Effect of Seismic wave scattering on the Response of Dam-Reservoir-Foundation Systems

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SUMMARY:
Traditionally the foundation in a dam is modeled by a sub-structuring approach for the purpose of seismic performance analysis. The main disadvantage of sub-structuring approach is that it cannot be used for solving nonlinear dynamic problems. Therefore, in that case seismic response analysis must be carried out in time domain. Among the different earthquake input mechanisms the deconvolved earthquake input model is a preferred method as it removes the seismic scattering effects due to artificial boundaries of the semi-infinite foundation domain. Deconvolution is a mathematical process which allows the adjustment of the amplitude and frequency contents of an earthquake ground motion applied at the base of the foundation to achieve the desired output at the dam-foundation interface. The existing procedures of deconvolution are not effective for all types of earthquake records including high-frequency and low-frequency ground motions. An improved procedure has for deconvolution been proposed here.

Keywords: Seismic wave scattering, deconvolution, concrete gravity dam, dam-foundation interaction

1. INTRODUCTION

The number and size of hydroelectric dams have increased greatly across the Canada since 1910 (CDA, 2007). While concrete gravity dams have shown satisfactory performance during the earthquake, there are a few dams around the world that have been shaken by strong earthquake (USSD, 2000). Shih-Kang Dam in Taiwan suffered complete loss of the reservoir during Chi-Chi earthquake in September 1999 (JSCE, 1999). Hsifengkiang dam in China and Koyna dam in India also suffered considerable damage in 1962 and 1967 earthquakes, respectively (Bolt and Cloud, 1974; Hall, 1988). Therefore, monitoring and assessment of dam performance is very important for ensuring dam safety (Garabedian *et al.*, 2006). It is necessary that the evaluation of the gravity dams should be done realistically by incorporating the effects of interaction among dam, foundation and reservoir. Chakrabarti and Chopra (1974), and Fenves and Chopra (1985) studied the dam-foundation interaction effect in the frequency domain using visco-elastic half space solutions to model the foundation. In many cases, the analytical models based on frequency domain analysis are insufficient as it cannot be used to model nonlinear and non-homogenous geometrical and material properties of the dam or foundation. In such cases, analysis must be done in time domain.

Clough *et al.* (1985), and Léger and Boughoufalah (1989) studied a set of various models to simulate different earthquake input mechanisms. The models used in those studies include rigid base, massless foundation, deconvolved earthquake records, and free-field input. In case of the deconvolved input model (Reimer, 1973) a deconvolution analysis is carried out to determine the foundation base acceleration for a specified free-field acceleration history at the base of the dam. Deconvolution is a mathematical process which allows the adjustment of the amplitude and frequency content of an earthquake ground motion to achieve the desired output. Computer program SHAKE developed by Schnabel *et al.* (1972) for deconvolution has been used in many previous studies (Léger and Boughoufalah, 1989; Luk *et al.*, 2005; Polam *et al.*, 2007). However, the deconvolution process using the procedure used in SHAKE is very cumbersome as the response obtained through this program is very sensitive to the values of the controlling parameters such as the shear modulus, and the equivalent viscous damping ratio in case of flexible foundations (Léger and Boughoufalah, 1989). The purpose of this paper is to develop a modified procedure for deconvolution of ground motions which is applicable for all types of ground motions. Luk *et al.* (2005) and Polam *et al.* (2007) recommended different constraint models to represent foundation models. In the present study, a similar approach has been undertaken and implemented using a commercial software ABAQUS (Abaqus, 2011).

2. SEISMIC WAVE SCATTERING IN DAM A FOUNDATION SYSTEM

To evaluate the response of a dam during a seismic event, the earthquake acceleration is applied at the base of the foundation and it propagates vertically by an elastic wave propagation mechanism until it reaches the top of the foundation. The size of the foundation in a numerical model is finite as compared to the semi-infinite foundation in the physical model. The seismic waves are reflected from the boundaries of the numerical model. This seismic wave scattering due to artificial boundaries in the numerical model results in altering the frequency content and amplitude of a ground motion as the wave propagates through the deformable foundation rock. A numerical model to evaluate the seismic performance must account for such wave scattering effect to obtain a reliable response. The use of transmitting boundaries, or deconvolved ground motion records are recommended for that purpose.

3. DECONVOLVED EARTHQUAKE INPUT MODEL

In this method, the analysis is carried out in two steps. First a deconvolution analysis is performed to determine the acceleration time history that can be applied to the base of the foundation to reproduce the specified free-field acceleration time history at the base of a dam (Figure 3.1). The calibrated base acceleration history is then applied to the base of the foundation to perform the seismic analysis. Deconvolution analysis can be performed using a mathematical process as described in Figure 3.2 (Reimer, 1973) which is explained below.

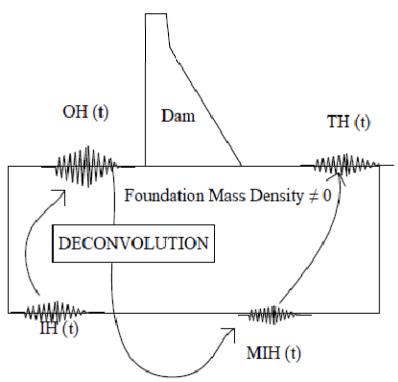

Figure 3.1 Representation of deconvolution procedure

Deconvolution analysis allows the adjustment of the amplitude and frequency contents of an earthquake ground motion applied at the base of the foundation to achieve the desired output ground acceleration at the dam-foundation interface. A step-by-step iterative procedure for deconvolution is shown in Figure 3.2. Initially, the ground acceleration applied at the base of the foundation is assumed to be the same as the free-field ground acceleration. The acceleration time history at the top surface (i.e., dam-foundation interface) is then estimated by solving the wave propagation problem of the dam-foundation system using the finite element analysis technique. This estimated or reproduced ground acceleration at a reference point on the dam-foundation interface is then compared to the original free-field ground acceleration after transforming both the signals into the frequency domain using Fourier analysis. Fast Fourier transform (FFT) and inverse Fast Fourier transform (IFFT) algorithms developed by (Cooley & Tukey, 1965) for transforming as time domain signal to a frequency domain signal, and back, respectibely. FFT of a time series yields complex Fourier amplitude values for a set of discrete frequencies. The complex Fourier amplitudes are then converted into absolute values to obtain the Fourier amplitude spectrum. On the other hand, IFFT of a set of complex Fourier amplitudes for a set of discrete frequencies yields a time domain signal. As mentioned earlier, the free-field acceleration or any arbitrary signal is initially applied at the base of the foundation and by solving the wave propagation problem, the acceleration signal at a selected point at the dam-foundation interface is obtained. The synthesized and free-field acceleration signal at the dam-foundation interface is then compared in the frequency domain, and a correction factor for each frequency is computed using the ratio of the Fourier amplitudes of the synthesized and free-field ground acceleration signals in a given iteration.

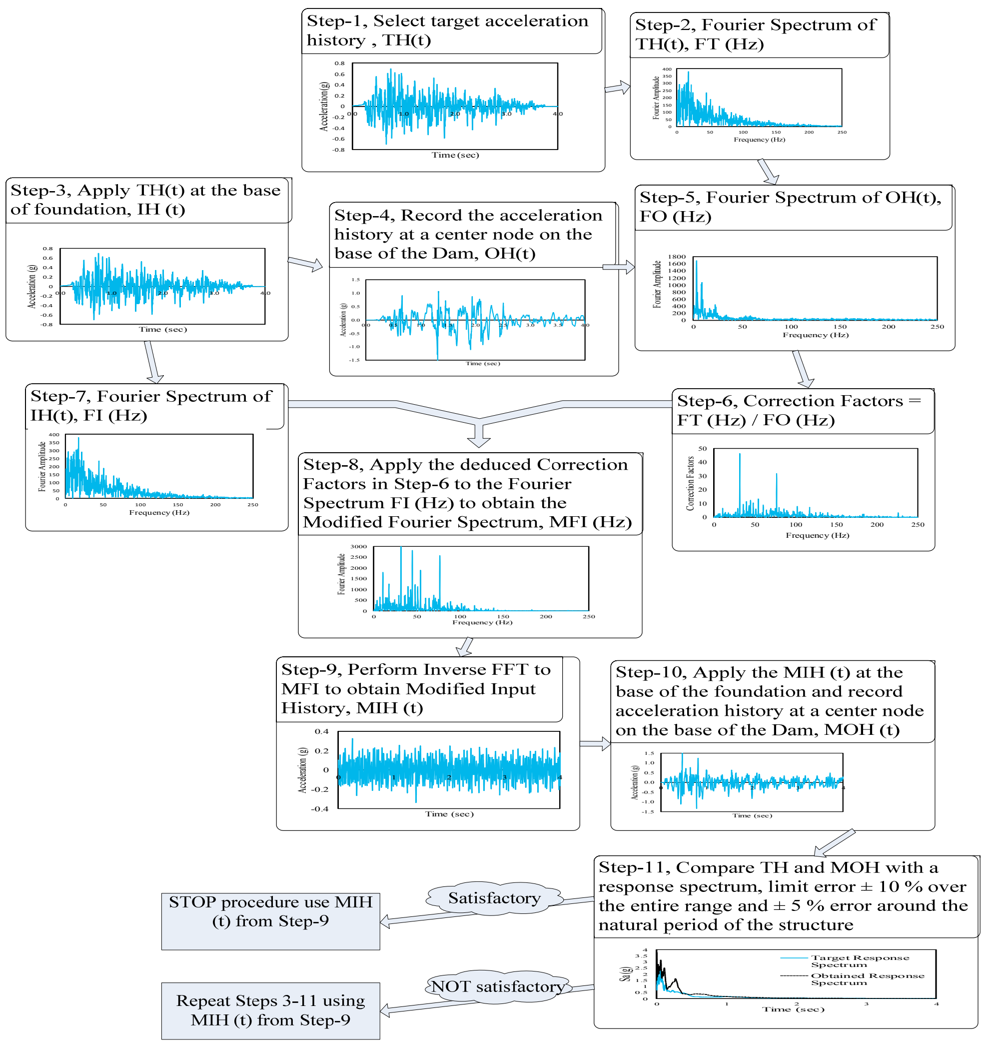

Figure 3.2 Existing deconvolution procedure

The acceleration signal applied at the base of the foundation is modified using the correction factor for each frequency. The modified acceleration history is then transformed back into time domain acceleration signal by employing IFFT and the analysis of the wave propagation analysis for the foundation system is repeated with the modified ground acceleration applied at the base of the foundation. The procedure is repeated until the original free-field ground motion at the dam-foundation interface closely matches the reproduced ground motion record generated by using the modified ground motion applied at the base of the foundation. The resulting ground motion at the foundation-base would be called the deconvolved ground motion that should be used in the dynamic analysis of the dam-foundation system.

4. MODIFIED DECONVOLUTION PROCEDURE

The existing procedure for deconvolution as discussed in the previous section does not produce appropriate results for high frequency ground motion records as will be shown later. However, it works quite well for the low frequency ground motion records in some cases. To overcome such limitation, a modified procedure has been proposed in this section. Figure 4.1 shows a flow chart for the modified deconvolution procedure.

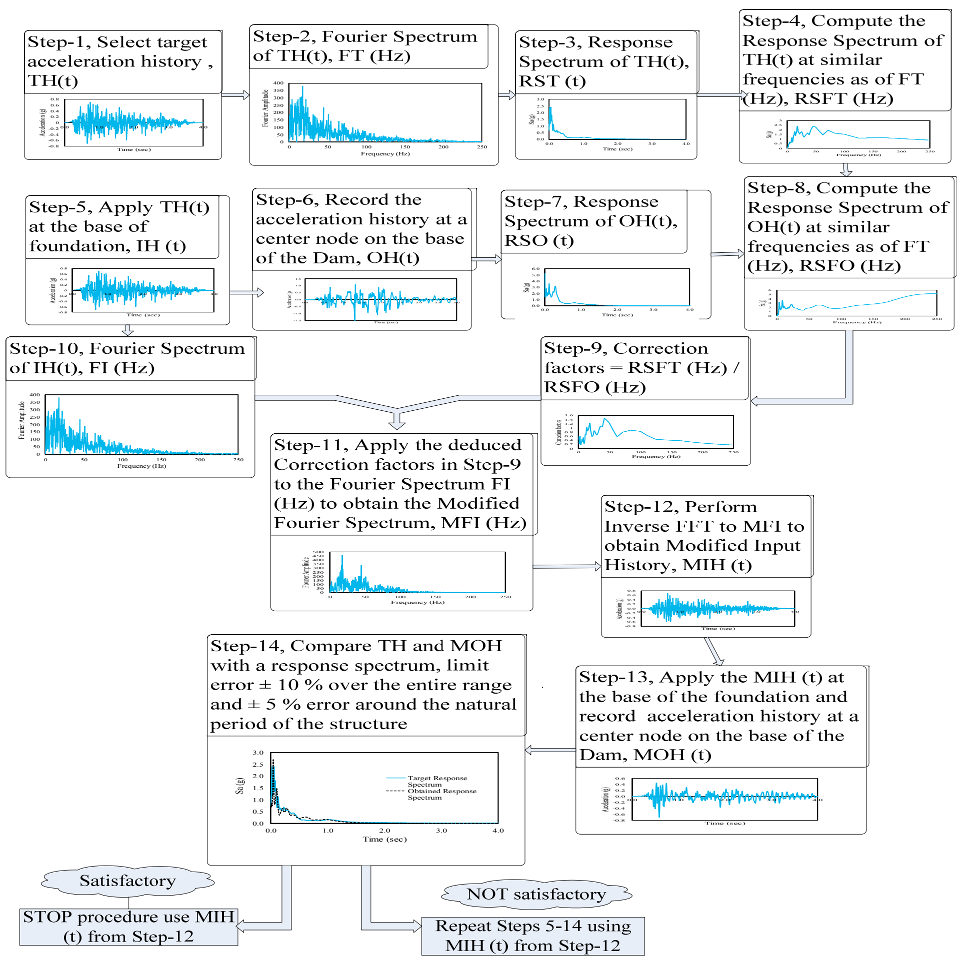

Figure 4.1 Modified deconvolution procedure

Similar to the existing procedure, here the reproduced acceleration history at the top of foundation is compared to the free-field acceleration, both converted to frequency domain using Fourier analysis. However, the correction factors to adjust the deconvolved signal are determined differently. Instead of adjusting the Fourier amplitudes at different frequencies, the spectral density at different frequency are adjusted. The response spectra of the reproduced acceleration history and the input ground motion (i.e. original free-field accelearion) are computed for the discrete set of frequencies. The correction factors (CF) are calculated for each frequency by the ratio of the target response spectrum amplitude $TS_a(j)$ and the response spectrum amplitude $RS_a(j)$ of the reproduced acceleration history (Eq. 4.1).

$$CF(j) = TS_a(j)/RS_a(j) \tag{4.1}$$

This correction factor is then applied to the frequency-domain acceleration signal applied at the base of the foundation. The complex Fourier coefficients (real *a(j)*, and the imaginary *b(j)*) coefficients of the acceleration at the foundation base are modified using Eqs. 4.2 and 4.3.

$$a(j)_{modified} = a(j)*CF(j) \tag{4.2}$$

$$b(j)_{modified} = b(j)*CF(j) \tag{4.3}$$

The modified acceleration signal is then transformed back to a time domain by using IFFT. The analysis of the dam-foundation system is carried with the modified time history of ground acceleration applied at the base of the foundation. The procedure is iteratively repeated until the reproduced ground motion at the base of the dam closely matches the original free-field ground motion. The response spectrum of the reproduced ground motion at the top of the foundation should match the target response spectrum. To determine the closeness of the response spectrum of reproduced ground motion to the free-field ground motion, the coefficient of determination (R^2) as defined in the texts in Statistics has been utilized. A value of 1 for R^2 represents a perfect match of the two data series which are represented here by the response spectra of the original and reproduced ground accelerations. The proposed modified deconvolution procedure is found to work very well for both high and low frequency ground motions.

5. FINITE ELEMENT MODEL AND CONSTRAINTS

Two geometrically different monoliths of concrete gravity dams have been considered here to study the seismic wave scattering in dam foundation systems. Figure 5.1 shows the two geometric configurations, G-1 and G-2 which are considered here.

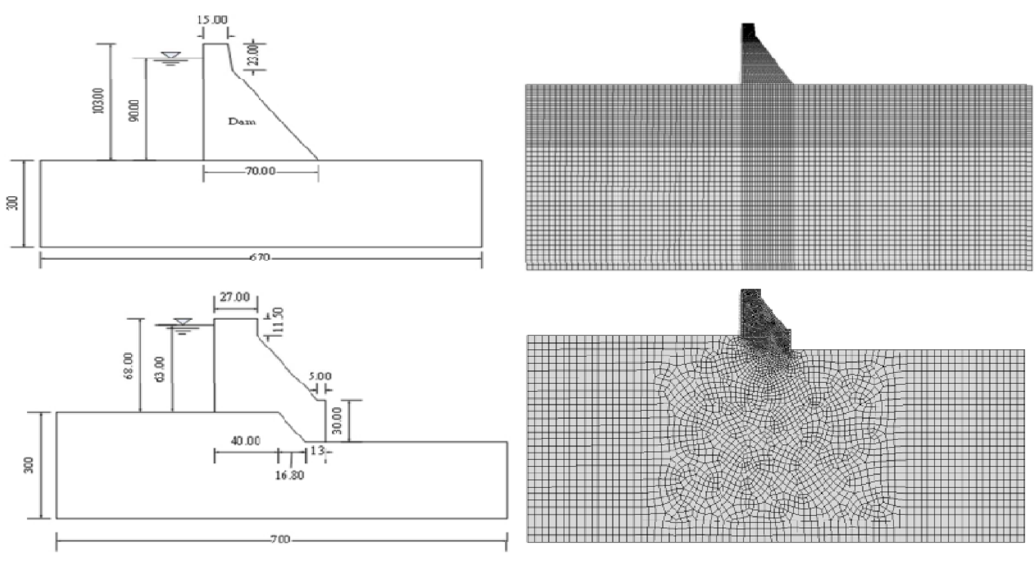

Figure 5.1 Dam-foundation system a) Geomtry G-1 and a) Geomtry G-2

In Figure 5.1, G-1 represents a geometrical configuration which is commonly used for dams. However, G-2 has an irregular foundation. These kinds of irregular foundations are popular in large surface toe hydroelectric projects located on good quality foundation rock (Liang *et al.*, 2011; Gupta *et al.*, 2009). The assumed material properties are summarized in Table 5.1. Five percent material damping is considered in the analysis with Rayleigh damping assumptions. The hydrodynamic interaction is modeled by added mass model considering incompressible water. The dam and foundation system is modeled using four noded bilinear plain strain finite elements. To perform the deconvolution procedure, the soil must act as a one dimensional soil column. To simulate the one dimensional soil column behavior, a set of constraints needs to be applied on the boundaries.

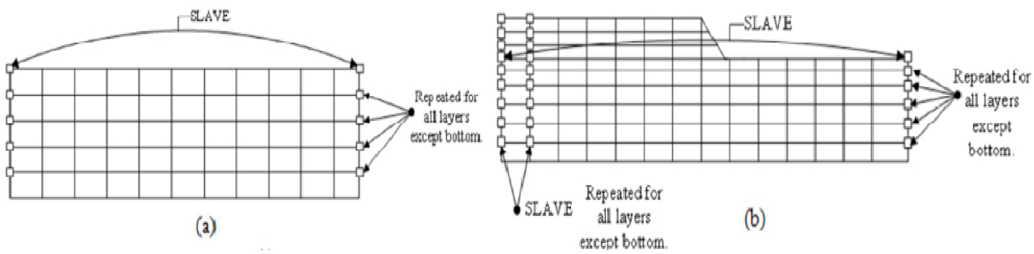

Figure 5.2 Representation of constraints

Figure 5.2 shows the representation of constraints which allow the shear deformations in foundation to simulate the propagation of waves but they do not allow the foundation to deform in bending mode. This includes constraining the boundary nodes of two sides at the same level to have the same displacement. In case the other side cannot be constrained in the same manner as in case of inclined slope, two adjustment nodes are constrained on the same side such that they act as shear column. Foundation size should be sufficiently large to accommodate the local displacements near the dam. Based on the study by Bayraktar *et al.* (2009), the size of the foundation is assumed to be three times the height of the dam or 3H, which is almost equal to 300 m on each side of the dam in this case.

Table 5.1: Material properties

Material	Concrete	Rock
Elastic Modulus (MPa)	3.45×10^4	2.76×10^4
Poisson's ratio	0.2	0.33
Unit weight (kN/m3)	23.5	25.9

6. SELECTION OF SEISMIC GROUND MOTIONS

Two different ensembles of ground motions containing high frequency and low frequency contents have been considered here. The ensembles contain both simulated and actual ground motion records at rock site condition.. The simulated records have been chosen based on those developed in Tremblay and Atkinson (2001), while the ground records of past earthquakes have been obtained from the PEER database (PEER, 2012). The first ensemble of high frequency ground motion includes the following records: (i) simulated record for Eastern Canada having magnitude M6 and distance 30 km; (ii) simulated record for Eastern Canada having magnitude M7 and distance 70 km; (ii) San Fernando 1971 earthquake record. These ground motion records are referred here as M #1, M #2 and M #3, respectively. The horizontal and vertical components of the ground motions are denoted here by H and V, respectively (Figure 6.1 a & b). The second ensemble of low frequency ground motions includes the following records: (i) Friuli 1976 earthquake record, (ii) Livermore 1980 earthquake record, and (iii) simulated record for Western Canada having a magnitude M6.5 and distance 30 km. These ground motion records are referred here as V #1, V #2 and V #3, respectively (Figure 6.1 c & d). The horizontal components of the high frequency ground motions have been scaled according to an expected level of seismic hazard (with 2% probability of exceedance in 50 years) that corresponds to Montreal (Eastern Canada). On the other hand, the horizontal components of the low frequency ground motions have been scaled according to an expected level of seismic hazard that corresponds to Vancouver (Western Canada). The vertical components of all ground motions have been scaled to the two third of the respective horizontal components. Figure 6.1 shows the scaled response spectra of the ground motions. The time periods of the dam-foundation systems for geometry G-1 and G-2 are found to be 0.628 s and 0.67 s, respectively.

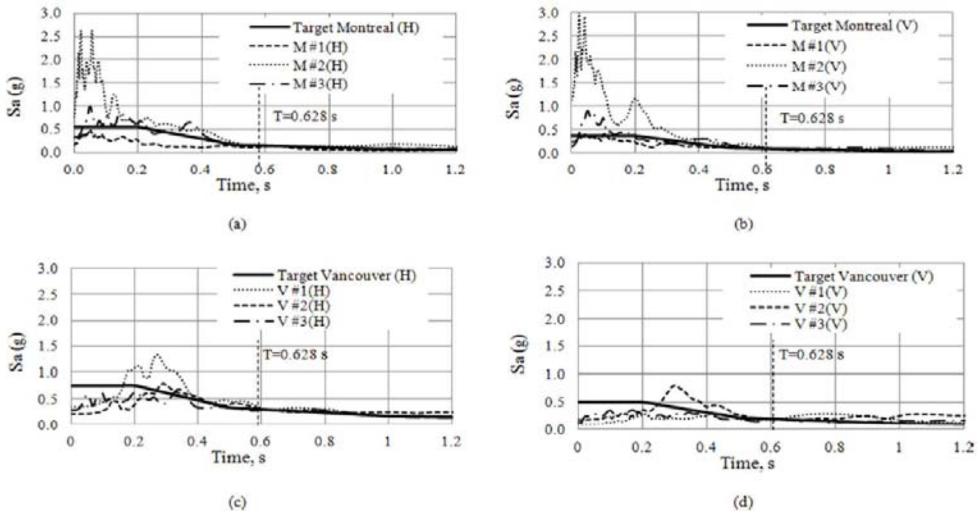

Figure 6.1 Response Spectra for the ground motion records: (a) Montreal – horizontal components, (b) Montreal – vertical components, (c) Vancouver-horizontal components, and (d) Vancouver-vertical components

7. PERFORMANCE OF THE MODIFIED DECONVOLUTION PROCEDURE

Figures 7.1, 7.2 and 7.3 present the results of the different deconvolved ground acceleration time history by modified (MDP) and existing deconvolution procedures (EDP) for dam-foundation system, G-1.

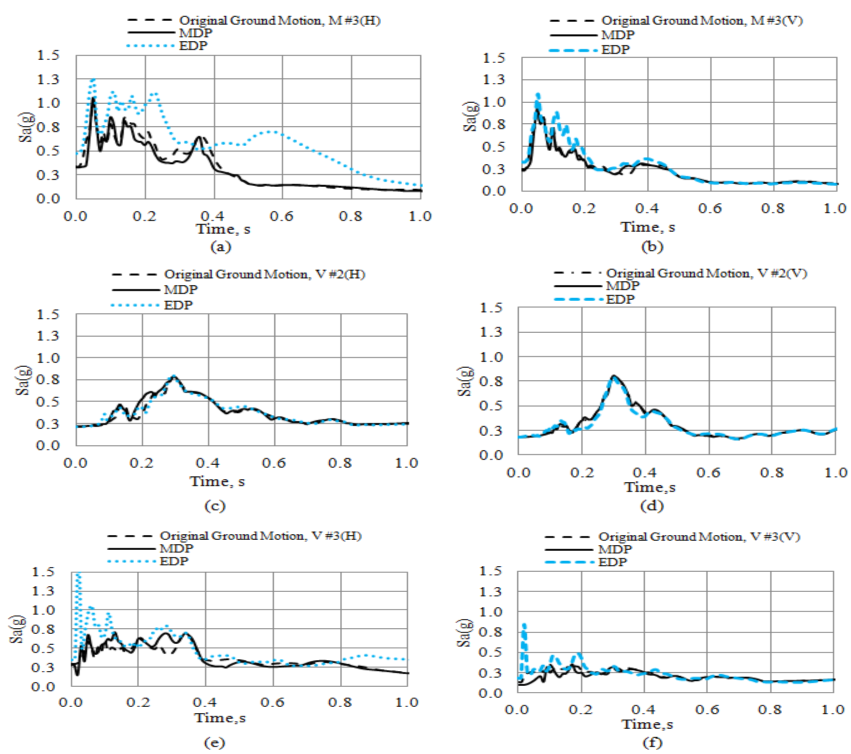

Figure 7.1 Response spectra of the original and deconvolved ground motions for G-1 in Montreal: a) M #3(H); b) M #3(V); c) V #2(H); d) V #2(V); e) V #3(H); f) V #3(V).

It is observed from the results that the MDP works very well for both high frequency and low frequency ground motions. However, EDP produces acceptable results only in the cases of some low-frequency ground motions such as, V#1 and V #2, but does not work other cases such as, V #3. To demonstrate the effectiveness of MDP as compared to EDP, the results of deconvolution have discussed for the following earthquake records: M #3 representing a high-frequency record; and V #2 and V #3 representing low-frequency records. Figure 7.1 shows the response spectra of the original record along with those generated from the deconvolved records. As indicated by Fig. 7.1 (a & b), for M #3, the MDP spectra match very closely with the spectra of the free field (original) ground motion for both horizontal and vertical components, while the EDP spectra do not match very well. Fig. 7.1 (c & d) show the comparison of the original spectra for V #2 with the MDP and EDP spectra for the horizontal and vertical components. In this case, both MDP and EDP spectra are observed to be close to the spectra of the original ground motion. Fig. 7.1 (e & f) show the comparison of the original spectra for V #3 with the MDP and EDP spectra for the horizontal and vertical components. In this case, MDP spectra match very closely with the spectra of the free field (original) ground motion, while the EDP spectra do not match very well. This is similar to what has been observed in the case of M #3 record.

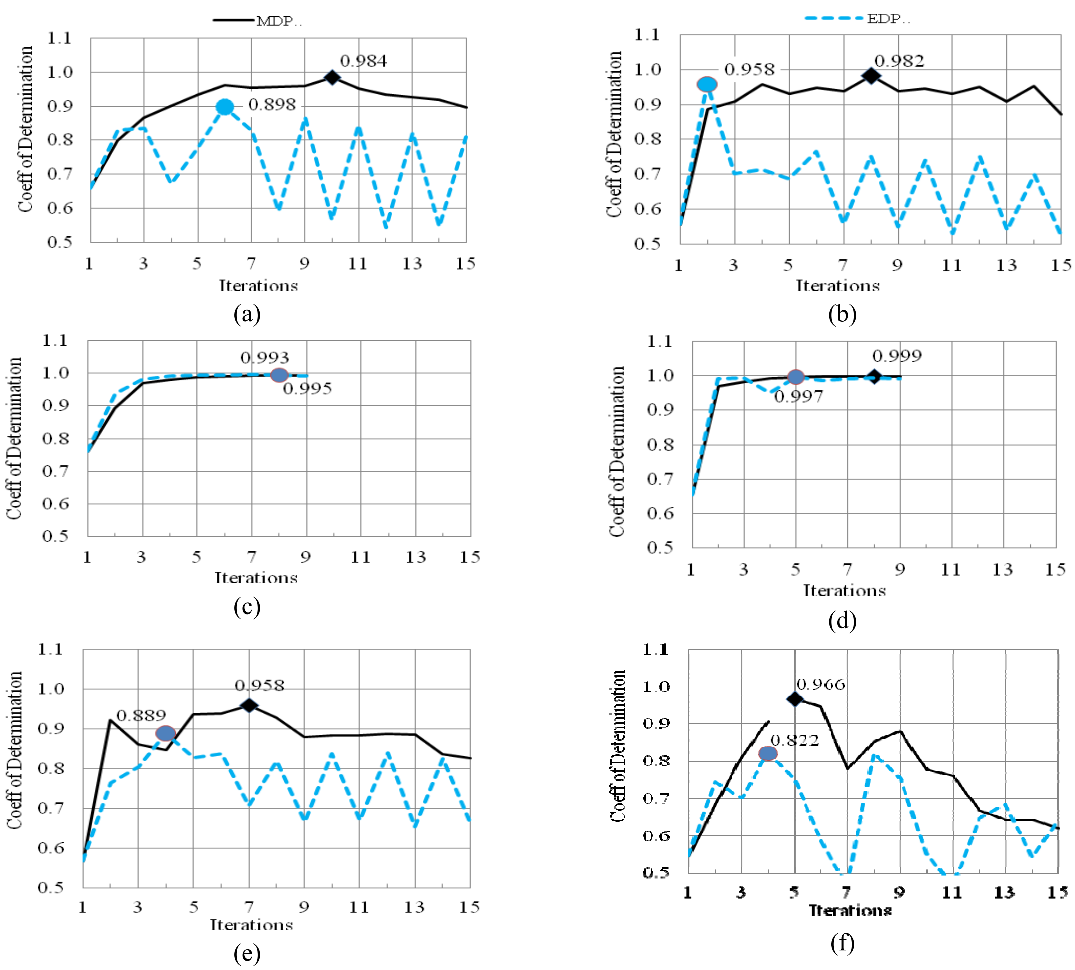

Figure 7.2 Coefficient of Determination (R2) for deconvolved ground motions for Geometry G1: a) M #3(H); (b) M #3(V); (c) V #2(H); (d) V #2(V); (e) V #3(H); and (f) V #3(V)

Figure 7.2 shows the values of the coefficient of Determination (R^2) for different iterations for MDP and EDP in the case of M #3 ground motion. The maximum values R^2 achieved for M #3(H) by MDP and EDP are 0.984 and 0.898, respectively (Fig. 7.2a), while that for M #3(V) are 0.982 and 0.958, respectively (Fig. 7.2b). It is observed that for MDP the value of R^2 approaches relatively more smoothly and converges well in both cases, while the R^2 values for EDP fluctuate at different iterations and the convergence is poor. The maximum values of R^2 achieved for V #2(H) by MDP and EDP are found to be 0.993 and 0.995 (Fig. 7.2c), respectively, while that for V #2(V) are 0.999 and 0.997 (Fig. 7.2d), respectively. In the case of V #2 ground motion, the results obtained by both MDP and EDP are satisfactory, and the R^2 values converge very smoothly in both cases. However, in case of V #3 ground motion, the results obtained from EDP are not satisfactory. The maximum values of R^2 achieved for for V #3(H) by MDP and EDP are 0.958 and 0.887, respectively (Fig. 7.2e), while that for V #3(V) are 0.966 and 0.822, respectively (Fig. 7.2f). From the above results, it can be concluded that the performance of EDP in the cases of low-frequency ground motions is better than its performance in the cases of high frequency ground motions. However, in some cases, even for low-frequency ground motions such as, V#3, the performance of EDP is not acceptable. MDP shows a satisfactory performance for all types of ground motions.

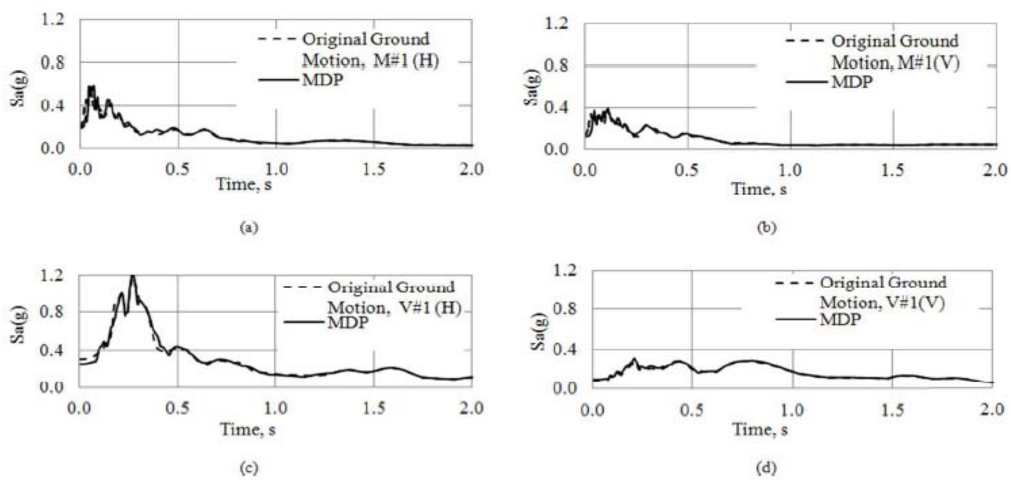

Figure 7.3 Deconvolved ground motions with MDP for dam-foundation, G2: a) M#1(H); b) M#1(V); a) V#1(H); b) V#1(V)

Figure 7.3 presents the response spectra of the deconvolved ground motions for M #1 and V #1 for the dam-foundation system, G-2 with MDP and the original ground motions. As the quality of the deconvolution process affects the response of a dam-foundation system, the performance of the deconvolution procedure used in the study is very important.

8. CONCLUSIONS

The study presents a modified deconvolution procedure for the deconvolution of input ground motions for the use in the seismic response analysis of dam-foundation systems. The modified deconvolution procedure performs well for both high frequency and low frequency ground motions. It is important here to note that while only two dimensional models are considered here, the modified deconvolution procedure proposed in study is expected to be more effective for three dimensional dam-foundation models. Further study is required in that direction.

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