.004	1.007	1.002	1.009	1.130	1.028	1.022
Wave Passage + Coh. Loss + Site Effects	0.615	1.196	1.439	0.837	1.942	3.148	1.469	1.451	1.071	0.609	1.054	1.348

Table 4: Comparison of Pier Top Displacements of Bridge C (638m) using the simplified (in blue) and the comprehensive approach (in red), the latter for various angles of wave incidence.



Case A: Uniform soil conditions



Pier Base Bending Moment (kNm)	PIER1	PIER2	PIER3	PIER4	PIER5	PIER6	PIER7	PIER8	PIER9	PIER10	PIER11	
UNIFORM EXCITATION	97198.0	82023.0	36708.0	77305.0	80281.0	82239.0	95648.0	93743.0	38283.0	58183.0	50557.0	
SET A - EC8	1376.4	1241.5	1086.5	803.6	851.4	872.4	852.8	801.6	698.3	564.9	264.8	
SET B - EC8	99490.0	97813.7	85626.1	81484.5	58542.0	69750.2	62908.5	55944.2	61282.7	76686.5	82906.4	
SRSS SIMPLIFIED EC8	139095.7	127659.1	93169.1	112322.9	99362.6	107838.3	114484.7	109170.3	72261.0	96262.1	97105.9	
RATIO Masynchronous / Msynchronous												AVERAGE
EC8 Approach (based on Elastic Spectrum)	1.431	1.556	2.538	1.453	1.238	1.311	1.197	1.165	1.888	1.654	1.921	1.577
Kozani Earthquake Record												
Wave passage parallel to chord	1.00	1.07	1.02	1.18	0.70	2.02	0.88	0.76	1.12	0.88	1.01	1.058
Wave passage at 30°	1.01	1.04	1.09	1.20	0.72	2.05	0.84	0.86	1.13	0.86	1.08	1.080
Wave passage at 45°	1.07	0.98	1.12	1.18	0.77	1.99	0.78	0.96	1.10	0.80	1.09	1.076
Wave passage at 60°	1.06	1.05	0.96	1.19	0.88	1.47	0.74	1.04	0.97	0.81	1.06	1.021
Wave passage at 75°	0.82	1.07	0.97	1.16	0.97	1.34	0.86	1.04	0.96	0.94	1.03	1.015
Wave passage perpendicular to chord	0.90	1.07	1.09	0.99	1.04	1.05	1.04	0.98	1.01	1.03	0.94	1.013
EC8 Compatible Record												
Wave passage parallel to chord	1.32	1.16	0.92	0.56	0.33	0.77	0.57	0.48	0.71	1.67	0.78	0.843
Wave passage at 30°	1.32	1.16	0.92	0.56	0.33	0.77	0.57	0.48	0.71	1.67	0.78	0.843
Wave passage at 45°	1.27	1.09	0.77	0.39	0.29	0.68	0.58	0.35	0.73	1.64	0.61	0.765
Wave passage at 60°	1.32	1.16	0.92	0.56	0.33	0.77	0.57	0.48	0.71	1.67	0.78	0.843
Wave passage at 75°	1.38	1.21	0.99	0.63	0.38	0.76	0.71	0.64	0.67	1.62	0.87	0.897
Wave passage perpendicular to chord	1.25	1.15	0.91	0.55	0.35	0.78	0.58	0.55	0.78	2.24	0.77	0.899

4. CONCLUSIONS

Based on the analysis of the aforementioned three bridges using both the refined multiple support dynamic excitation approach and the new Eurocode 8 - Part 2 provisions for accounting for spatial variability, the following relative advantages and drawbacks of the latter can be identified:

Advantages

- The provisions adopted by EC8 are clearly a step forward not only compared to the previous version of the code, but also with respect to all modern seismic codes worldwide. The sources of spatial variability are adequately addressed and due consideration is given both in a qualitative and a quantitative sense.

- Spatial variability analysis is now required to be performed for bridges with overall length shorter than 600m, which was the limit imposed in the previous version of the code for mandatory consideration of spatial variability effects; this is in line with the findings of recent research (e.g. Sextos et al. 2003).
- The soil-dependent length limits set for considering spatial variability effects is in good agreement with recent research results from the literature even for bridges of moderate overall length.
- A detailed and well-established method is provided in the Informative Annex for addressing ground motion asynchronism in the frequency domain. Moreover, guidance is provided for the performance of (asynchronous) non-linear time-history analysis using sample motions that are generated starting from power spectra consistent with the elastic response spectra at the various supports.
- The simplified method provided is efficient, simple and can be physically justified.
- The EC8 simplified approach captures well the bridge response in cases where the effect of asynchronous motion is either minor or beneficial (i.e. for short bridges on uniform soil conditions) and results to no stress increase in the bridge.
- From a qualitative point of view the EC8 simplified approach predicts well the magnitude of the average bending moment increase among all piers.
- The above increase is higher for longer bridges and/or non-uniform soil conditions as also confirmed by both previous studies and physical interpretation.

Disadvantages

- The simplified method of EC8 is a pseudostatic method. As such it is evident that it cannot capture either the dynamic properties of the structural system or the frequency content of the excitation as far as the asynchronous character of the action is concerned. These effects remain the same as resulting from synchronous excitation of all supports. As a result, the (modified) seismic demand distribution and the potential triggering of higher modes cannot be captured.
- Although the simplified method as given in the standard is applicable for bridges of general geometry (straight or curved), larger deviations from the more rigorous solution are probable in the case of curved bridges. This may be attributed to the fact that curved bridges present a coupling of longitudinal and transverse modes that is not present in straight bridges.
- Sets A and B of EC8 displacement patterns are essentially independent of the motion used, its amplitude and frequency content. Hence, the extent of their contribution to the action effects resulting from uniform excitation dynamic analysis is essentially dependent on the decisions made for the initial (uniform excitation) analysis.
- Regarding displacements, the simplified method cannot be directly used. However, EC8-2 in general adopts the need for an increased reliability against unseating at moveable joints (i.e. the relevant support lengths are estimated in § 6.6.4 with an additional safety margin equal to $\sqrt{2}$ times the safety margin used for stresses resulting from the simplified method at the bridge system).
- Although the simplified approach is easy to follow, the calculation of Set A and Set B input displacements is relatively time consuming in the case of parametric analyses and/or long bridges. Therefore the development of a relevant computer code may be deemed necessary [i.e. Sextos, 2005].

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