Synthesis of displacements for the SVEGM analysis

Síntese de deslocamentos para a análise da variabilidade espacial da acção sísmica

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Abstract

The spatial variability of earthquake ground motion (SVEGM) can be an important issue in seismic analysis. This phenomenon has three main sources: (i) the wave passage effect, due to the finite velocity of propagation; (ii) the incoherence effect due to the multiplicity of wave propagation paths and its consequent overlapping; and (iii) the local site amplification, due to the spatial variation of soil characteristics. The first two factors increase with the horizontal dimension of the structures, while the latter may occur for small distances. It is thus recommended to study a large number of structures with respect to SVEGM. This paper describes a procedure to synthesize correlated displacements, in time domain, complying with the requirements of codes of practice, which can be used with a step-by-step approach for the direct integration of the dynamic equilibrium equations. In order to illustrate the procedure, the results from the analysis of a single degree of freedom (SDOF) model, subjected to multi-support excitation, are presented and discussed.

Resumo

A variabilidade espacial da acção sísmica pode desempenhar um papel relevante na análise sísmica. O fenómeno deve a sua origem a, essencialmente, três causas distintas: (i) o efeito de passagem de onda, originado pela velocidade finita de propagação; (ii) o efeito de incoerência, devido à multiplicidade de caminhos de propagação das ondas e sua consequente sobreposição, e (iii) o efeito de amplificação local devido à variação espacial das características geológicas. Enquanto que os dois primeiros factores aumentam com a distância entre as fundações, o terceiro pode ocorrer em distâncias pequenas. Consequentemente, deve estudar-se o efeito da variabilidade espacial num conjunto vasto de estruturas. Neste artigo propõe-se uma metodologia para a síntese de deslocamentos correlacionados, no domínio do tempo, com base nos regulamentos que definem a acção sísmica, e que poderão ser utilizados numa análise por integração directa das equações de equilíbrio. Para ilustrar o procedimento descrito apresentam-se e discutem-se os resultados da análise dum OL1GL sujeito a uma excitação diferente em dois apoios.

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1   Introduction

The spatial variability of earthquake ground motion can change qualitative and quantitatively the results of seismic analysis for extended in-plane structures or stiff structures located in regions with rapidly varying soil profiles. The Portuguese code (RSA 1984) and the Eurocode 8 state that the assumption of a unique seismic input, in all foundations of a structure, is not acceptable when its horizontal dimension is large. The justification is clearly identified by the measures in seismograph array and in the published studies (Hao 1989, Harichandran 1991, Laouami 2001).

This analysis requires a characterization of seismic load through a displacement history for each support. The definition of those correlated series is influenced by different factors such as the incidence angle of propagation, three-dimensional combination of seismic effects, multiplicity of soil foundations, etc.

The time series reflects the overlap of different waves arriving at each station, each one with its own path and history of refractions, reflections, velocities, etc. The displacements used for seismic analysis must contain all this information. One way to define the displacement histories is based on records from a real earthquake, because they have the influence of the site ground conditions.

However, those records must be changed in order to obtain time records complying with the requirements of codes of practice. In regions of low to moderate seismic risk, this may be difficult to achieve, given the scarcely of registered events. The lack of such databases suggests the generation and use of artificial records. This is an attractive alternative, given the smaller amount of data required to achieve a code-compliant modeling of the action and will thus be the option for this study.

The definition of the earthquake load is frequently achieved through the use of response spectra, given the simplicity and information contents of such functions. The artificial generation shall therefore be based on response spectrum.

The Portuguese code of practice (RSA 1984) states that the seismic acceleration may be defined as a Gaussian ergodic stationary stochastic process, characterized by the auto--power spectral density function (APSDF), $G_{\ddot{u},\ddot{u}}$. That function includes information on the rate of variation, with frequency, of the mean value of the squares of the records. Therefore, one may estimate a record from a stochastic process by mean of a sum of harmoniums whose amplitudes are related to that function. This was the approach used in the study and will be detailed in this paper.

In a rigid-base analysis, the load may be represented as a uniform and synchronous ground acceleration, which is easily converted into equivalent forces to be imposed to the structure. However, when the effect of spatial variability of earthquake ground motion is considered, it is preferable to use displacement records instead of accelerations, because that is how the earthquake affects the structure.

The spatial variability of earthquake ground motion can change qualitative and quantitatively the results of seismic analysis for extended in-plane structures or stiff structures located in regions with rapidly varying soil profiles. The Portuguese code (RSA 1984) and the Eurocode 8 state that the assumption of a unique seismic input, in all foundations of a structure, is not acceptable when its horizontal dimension is large. The justification is clearly identified by the measures in seismograph array and in the published studies (Hao 1989, Harichandran 1991, Laouami 2001).

In order to simulate the spatial variability in time domain it is necessary to develop a method for the displacements correlation. That method is presented in Chapter 3.

2   Synthesis of displacements

The procedure for the generation of artificial displacement for the supports may be described as follows: given the response spectrum, (i) estimate the acceleration APSD function; (ii) generate a record from a stationary process; (iii) multiply the record by an intensity function in order to convert it into a non-stationary event; (iv) correct the baseline and (v) evaluate the displacements by integration (Ferreira 2004).

2.1   Estimate the APSD Function

In most codes of practice, the earthquake load is described in terms of response spectra. These functions will thus be used as the starting point.

The estimative of APSDF, $\hat{G}_{\ddot{u},\ddot{u}}$ is achieved by an iterative process, as suggested by Clough (1993) or Barbat (1994). An initial shape for the function, such as constant, half wave, Kanai-Tajimi APSDF (Clough 1993), is assumed and artificial accelerograms are then generated. The median response spectrum, *RS*, is evaluated thereafter and the estimative of APSDF is corrected by comparison of the response spectrum evaluated with that prescribed in the code. This correction is made for each frequency band

$$\hat{G}_{\ddot{u},\ddot{u},\,j+1}\left(f_i,\Delta f\right) = \hat{G}_{\ddot{u},\ddot{u},\,j}\left(f_i,\Delta f\right)\cdot\left(\frac{RS_{code}\left(f_i\right)}{RS_{calculate}\left(f_i\right)}\right)^2 \qquad (1)$$

Though the convergence of this method may not be ensured, because it assumes that each frequency band affects that band only, it was nevertheless achieved in 6 iterations, in the studied examples.

In order to measure the accuracy of the estimative, the Eurocode 8 (1994) states that the calculated response spectrum must be at least 90% of the response spectrum given in code.

2.2   Generate a stationary record

Among the various models presented in literature for the generation of a stationary record, in discrete form in time domain $t_n = n \cdot \Delta t$, from a stochastic process, the one proposed by Kiureghian (1996) was considered for this study. In it, the acceleration, *a* (*t*) is given by a sum of harmoniums a_i (*t*), each one with its own frequency, $f_i = i \cdot \Delta t$, amplitude, A_i (*t*) and phase angle, ϕ_i. The estimation of each harmonium is made numerically through expression (2).

$$a_i\left(t_n\right) = A_i\left(t_n\right)\cdot\cos\left(2\pi\left(i\cdot\Delta f\right)\cdot\left(n\cdot\Delta t\right)+\phi_i\right) \qquad (2)$$

Kiureghian (1996) suggests that the amplitude of each harmonium may be considered as a function of time without affecting the process stationariness, because of the random nature of the seismic phenomena.

The model proposed for such variation assumes that each cycle has randomly evaluated amplitude. These amplitudes are inter-related with low correlation coefficient and through a zero-mean normal distribution with standard deviation given by

$$\sigma_{A_i} = \sqrt{G_{\ddot{u},\ddot{u}}\left(f_i\right)\cdot\Delta f} \qquad (3)$$

This characteristic allows synthesizing long time-records for bigger frequency band then those generated with harmoniums with constant amplitude. The records given by this last option are periodic in $1/\Delta f$.

Each harmonium, a_i (t_n), which is defined with *N* timesteps in interval $[0, (N-1)\cdot\Delta t]$, must comply with the characteristics of the process it belongs to, namely the mean value and the mean value
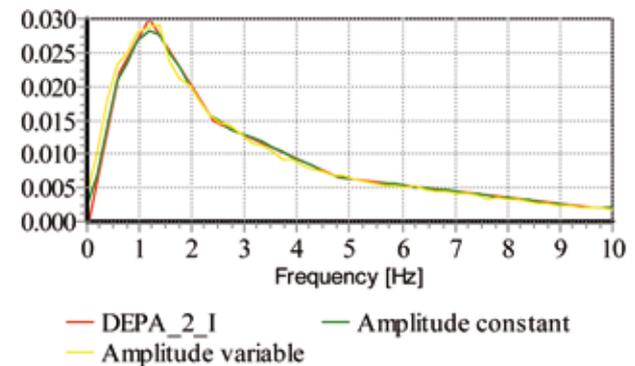
Figure 1   Comparison between APSDF given by RSA code and evaluated from artificial records with variable and constant amplitudes

of the square. These conditions are not usually fulfilled by (2) and a correction is necessary. In order to achieve that, each harmonium is thus redefined as stated by equation (4), with constants and given by expressions (5) and (6). The corrected harmonium is called $\ddot{u}_i(t)$, which can be summed to get the acceleration record, $\ddot{u}(t)$

$$\ddot{u}_i(t_n) = c_0 + c_1 \cdot a_i(t_n) \tag{4}$$

$$c_1 = \sqrt{\frac{N \cdot G_{\ddot{u},\ddot{u}}(f_i) \cdot \Delta f}{\sqrt{\sum_{n=0}^{N-1}(a_i(t_n))^2 - \frac{1}{N+1}\left(\sum_{n=0}^{N-1}a_i(t_n)\right)^2}}} \tag{5}$$

$$c_0 = -\frac{1}{N} \cdot c_1 \cdot \sum_{n=0}^{N+1} a_i(t_n) \tag{6}$$

The use of time-dependant amplitudes represents an additional problem in the simulation of a stationary stochastic process. In order to investigate this issue, the acceleration APSD function corresponding to the type-1 soil, A zone and type-2 earthquake, according to RSA (1984), was used to generate five artificial records of one such process for both constant and variable amplitudes. The estimation of auto-power spectral density function, $\hat{G}_{\ddot{u},\ddot{u}}$, for each one, was computed by the method presented by Bendat (1986).

$$\hat{G}_{\ddot{u},\ddot{u}}(f_i) = \frac{2}{N \cdot \Delta t \cdot n_d} \cdot \sum_{k=1}^{n_d} \left| FFT(\ddot{u}(f_i)) \right|^2 \tag{7}$$

The accuracy of the estimative depends of the number of segments, that the original record is subdivided, n_d, and the precision of the FFT calculation. Bendat (1986) states the error, ε, may be estimate by

$$\varepsilon = \sqrt{1/n_d} \tag{8}$$

The FFT algorithm used in this study is based on Cooley-Tukey procedure (NR 1992), so it is convenient to have sub-records with $N = 2^p$ values. The five records have durations of $512.100.0.01s = 512s$ and are made up with harmoniums with frequencies at intervals of 0.01Hz. The Figure 1 shows the results.

The resulting APSDF is compared through a method leading to a value between 0 (absolutely distinct) and 100% (exactly the same). The comparison method leads to values of 93.0% and 97.9% for variable and constant amplitudes, respectively. In spite of the reduction of nearly 5% for the variable amplitude case, this type of generation has the advantage of resulting in a non-periodic function. In order to avoid this problem with the constant amplitude approach, low starting frequency and frequency step must be used. Therefore, the variable amplitude approach was used, because of its superior simulation of the actual phenomena (Kiureghian 1996). Besides this test, the record is still submitted to a test of stationarity (reverse arrangements test) which confirm the synthesis process, see Bendat (1986).

The phase angle may assume any random value between 0 and 2π, according to a uniform probability distribution law. These values must be statistically independent altogether and with the amplitude values.

2.3 Intensity function

The procedure described up to this point generates a record of a stationary stochastic process. It differs from the earthquake ground motion acceleration in that this is non-stationary by nature. This characteristic may be achieved by using a time-varying APSDF or, as an alternative, by affecting the stationary process with an intensity function $I(t)$. The latter approach results in an artificial acceleration with a stationary stage between two time instants $[t_1, t_2]$, preceded by an increasing period $[0, t_1]$ and followed by a decay period $[t_2, t_N]$. Both the RSA (1984) and the DNA (1994) define the duration of the stationary period as a function of the earthquake type. One commonly used modulation function is presented in (9), (Clough 1993 and Barbat 1994):

$$I(t) = \begin{cases} (t/t_1)^2 & 0 \le t < t_1 \\ 1 & t_1 \le t \le t_2 \\ e^{-c\cdot(t-t_2)} & t > t_2 \end{cases} \tag{9}$$

2.4 Correction of the acceleration baseline

The correction previously mentioned must be complemented with another one, intended to remove the long-period components from the acceleration record. This is the so-called baseline correction. Two alternative approaches to undertake this are referred in literature and they are both described in the following sections.

2.4.1 Barbat baseline correction

This method is used in the programmes GENONDAS, Barbat (1994), and in one supplied by NISEE, Berkeley (2004). According to this procedure, the corrected acceleration $\ddot{u}_c(t)$ is computed through the expression (10), and the constants used are evaluated by expressions (12) and (13).

$$\ddot{u}_c(t_n) = \ddot{u}(t_n) - d_0 - d_1 \cdot tt_N - d_2 \cdot tt_N^2 \tag{10}$$

$$tt_i = (i - 1/2) \cdot \Delta t \qquad i = 1, 2 \ldots, N \tag{11}$$

$$\beta_1 = \frac{\Delta t^2}{tt_N^3} \cdot \left[\sum_{i=1}^{N} tt_i \cdot \left(\sum_{j=1}^{i} \ddot{u}(tt_j) \right) \right]$$

$$\beta_2 = \frac{\Delta t^2}{tt_N^4} \cdot \left[\sum_{i=1}^{N} tt_i^2 \cdot \left(\sum_{j=1}^{i} \ddot{u}(tt_j) \right) \right] \tag{12}$$

$$\beta_3 = \frac{\Delta t^2}{tt_N^5} \cdot \left[\sum_{i=1}^{N} tt_i^3 \cdot \left(\sum_{j=1}^{i} \ddot{u}(tt_j) \right) \right]$$

$$\begin{aligned} d_0 &= 300 \cdot \beta_1 - 900 \cdot \beta_2 + 630 \cdot \beta_3 \\ d_1 &= (-1800 \cdot \beta_1 + 5760 \cdot \beta_2 - 4200 \cdot \beta_3)/tt_N \\ d_2 &= (1890 \cdot \beta_1 - 6300 \cdot \beta_2 + 4725 \cdot \beta_3)/tt_N^2 \end{aligned} \tag{13}$$

2.4.2 Trifunac baseline correction

Trifunac (1970) observes that the previous approach does not provide good results when the records have long time duration. His study refers that those harmoniums with a period shorter than 16s are correctly evaluated, while significant distortions may occur in the case of longer periods. As a solution for this problem, Trifunac suggests a filtering procedure in order to remove the harmoniums with the lowest frequencies. On the contrary of the Barbat procedure, this method does not depend on the record duration and allows an iterative enhancement of the result. The algorithm proposes that the baseline is evaluated by using the contributions of harmoniums with period longer than 16s. The correct value of the acceleration is computed by subtracting the baseline values from those of the original record. This procedure is done in frequency domain using the FFT and the IFFT algorithms (NR 1992).

2.5 Evaluation of the displacement records

A displacement record may be computed by integration of the acceleration function. For the present case, it is assumed that the initial values of both the displacement and the velocities are null. If the mean values of the acceleration and velocity functions are non null, the displacement will oscillate around a quadratic function. As this is physically meaningless, the records of acceleration and velocity are changed so that its mean values are null.

A linear variation of the acceleration within the time step was considered for the time integration. The expressions (14) and (15) provide the velocity, $\dot{u}$, and the displacement, u, in each time instant.

$$\dot{u}(t_n + \tau) = \dot{u}(t_n) + \Delta t / 2 \cdot \left[\ddot{u}(t_{n+1}) + \ddot{u}(t_n) \right] \tag{14}$$

$$u(t_{n+1}) = u(t_n) + \dot{u}(t_n) \cdot \Delta t + \Delta t^2 / 6 \cdot \left[\ddot{u}(t_{n+1}) + 2 \cdot \ddot{u}(t_n) \right] \tag{15}$$

3 Model for spatial variability

Most studies on the spatial variability of earthquake ground motion define this characteristic in the frequency domain by mean of a coherence function, see Nazmy (1992), Zanardo (2002) and Lupoi (2005). One objective of this paper is to present a procedure to synthesize displacements records in the time domain and, for that purpose, a model for the SVEGM must be defined.

The characterization of the parameters of the proposed model may be achieved with the geotechnical information of the site conditions or with the information included in the codes of practice, when the influence of different soil conditions in the load is foreseen.

3.1 Basic assumptions

The size of the structure is neglectable in comparison with the overall dimension of the earthquake event and, as a consequence, no energy loss of the seismic waves takes place while they travel between the supports. It is also assumed that the combination of the various types of waves does not change significantly in different positions, which is to say that the displacements in the supports are generated by the same wave composition throughout the structure. Therefore, the differences arising between those displacements shall result from the finite speed of the wave. Consequently the APSDF shall thus be the same and the incoherence effect is not considered in this study. The soil was considered to be made up of a number of elastic and homogeneous layers.

3.2 Model description

In accordance to the simplifying assumptions previously referred, the implemented model considers that the seismic waves travel through the bedrock and reach the foundation level like shear waves propagating vertically. This behaviour is also presented by Dowrick (1977).

An important issue in the characterization of the spatial variability of the seismic load is the wave speed. For the sake of simplicity, it is usually accepted that all the waves components travel with the same velocity and follow a straight path, regardless its frequency. The wave speed is thus a mean value and, once defined its value, the time delay for the arrival of the wave to the various supports may be easily evaluated.

The direction of the propagation is another important issue to take into consideration. As the wave delay depends on the distance between supports, the propagation in the longitudinal direction will produce more changes with respect to the rigid-base approach. For this reason, this was the assumed direction for the wave travel.

The site amplification effect also plays a fundamental role in the seismic response. In order to define it, the shear building transfer model was considered, in which the bedrock vibration propagates as shear waves to the overlaying layers with resonant frequencies (16) that amplify the seismic record. Those depend on the layer thickness h, and the speed of the shear waves v_s.

$$f_k = \frac{k \cdot v_s}{4 \cdot h} \qquad k = 1, 2, 3, \ldots \tag{16}$$

The model used in this study is illustrated in Figure 2. Each support can have a different definition for the soil properties. Considering foundation soils with distinct shear speeds, different frequency ranges will be filtered as a consequence.
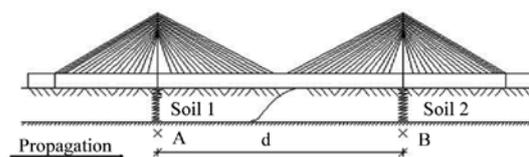

Figure 2 Spatial variation model

3.3 Calculation procedure

With respect to Figure 2, it is assumed that the seismic waves travel from A to B. The first step consists in generating a record for a stationary stochastic process, which will be taken as the basis for

the artificial seismic load record. This generation is based on the APSDF characterizing the load in the bedrock. This random record is assigned to support A and represents its "pseudo-acceleration". Once this record is available for the support A, it is a straightforward task to estimate the records for the remaining supports, based on the assumed wave speed and the distance between A and each of these points. These records must then be filtered, in order to simulate the propagation throughout the foundation soil layer.

Safak (1995) describes the transfer model used in this step. That function, $H(f)$, is defined by 3 parameters r, τ and Q, which depend on the speed of the shear waves, the unit mass of the rock (v_r, ρ_r) and soil (v_s, ρ_s) layers, the soil layer height h and the damping quality factor, Q. The function is as shown in (17) and the parameters are evaluated through expressions (18) and (19).

$$H(f) = \frac{(1-r) \cdot e^{-j2\pi f \tau \cdot (1 - j/(2Q))}}{1 + r \cdot e^{-j4\pi f \tau \cdot (1 - j/(2Q))}} \tag{17}$$

$$r = \frac{\rho_r \cdot v_r - \rho_s \cdot v_s}{\rho_r \cdot v_r + \rho_s \cdot v_s} \tag{18}$$

$$\tau = \frac{h}{v_s} \tag{19}$$

The transfer function $H(f)$ relates the displacements in the bedrock with those at the foundation level. If these are characterized by APSD functions, which in turn are estimated based on response spectra, then the parameters r and τ of the function $H(f)$ may be estimated by expression (20).

$$APSDF_{soil}(f) = |H(f)|^2 \cdot APSDF_{bedrock}(f) \tag{20}$$

The corresponding time filter of transfer model is also presented in Safak (1995). The representation in time domain of the local site amplification clearly identifies the two components of this effect, the amplification at the resonant frequencies defined by (16) and the delay due to the difference between the shear velocities (21).

$$delay = \tau_2 - \tau_1 = h / v_{s,2} - h / v_{s,1} \tag{21}$$

The quality factor must be adopted according to literature recommendations. See Mari (1996) or Bowles (1996) as examples.

In short, each different soil condition has a different transfer function. Once the acceleration record is available at the foundation level, it is necessary to multiply that record by the intensity function, to perform the baseline correction and, finally, to integrate in order to obtain the displacements at the supports.

4   Simplified analysis of a cable-stayed bridge

A Finite Element (FE) model based in the International Guadiana Bridge (Portugal-Spain border in Algarve) was used for the numerical test. This is a symmetric cable-stayed bridge with a total span of 666 m and a main (central) span of 324 m. In this type of bridges, the dynamic behaviour is strongly affected by the deck-to-pylon and deck-to-abutment connections. In the developed FE model, no longitudinal connection was considered in any of those locations. With such structural system, the seismic response is basically controlled by the first vibration mode, with a period of about 7.87s. This value is evaluated based in a 3D Finite Element model of the bridge.

ENV 1998-2 (1994) allows using the analysis of a single degree of freedom system (SDOF) to evaluate the structural response, on the condition that the axial deformation of the deck is neglectable. Therefore, this was the adopted approach in this study, although this is a special bridge in the Eurocode nomenclature.

4.1   Characterization of the SDOF system

The spatial variability of the earthquake ground motion leads to differences between the displacements records for each tower of the bridge. Simulating this actual behaviour requires that the SDOF system is excited by two different records. The model for the SDOF system used in this study is shown in Figure 3.

This model evaluates the damping forces through the absolute velocity in order to simplify the expressions (23) to (26). However, the results had been compared with another model with damping forces proportional to relative velocities and it was found that the error is small, because these forces are small in the context of this analysis.

The damper position illustrates how the damping forces are evaluated. These are computed based in the velocity, the damping ratio, ζ, the stiffness, k, and mass, m, of the oscillator (22).

$$c = 2 \cdot \zeta \cdot \sqrt{k \cdot m} \tag{22}$$

The complete description of the SDOF system requires the values of the damping ratio and of the period. Those values are of, respectively, 2% and 7.87s.
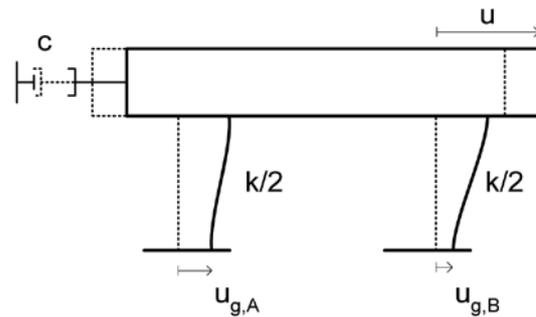

Figure 3   Schematic represent of the SDOF model

4.2   Equilibrium equation for SDOF system

From the inspection of Figure 3, the equilibrium equation may easily be written as

$$m \cdot \ddot{u} + c \cdot \dot{u} + k/2 \cdot \left(u - u_{g.A}\right) + k/2 \cdot \left(u - u_{g.B}\right) = 0 \qquad (23)$$

The inertia and damping forces are proportional to the velocity, acceleration and the mass of the oscillator. On the examples shown here, the differences between displacements in distinct supports will result only from the time delay of the waves. Therefore, one may write equation (23) in the following form

$$\ddot{u} + 2 \cdot \zeta \cdot \omega_n \cdot \dot{u} + \omega_n^2 \cdot u = \omega_n^2/2 \cdot u_{g.A} + \omega_n^2/2 \cdot u_{g.B} \qquad (24)$$

$$\Delta u_g\left(t\right) = u_{g.A}\left(t\right) - u_{g.B}\left(t\right) = u_{g.A}\left(t\right) - u_{g.A}\left(t - d/V_{app}\right) \qquad (25)$$

$$\ddot{u} = 2 \cdot \zeta \cdot \omega_n \cdot \dot{u} + \omega_n^2 \cdot u = \omega_n^2 \cdot u_{g.A} - \omega_n^2/2 \cdot \Delta u_g \qquad (26)$$

d is the distance between the two supports, A and B. V_{app} is the apparent average velocity of seismic. The time delay is achieve dividing the distance by the velocity.

Equation (26) highlights the two contributions for the overall effect of the seismic load, one related to the ordinary rigid-base analysis and the other resulting from the spatial variability of the earthquake ground motion, mostly caused by the finite propagation speed. The pseudo-static displacement that is imposed to the structure is given by

$$u_{ps} = 1/2 \cdot \left(u_{g.A} + u_{g.B}\right) = u_{g.A} - \Delta u_g/2 \qquad (27)$$

4.3 Displacements computed by using the SDOF approach

The method of Newmark with average acceleration was used for the numerical solution of equation (24).

The seismic load used in these examples refers to a type-2 earthquake (strong earthquake with large focus distance) as defined in RSA (1984). The values in Table I are the average of maxima of the SDOF system responses subjected to a 1000 different input time-series. In order to evaluate the wave-passage effect it was chosen different velocities to establish a tendency for the variation. The lower velocity has that propose.

Table 1   Average maxima of 1000 different displacement evaluated by (23)

		Barbat	Trifunac
	Input	0.2371m	0.1327m
Velocity	∞	0.4532m	0.3843m
	3000m/s	0.4529m	0.3840m
	2000m/s	0.4526m	0.3836m
	1000m/s	0.4506m	0.3814m
	200m/s	0.3900m	0.3168m

From Table I one may conclude that the smaller the wave velocity, the smaller the maximum displacement, in spite of the larger wave delays. However, the difference is very small (0.18%) for a propagation velocity of 2000m/s and Trifunac baseline correction. This conclusion is clearly identified in equation (26), because the second contribution has not the same sign of the first contribution.

Table I also shows the results obtained with the two previously mentioned methods for the baseline correction. The values computed with the Trifunac approach are smaller because the contributions of the components with long period are removed, which are the most significant in a displacement record. One must also remark that the dynamic amplification of the Trifunac baseline corrected displacements is larger (2.89 versus 1.91 – for a 2000m/s for the propagation speed). This is due to the nonzero initial velocity introduced by this method. Figure 4 represents the displacement records resulting from the same acceleration record and evaluated by the two baseline correction methods presented.
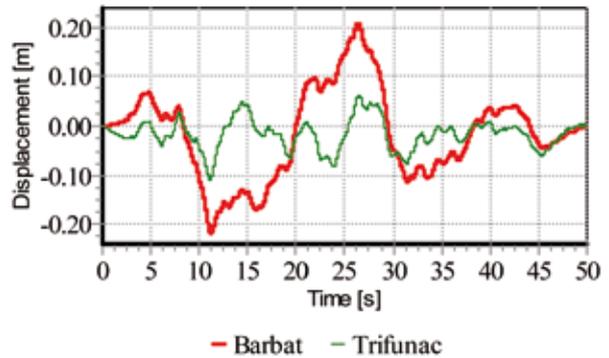
Figure 4   Displacements corrected by two different methods

5 Conclusions

In order to undertake the seismic analysis of a structure to be erected in a region with scarce data from actual earthquakes, a possible alternative is the use of artificial records, generated in order to match the requirements of codes of practice. In this paper, expressions for the generation of such imposed displacements are presented and discussed. A particular attention is paid to the procedure to account for the spatial variability of the earthquake ground motion. This issue is of some relevance in long structures such as suspended or cable-stayed bridges, or when the spatial variation in the soil characteristics is important.

The displacements synthesis process may be evaluated carefully due to the baseline correction and integration processes. The Trifunac baseline correction method generally leads to better results, for a long total time record, than those provided by the Barbat Baseline correction, but introduces a nonzero initial velocity that amplifies the structure response. In the integration scheme it is very important to achieve or impose a null average velocity in order to get a null final displacement.

The analysis of the SDOF system discussed here, characterised by a

long period oscillation, shows that the wave-passage effect has not a significant effect in seismic response. This conclusion can not be simply extrapolated to a bridge analysis, in which the investigation of the influence of all vibration modes is required. This paper shows a procedure to synthesize displacements for that analysis.

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