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EVALUATION OF THE NEW EUROCODE 8-PART 2 PROVISIONS REGARDING ASYNCHRONOUS EXCITATION OF IRREGULAR BRIDGES

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ABSTRACT

During strong ground motion, it is expected that bridge structures are subjected to excitation that is non-uniform along their longitudinal axis in terms of amplitude, frequency content and arrival time, a fact primarily attributed to the wave arrival delay, their 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 provisions for tackling this complex phenomenon of asynchronous motion, which have been revised in its final version. 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 developed by the authors, the simplified approach proposed by EC8 is critically compared with the results of more refined analysis; the latter involves multiple support excitation of the bridges using pier-dependent artificial accelerograms that account for all the aforementioned three main sources of spatial variability of ground motion. The results indicate that although the new EC8 provisions contribute to a more accurate representation of earthquake loading, their application is subject to a number of limitations and has to be performed with particular caution and exercising engineering judgement.

INTRODUCTION

As bridge seismic design projects aim at a safe performance under unfavourable geological, seismotectonic and geotechnical conditions, the need typically arises for advanced design capabilities towards a number of uncertainties involved especially, and rather disproportionally, related to the definition of a ‘reasonable’ earthquake loading. Nowadays, 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 length and/or span length. It should be pointed out that the assumption of identical support ground motion is implicitly made also 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 [1] to the: *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), e*ffect of local soil conditions,* and *attenuation of motion* due to geometrical spreading of the wave front and loss of kinetic energy. Additionally 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 the ground motion on bridge response date back to the ’60s [2], though it is only since the ’90s that this phenomenon has been seen from a more practical perspective. Having set up the fundamental and constitutive framework of the spatially variable ground motion, the effort was gradually extended to applications on simple structures, while analytically derived solutions for generating spatially variable seismic motions were developed [3], [4], [5], [6]. More realistic bridge configurations were also studied by various researchers such as in references [1], [7] and [8], 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 [9], or a set of parametrically modified realistic bridge structures [10], [11], [12], [13]. Extension of the proposed methodologies to account for the coupling effect of spatial variability, site effects and soil-structure interaction within a comprehensive framework has been performed by Sextos et al. [14], while the importance of asynchronous excitation on curved bridges has also been studied [15], [16], [17]. 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 [18].

Despite the major practical interest of generating such motions and the considerable research carried out over the last years, the multi-parametric character and the complexity of the problem has not yet led to the issue 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) [19], ATC-32 [20], and the 2002 Japanese Design Specifications for Highway Bridges [21]. According to the AASHTO code, the required seismic design displacements are determined through any seismic analysis of the bridge provided that the analysis method is acceptable. Additionally, AASHTO also prescribes a minimum bearing support length N for the expansion ends of all girders [19], [22], 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):

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

In case the displacements resulting from the 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:

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

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. [12]:

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

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 a SDOF linear elastic system has also been proposed by Nuti & Vanzi [23], while Kawashima and Sato [24] suggested an alternative approach based on the use of a 'relative displacement spectrum'. The aforementioned research has not yet been reflected on modern seismic 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 [25] and the Rion-Antirion bridge [26] in Greece.

Along these lines, the latest version of Part 2 of Eurocode 8 (Bridges) [27], 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. In the light of this, 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). Furthermore, a computer program is developed that calculates the EC8-prescribed imposed displacements, facilitates their automatic import at the supports of finite element models (of any modelling complexity) to be set up independently, performs the subsequent static analyses for both displacement scenarios and returns the resulting additional action effect of the selected bridge members. 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 critically evaluated. The description of the new design framework for spatial variability is first provided in the following section.

OVERVIEW OF THE 2004 EC8 PROVISIONS REGARDING SPATIAL VARIABILITY OF GROUND MOTION

Part 2 of Eurocode 8, in its latest (prENV) version [27], clearly recognises that as 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 proposed 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).

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

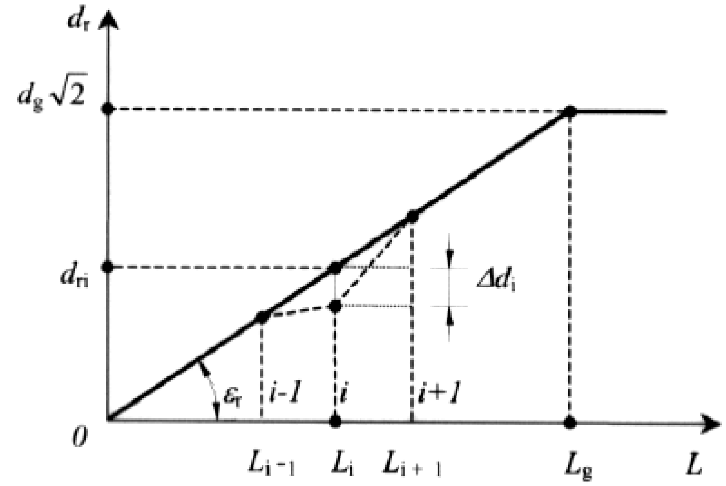


Figure 1: Set A of imposed displacements according to Eurocode 8 simplifying approach

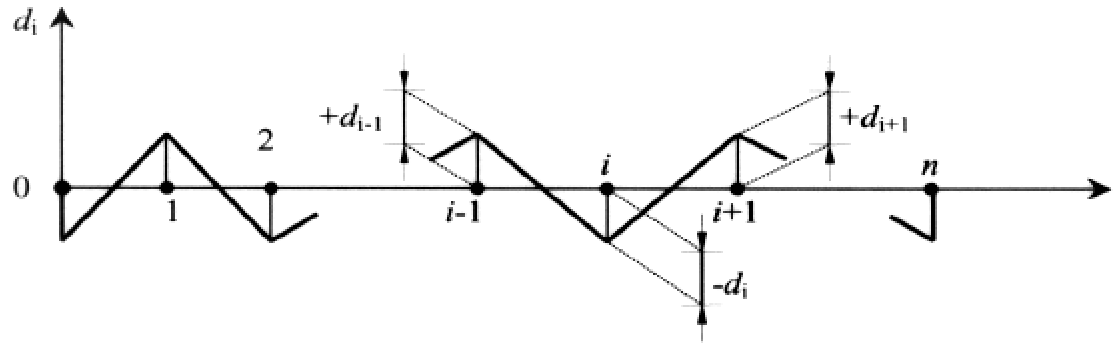


Figure 2: Set B of imposed displacements according to Eurocode 8 simplifying approach

− Soil properties along the bridge are approximately uniform, but the length of the continuous deck exceeds an appropriate limiting length, L_{lim} (of recommended value $L_{lim} = L_g/1.5$) where the length L_g is the distance beyond which the soil 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 with no exception unless a more accurate analysis is carried out. Additionally, a more detailed procedure is also provided as an informative annex for the assessment of the asynchronous motion effects in the frequency domain according to Der Kiureghian [28].

Overview of the simplified approach

The idea is simple and interesting: 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, see Fig. 1, 2), 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. The basic steps of the procedure are summarized below:

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:

$$d_{ri} = \varepsilon_r L_i \leq d_g \sqrt{2} \tag{4}$$

where $\varepsilon_r = \dfrac{d_g\sqrt{2}}{L_g}$ and $d_g = 0.025 \cdot a_g \cdot S \cdot T_C \cdot T_D$ 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).

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

where $L_{av,i}$ 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*) $L_{av,0} = L_{01}$ and $L_{av,n} = L_{n-1,n}$ 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. The desired Set B displacements d_i are then calculated on the basis of the already derived displacements Δd_i as follows:

$$\begin{aligned} d_i &= \pm \Delta d_i / 2 \\ d_{i+1} &= \mp \Delta d_{i+1} / 2 \end{aligned} \qquad (6)$$

As also shown in Figure 2, Set B displacements are eventually of opposite sign between all adjacent supports *i* and *i+1*, for *i* = 0 to *n-1*.

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:

$$\mathrm{E} = \sqrt{\mathrm{E}_x^2 + E_y^2 + E_z^2} \qquad (7)$$

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).

IMPLEMENTATION INTO A COMPUTER CODE

Due to the structure of the above calculation scheme, it was deemed useful to write a program carrying out the successive calculation steps, in order to be facilitate the application of the method for the case of long bridges (especially those having a large number of supports) and the execution of parametric analyses. Along these lines, the computer program EC8-SSVAB (Eurocode 8-based Simplified Spatial Variability Analysis of Bridges) was developed that allows the quick and accurate analysis of the structure due to the derived Set A and Set B support displacements. A snapshot of the code is illustrated in Figure 3. As it can be seen in the logical diagram presented in Figure 4, apart from the calculation of the required displacement sets, EC8-SSVAB also generates an export file with the use of APDL (ANSYS Parametric Design Language), a scripting language for optimizing model pre- and post-processing [29]. This file is then directly imported into the FE code ANSYS and the calculated displacements are automatically imposed at all the corresponding bridge supports of the original finite element model that is assumed to have been prepared using ANSYS. This import facility is easily performed, independently of the complexity of the FE model created. 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 super-imposed with the effects of the uniform excitation analysis described in Step 1.

Having focused on the EC8 provisions for performing simplified spatial variability analyses of bridges and developed the EC8-SSVAB tool to automatically derive the required displace-ment sets, it was deemed necessary to attempt to assess the range of applicability of the particular simplified provisions. To this effect, 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 477m, respectively) span length, pier-deck connection, and curvature in plan. All structures have been extensively studied [12],[15] under both synchronous and asynchronous excitation. The particular characteristics of each structure, as well as the comparison between the application of the EC8 simplified procedure and a comprehensive spatial variability analysis are presented in the following.

THE CASE OF A 200m FOUR-SPAN BRIDGE

Bridge A is a straight, asymmetric, four-span bridge of 200m total length, supported on hollow section piers of height that varies from 7 to 21m. Despite the fact that the particular bridge is short, spatial variability issues shall be considered due to the abrupt change in soil conditions along its length. The concrete deck consists of a hollow box cross-section, which was taken as uniform along the length of the bridge, while the pier and foundation character-istics are presented in Figure 5(left). The piers are assumed monolithically connected to the superstructure and the abutment bearings are pinned in the transverse direction, and free to slide in the longitudinal one. A more detailed description of the bridge under consideration can be found elsewhere [12]. 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.

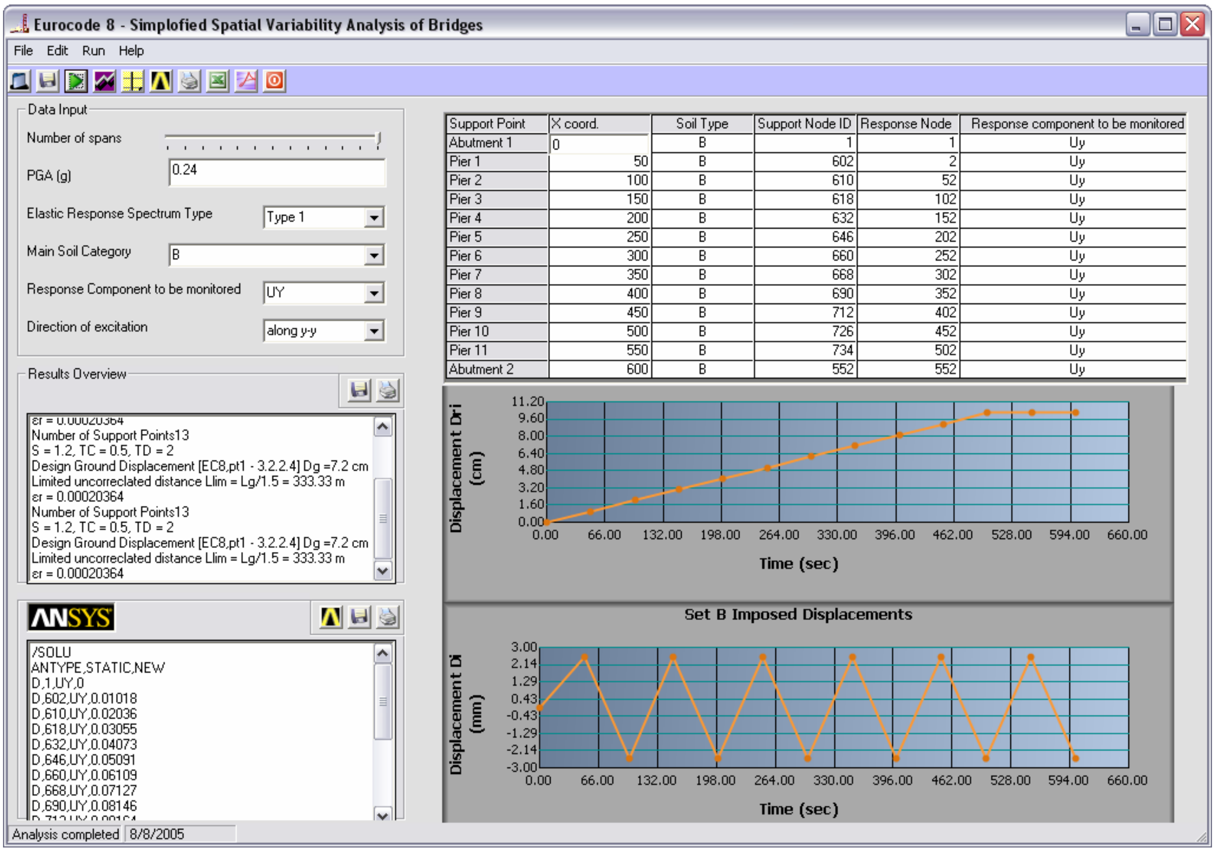


Figure 3: Overview of the EC8-SSVAB tool for Simplified Spatial Variability Analysis of Bridges

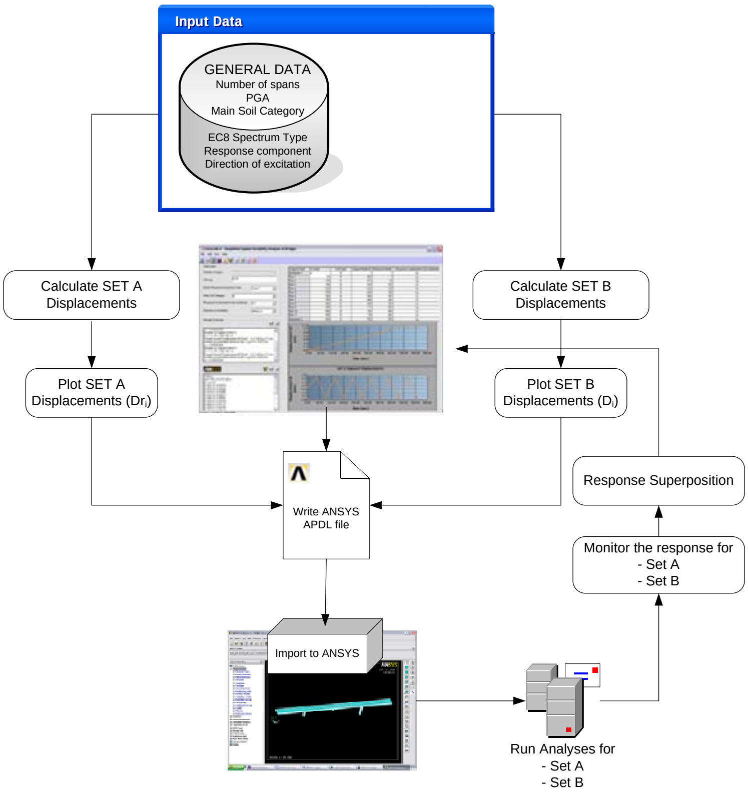


Figure 4: Schematic overview of the EC8-SSVAB computer code

The ratio of the resulting total moment to the initial moment (from inertial loading) is summarized in Table 3 for two distinct cases: a) the assumption of a uniform soil profile and b) the adoption of a realistic support-dependent foundation soil that matches the given properties in Figure 5(left). Apparently, in the framework of the EC8 simplified approach, this ratio by definition exceeds 1.0 independently of the above distinct cases. For the case of uniform soil conditions it is observed that the relative increase due to pseudo-static distress is indeed minor (i.e. lower than 1%) an observation that is anticipated for a short bridge on a uniform soil – in fact, for such a case it is not mandatory to apply the EC8 spatial variability provisions. Moreover, a refined dynamic analysis using different accelerograms at the five bridge support points, generated using the computer code ASING [12] to match the EC8 response spectrum, the arrival time delay effect and the (adopted) Luco and Wong [30] 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 [6]. When the more realistic non-uniform soil conditions are accounted for, the EC8 simplified approach again initially leads to relatively low increase of the pier base bending moments (i.e. not more than 1%). It has to be recalled, though, that this ratio is dependent on the action effects that are derived from the dynamic analysis for uniform excitation; hence the ratio is inevitably dependent on the frequency content of the selected ground motion (response spectrum); this sensitivity has to be investigated through further parametric analysis.

Another important aspect that is not clarified in the code commentary is whether, both inertial and pseudo-static (due to Set A and Set B) action effects are subject to reduction by the behaviour factor *q* or it is only the (elastic) bending moment resulting from the time history analysis that has to be reduced before being superimposed with the Set A and Set B induced static bending moments. In order to investigate both possibilities, the previous analysis additio was rerun, usimg the latter assumption, with a behaviour factor q=3.0. In this case, the ratio was higher leading to a distress increase of 9%. This latter increase is generally confirmed by the refined asynchronous time history analysis, using spatially distinct accelerograms that account for the effect of support-dependent multi-layered soil profiles over an elastic bedrock. The latter comprehensive method, reveals a potential increase of 10% to 30%, leading to the conclusion that the EC8 simplified approach is in the right direction in requiring an increase of the pier base bending moments, even for such a relatively small bridge.

Nevertheless, the piers which are detrimentally affected (i.e. Pier 2) or relieved (i.e. Pier 3) according to the dynamic analysis, not only cannot be predicted by the EC8 simplified method, which indicates quite different trends. This important discrepancy can be attributed to the fact that the implementation of Set A and Set B imposed displacements prescribed in the simplified approach, accounts excessively for the pseudo-static component that results from the asynchronous excitation, and neglects the effect of spatial variability on the dynamic response of bridges, which has been shown ([14], [31]) to be equally important, especially with respect to triggering higher modes. This trend is expected to be even more pronounced in the case of longer, hence more flexible, bridges, since it has been shown by focusing on 20 different structures [14] that the pseudo-static forces that result from asynchronous excitation decrease as structural stiffness decreases. It is clear therefore, that the dynamic behaviour of bridges subjected to spatially variable support motion is a very complex issue, being depended not only on the modes excited, but also on the interplay between the pseudo-static and the dynamic component that is triggered in each case.

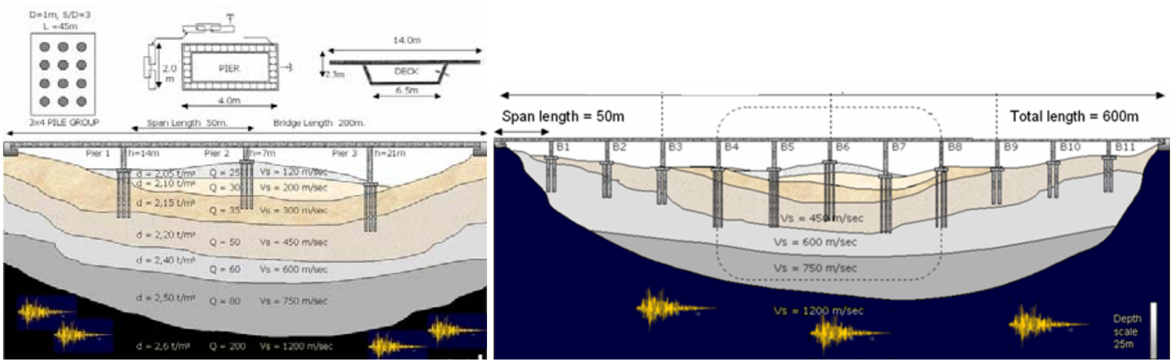
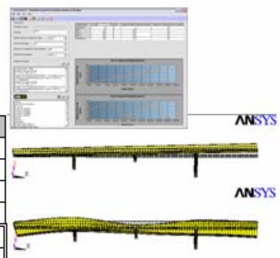


Figure 5: Overview of Bridge A (left) and Bridge B (right) after [14]

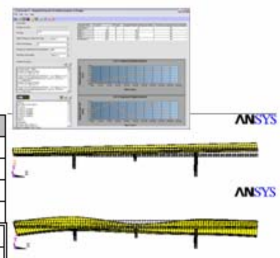
Table 2: Comparison of Pier Top Displacements of Bridge A (200m) using the simplified and the comprehensive approach.

Case A: Uniform soil conditions



Pier Base Bending Moment (kNm)	ABUT1	PIER1	PIER2	PIER3	ABUT2
UNIFORM EXCITATION		69980.00	49640.00	39040.00	
SET A - EC8		112.51	29.58	87.00	
SET B - EC8		3277.80	1062.70	2913.70	
SRSS SIMPLIFIED EC8		70056.81	49651.38	39148.68	
RATIO Masynchronous / Msynchronous					
SRSS Simplified EC8 Approach (based on Elastic Spectrum)		**1.001**	**1.000**	**1.003**	
SRSS Simplified EC8 Approach (based on Design Spectrum)		**1.010**	**1.002**	**1.025**	
Refined: Wave Passage Effect		**0.94**	**0.99**	**0.99**	
Refined: Wave Passage + Coherency Loss		**0.98**	**0.96**	**0.94**	

Case B: Soil conditions change considerably



Pier Base Bending Moment (kNm)	ABUT1	PIER1	PIER2	PIER3	ABUT2
UNIFORM EXCITATION		69980.00	49640.00	39040.00	
SET A - EC8		112.51	29.58	87.00	
SET B - EC8		6555.60	2125.50	5827.40	
SRSS SIMPLIFIED EC8		70286.48	49685.49	39472.62	
RATIO Masynchronous / Msynchronous					
SRSS Simplified EC8 Approach (based on Elastic Spectrum)		**1.004**	**1.001**	**1.011**	
SRSS Simplified EC8 Approach (based on Design Spectrum)		**1.039**	**1.008**	**1.096**	
Refined: Wave Passage + Coherency Loss + Site Effects		**1.108**	**1.309**	**0.995**	

THE CASE OF A 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, the bridge shown in Figure 5(right) is analysed. Bridge B consists of 12 spans of 50m length each, having identical pier and deck cross-sections as Bridge A. Its larger overall length (600m) and fundamental period in the transverse direction allow for the comparative assessment of the effect of spatial variability on long and flexible structures using again both the EC8 simplified approach and the afore-mentioned comprehensive approach [12]. 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 ratio of the resulting bending moments using both methods is summarized in Table 4, while the corresponding deformed shape of the bridge under the specific static displacements is depicted in Figure 6. At first, the simplified approach is indeed capable to predict that a longer bridge is more sensitive to the effects of asynchronous motion. For the case of uniform soil profile assumption, the EC8 spatial variability simplified approach leads to a moderate increase of pier base bending moments that varies from 1% to 6% with the exception of an extreme 32% increase that is observed in Pier 10 (corresponding to the assumption that q applies to the inertial effects only otherwise the effect of spatial variability does not exceed 4%); the latter is contrasted with the 39% and 52% demand increase at Piers 3 and 5 that arises from the performance of the refined time history analysis scheme. Despite the overall similarity in the predicted increases by the two approaches, in terms of the *maximum* detrimental effect of spatial variability on the (elastic) bending moments developed at the base of the piers, the distribution of the (modified) demand along the bridge is again completely different.

The same observation can be made 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 indeed predicted that the seismic demand on the (longer) Bridge B can be almost doubled (i.e. in Pier 10, for q=3.0), a magnitude that is verified by the refined time history approach whereas a similar increase (at a ratio of 1.9 and 3.1) is observed. Again, though, 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.

Based on the above, it can be stated that the EC8 simplified approach is a useful tool for assessing the *magnitude* of the expected seismic demand increase but its spatial distribution along the bridge cannot be accurately assessed. However, this is a limitation that also applies to a certain degree for the asynchronous dynamic excitation as well, since it has been shown by numerous studies that a number of parameters (for instance the target frequency content of the generated artificial motions) affect the way that the structure responds under multiple support excitation. The latter, though, can be tackled through a parametric analysis scheme, an approach that is not feasible nor prescribed when applying the EC8 simplified provisions.

Another interesting aspect of the EC8 approach is that for both Bridges A (200m) and B (600m) the relative contribution of Set B displacements to the additional pseudo-static forces is significantly higher than that of Set A. In other words, the structure complies more easily to the smooth deformation shape resulting from Set A dislocations, despite their higher magnitude with respect to the Set B relative displacements. An additional problem arising from this is that since Set B displacements are direction independent, there is no real merit in utilizing both abutments as reference point for defining Set A displacements.

What is also not clear in the simplified EC8 provisions is whether (and how) the simplified approach can be implemented to estimate the potential increase in the bridge deck displacements. First, it is clear that Set A imposed pseudo-static displacements, which are proportional to the distance from the reference point, essentially dominate the displacement calculation process. If displacements resulting from equation 4 are applied indiscriminantly to all piers (as opposed to applying equation 4 twice, starting from either abutment each time, and taking the minimum of the two), the resulting deformation pattern is clearly unrealistic (Fig. 6). This is attributed to the fact that Set A results to a deformation shape where the maximum (transverse) displacement of the deck is located at the edge piers where the inertially derived transverse response is significantly smaller (i.e. the central piers exhibit the maximum displacement). As a result, the ratio of the superimposed displacements to the ones arising from the uniform excitation dynamic analysis is not realistic (i.e. for the particular case, it exceeds 40).

Table 4: Comparison of Pier Top Displacements of Bridge B (600m) using the simplified and the comprehensive approach.

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											
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**
EC8 Approach (based on Design Spectrum)	**1.065**	**1.051**	**1.015**	**1.003**	**1.011**	**1.010**	**1.015**	**1.005**	**1.023**	**1.324**	**1.068**
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**
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**

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											
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**
EC8 Approach (based on Design Spectrum)	**1.238**	**1.192**	**1.057**	**1.014**	**1.043**	**1.039**	**1.059**	**1.021**	**1.081**	**1.867**	**1.233**
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**

Case A: Uniform soil conditions

Pier Top Displacement (cm)	PIER1	PIER2	PIER3	PIER4	PIER5	PIER6	PIER7	PIER8	PIER9	PIER10	PIER11
UNIFORM EXCITATION	0.537	0.583	4.920	7.530	5.07	0.625	6.77	8.87	3.57	0.25	0.52
SET A - EC8	1.018	2.04	3.05	4.07	5.09	6.11	7.12	8.17	9.12	10.15	10.2
SET B - EC8	0.83	0.24	0.22	0.02	0.08	0.23	0.08	0.01	0.06	0.22	0.24
SRSS SIMPLIFIED EC8	1.419	2.135	5.793	8.560	7.185	6.146	9.825	12.059	9.794	10.155	10.216
RATIO Masynchronous / Msynchronous											
SRSS Simplified EC8 Approach	**2.642**	**3.662**	**1.177**	**1.137**	**1.417**	**9.834**	**1.451**	**1.360**	**2.743**	**40.622**	**19.646**

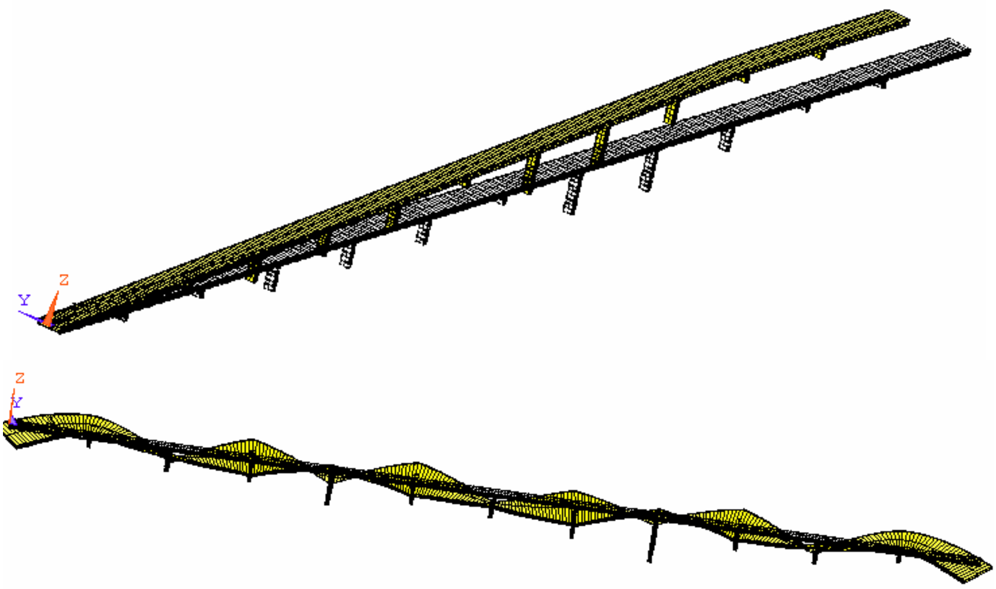


Figure 6: Deformed shape of the (600m) Bridge B when statically subjected to the EC8 Set A (top) and Set B (bottom) imposed displacements

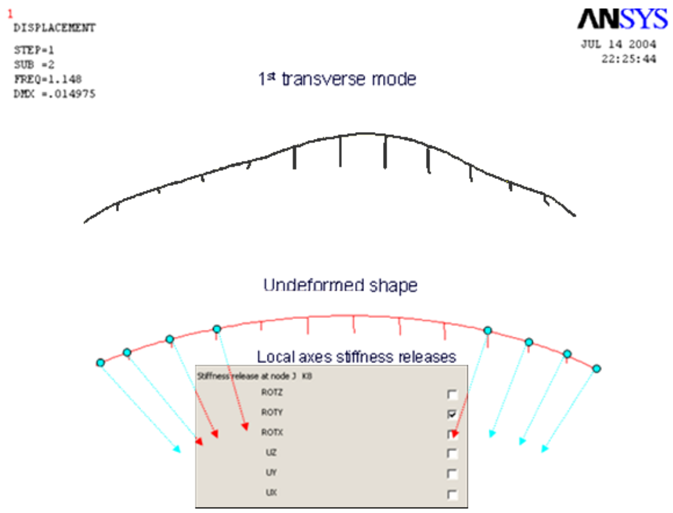
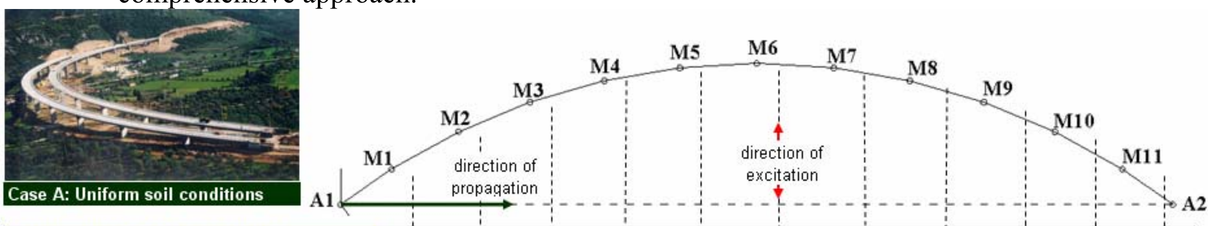


Figure 7: FE model and pier-deck connection of Bridge C

Table 5: Comparison of Pier Top Displacements of Bridge C (638m) using the simplified and the comprehensive approach.



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											
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**
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**
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**
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**
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**
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**
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**
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**
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**
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**
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**
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**
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**

THE CASE OF KRYSTALLOPIGI BRIDGE

Having preliminary assessed the applicability of the new EC8 provisions regarding spatial variability by focusing on two straight bridges, it was deemed interesting to extend the investigation in 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. A bearing type pier-to-deck connection (Figure 7) is adopted for the 8 edge piers, while the interior piers are monolithically connected to the deck. This structure has been extensively studied in terms of its inelastic dynamic behaviour, including the effect of asynchronous motion [15]. 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 5 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 computer code ASING [12] and accounting for the time delay and the (adopted) Luco and Wong [30] coherency loss pattern.

As a general trend, it can be stated that the EC8 approach is found to be in agreement with previous studies [15] with respect to the importance of the pseudo-static component of the structure, despite its significant length, a fact that is highlighted by the significant Set B bending moments summarized in Table 5. However, it is also noted that, despite the fact that the *maximum* increase ratio in the EC8 induced bending moments at the base of the piers (i.e. 2) is comparable to the *maximum* increase observed when the comprehensive multiple excitation analysis is performed, the seismic demand distribution along the length is different in the two approaches. This may be attributed to the fact that the dynamic response of the Bridge C under asynchronous excitation is even more complicated than in A and B, since the interplay between the pseudo-static and the dynamic component of motion depends also on the selected 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 is not adequately reliable, at least with respect to seismic demand distribution.

CONCLUDING REMARKS REGARDING THE NEW EC8 PROVISIONS FOR THE SPATIAL VARIABILITY OF GROUND MOTION

Based on the analysis of the aforementioned three bridges using both the refined multiple support dynamic excitation approach and the new EC8 provisions for accounting for spatial variability, the following relative advantages and drawbacks of the latter can be identified:

Advantages

- The provisions adopted by Eurocode 8 - Part 2 are definitely 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 qualitatively and quantitatively.
- 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.
- A detailed and well-established method is provided in the Informative Annex [28] for addressing ground motion asynchronism in the frequency domain.
- The simplified method provided is efficient, simple and can be physically justified.
- There is no stress increase in the bridge in cases that the effect of asynchronous motion is either minor or beneficial (i.e. for short bridges on uniform soil conditions).
- From a qualitative point of view the simplified approach predicts well the magnitude of the *maximum* 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

- 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.
- It is questionable whether the simplified approach can be used for the estimation of the effect of asynchronous motion on structural displacements (absolute or differential). If this was indeed the purpose of the Eurocode 8 drafting committee, the procedure to be applied should have been clearly stated in the code; a background note or reference to published work is clearly advisable for the entire simplified procedure.
- Within the proposed pseudo-static approach, and despite the fact that the *maximum* stress increase in any of the piers is generally adequately predicted, the application of the EC8 simplified approach *neglects the dynamic effect* of spatial variability and the potential triggering of higher modes. Consequently, it may lead to erroneous results with respect to the (modified) seismic *demand distribution*.
- Although EC8 hints to the applicability of the simplified approach to curved bridges, it was found that their complexity probably limits its application to straight bridges.
- Although the simplified approach is relatively easy to follow it requires support by software currently not part of FE codes, especially in the case of parametric analyses and/or long bridges; such software was developed by the writers as part of this study.

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