Originally published as:

Ponzo F. C., Ditommaso R., Auletta G., Mossucca A. (2010).
A Fast Method for Structural Health Monitoring of Italian Strategic Reinforced Concrete Buildings. Bulletin of Earthquake Engineering. DOI: 10.1007/s10518-010-9194-6. Volume 8, Number 6, Pages 1421-1434.

A Fast Method for Structural Health Monitoring of Italian Reinforced Concrete Strategic Buildings

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ABSTRACT

The growing number of demand for a widespread of health monitoring for strategic buildings in seismic areas has emphasized the need to realize in-depth scientific studies, in order to verify the feasibility of economic and fast methods to detect anomalous vibrations, to execute post earthquake warning and monitoring, damage assessment and first damage scenarios. Generally, an effective system for structural health monitoring requires an appropriate number of sensors, suitably located in the structures, and complex elaborations of big amounts of data. The simplified method presented in this paper is based on a statistical approach that uses the most significant data recorded on the top floor of the building, with the purpose of extracting information on the maximum inter-story drift, used as damage indicator. The parameters considered in the method are (i) maximum top acceleration, (ii) the first modal frequency variations and (iii) the equivalent structural viscous damping variation. A big amount of experimental data relevant to several tests carried out on scaled R/C models and numerical non linear dynamic analyses have been used to verify the feasibility of this approach.

Keywords: Dynamic Identification, Structural Health Monitoring, Damage Evaluation.

INTRODUCTION

The assessment of an increasing number of aged structures and infrastructures requires a huge effort, especially if the purpose is to provide a faithful evaluation of seismic risk. The current practice of periodic visual inspections, for the safety evaluation appears more and more inadequate.
A specific task of the Italian research RELUIS project, funded by the Department of Italian Civil Protection (DPC), deals with devising and implementing a fast procedure to get useful information about the damage evolution in a large number of strategic buildings during and after seismic events, using the records of just few sensors located on the structure.

The feasibility and the cost optimization are the most important goals for the simplified monitoring system in order to favour the a widespread use of such systems.

During the past two decades, a significant amount of researches have been carried out in the field of Non-destructive Damage Evaluation (NDE) methods using the changes in the dynamic response of a structure (Picozzi *et al.*, 2010a and 2010b; Ditommaso *et al.*, 2010). The NDE methods can be classified into four levels (Stubbs et al., 2000), according to the specificity of the information provided by a given approach (Rytter, 1993). (i) Level I methods, i.e. those methods that only identify if damage has occurred. (ii) Level II methods, i.e. those methods that identify if damage has occurred and simultaneously determine the location of damage. (iii) Level III methods, i.e. those methods that identify if damage has occurred, determine the location of damage as well as estimate the severity of damage. (iv) Level IV methods, i.e. those methods that identify if damage has occurred, determine the location of damage, estimate the severity of damage and evaluate the impact of damage on the structure. Each level of damage identification described above requires a gradual increasing amount of data and more complex algorithms. Consequently, their set-up and effectiveness often require increasing costs, with higher error probability.

The objective of this paper is to set-up a simplified practicable method of Level I, based on a statistical approach, to continuously check the safety and reliability of strategic buildings. The method detects the evolution of the damage by comparing the dynamic response of the building before, during and after the earthquake. The response is evaluated considering the following approach: i) structural dynamic parameters (maximum top acceleration, first modal frequency variations and equivalent structural viscous damping variation) are evaluated just from top floor records; ii) a non linear correlation between all dynamic parameters and the maximum inter-storey drift, considered as damage index, is defined. The data of two experimental applications and many numerical simulations are provided to verify the effectiveness of the method.

METHODOLOGY

Level I methods are generally based on the variation of the main vibration frequencies or the variation of equivalent viscous damping. Such methods are often convenient because they are simple, robust and employ a reduced number of sensors installed within the structure, although they can lead to wrong evaluations. In fact, the variation of vibration frequencies is not necessarily representative of the damage evolution (Doebling et al., 1998; Marijan Herak and Davorka Herak, 2010; Todorovska et al., 2006), but it can also be determined by the variation of the temperature or the configuration of masses and stiffness, especially for deformable structures as reinforced concrete or steel frames.

The proposed simplified fast method for structural health monitoring of strategic buildings starts from a limited number of records acquired on top floor (velocity or acceleration) and overcomes some limitations of the Level I methods. Indeed, the method considers some parameters evaluated by the recorded signals: (i) Maximum Absolute Top Acceleration (MATA); (ii) variations of the fundamental frequency (iii) variation of the equivalent viscous damping, and provides a combination of these parameters to estimate the maximum inter-storey drift by means of nonlinear correlation analysis. Also the maximum top displacement was adopted, but using accelerometric station there were some problems for the double integration. In fact, in order to avoid integration problems a band-pass filter is a major importance. Indeed, it is possible to evaluate both velocity and displacement by mean a filtered signal. In this case the problem is related to the automatization of the procedure: considering the non-linear behaviour of a structure, and the related frequency variation, it is very hard to set up an automatic selection of limits of band-pass filter. For these reasons, and in order to simplify the procedure the acceleration signal were selected.

All signals are filtered with band-pass filter 0.1-20 Hz. Finally, the maximum value of the estimated drift along the building height can be assumed as correction factor of the Damage Index determined by classic first levels methods.

The MATA represents the first instrumental parameter considered in this method. It can be evaluated directly by the filtered signal (acceleration), or by a derivation of filtered signal in velocity, recorded on the top floor of the building. An appropriate arrangement of recording sensors on the structure permits to reconstruct all displacement and rotation components of the floor.

The other two instrumental parameters considered in the method are i) the percent variations (Δf_1) between the fundamental frequency of the building before the seismic event f_{init} and the minimum one f_{min}, corresponding to the maximum non linear behaviour of the building and ii) the percent variations (Δf_2) between initial and final frequency (f_{fin}), as given by equations 1 and 2.

$$\Delta f_1 = (f_{init} - f_{\min}) / f_{init} \qquad (1)$$

$$\Delta f_2 = (f_{init} - f_{fin}) / f_{init} \qquad (2)$$

All frequencies can be evaluated by a Short Time Fourier Transform (STFT) (Gabor, 1946) applied on the whole signal, as shown in Figure 1, that refers to an earthquake recorded on a real building

(Laurenzano et al., 2010). This particular analysis allows to describe the variation in the time of the main frequencies of the structure.

The last instrumental parameter considered in the method is the variation of equivalent structural viscous damping $\Delta\xi$ related to the first structural mode. Information about damping can enrich the quality and quantity of the knowledge on the global damage, particularly if the damping is associated to the other parameters above described. For non stationary signal the damping can be estimated using the *only output* non-parametric technique elaborated by Mucciarelli and Gallipoli (2007). It measures the viscous equivalent damping of the signal recorded using a semi-probabilistic approach. The damping is estimated by logarithmic decrement method on minimum of three consecutive decreasing peaks separated by the same period T, as shown in Figure 2a. The period T is assumed considering a fixed tolerance ε as function of T.

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The routine output supplies a vector $\boldsymbol{\xi} = (\xi_1, \xi_2, \ldots, \xi_n)$, where ξ_i is the value obtained by two consecutive decreasing peaks. Then, the lognormal distribution of the vector $\boldsymbol{\xi}$ is evaluated in order to get a median value of distribution for the whole signal, as shown in Figure 2b. A check on the effectiveness of the result is, then, carried out by comparing the fit curve between theoretical and empirical cumulative distribution function, through the Kolmogorov-Smirnov test (Piccolo, 1998). The variation $\Delta\xi$ between the equivalent viscous damping evaluated on the signal before and after the earthquake is assumed as reference parameter in the method.

REGRESSION ANALYSIS FOR STRUCTURAL CONSTANTS EVALUATION

In order to verify the existence of a relationship between the selected parameters and the maximum inter-storey drift, several correlation analyses were done considering: (i) the outcomes of experimental shaking table tests carried out by the University of Basilicata within the research projects TREMA and POP (Dolce et al. 2005, 2006); (ii) numerical non linear dynamic analysis carried out considering different natural accelerograms compatible with Italian OPCM 3431, selected in the European Strong Motion Database, and different structure configurations. The flow-chart shown in Figure 3 explains the proposed approach.

Considering the output of each experimental or numerical test, a matrix containing the maximum inter-storey drift along the height of the model (dependent parameter) and the four instrumental parameters above defined (independent parameters) is built (eq. 3) in order to perform the correlation and regression analyses. Each row refers to a single test or numerical analysis.

$$\begin{vmatrix} \Delta_{\max 1} & a_{\max 1} & \Delta f_{11} & \Delta f_{21} & \Delta\xi_1 \\ \Delta_{\max 2} & a_{\max 2} & \Delta f_{12} & \Delta f_{22} & \Delta\xi_2 \\ .... & .... & .... & .... & .... \\ \Delta_{\max n} & a_{\max n} & \Delta f_{1n} & \Delta f_{2n} & \Delta\xi_n \end{vmatrix} \qquad (3)$$

In case of structural damage the global non linear behaviour of the building makes the linear approach ineffective and a non linear multivariable regression becomes necessary to describe the relation between the considered parameters and the maximum inter-story drift.

For this study a non linear relation for each structure characterized by a second-degree polynomial, with eight coefficients, can be sufficient to describe efficiently the correlation between drift and other parameters, as shown in the following paragraphs.

$$\begin{pmatrix} \Delta_1 \\ \Delta_2 \\ . \\ . \\ . \\ \Delta_n \end{pmatrix} = \begin{pmatrix} a^2{}_{\max 1} & a_{\max 1} & \Delta f^2{}_{11} & \Delta f_{11} & \Delta f^2{}_{21} & \Delta f_{21} & \Delta\xi^2{}_1 & \Delta\xi_1 \\ a^2{}_{\max 2} & a_{\max 2} & \Delta f^2{}_{12} & \Delta f_{12} & \Delta f^2{}_{22} & \Delta f_{22} & \Delta\xi^2{}_2 & \Delta\xi_2 \\ . & . & . & . & . & . & . & . \\ . & . & . & . & . & . & . & . \\ . & . & . & . & . & . & . & . \\ a^2{}_{\max n} & a_{\max n} & \Delta f^2{}_{1n} & \Delta f_{1n} & \Delta f^2{}_{2n} & \Delta f_{2n} & \Delta\xi^2{}_n & \Delta\xi_n \end{pmatrix} \cdot \begin{pmatrix} c_1 \\ c_2 \\ c_3 \\ c_4 \\ c_5 \\ c_6 \\ c_7 \\ c_8 \end{pmatrix} \qquad (4)$$

The constants c_1, c_2…, c_8 are the regression factors determined by solving the equation system above for each single structure, starting from experimental or numerical data. They constitute the characteristic factors of the structure obtained considering all set of results.

Finally, the analytical expression to evaluate the maximum expected drift along the height of the building can be written as:

$$\begin{aligned} \Delta_{an} &= c_1 \cdot a_{\max}^2 + c_2 \cdot a_{\max} + c_3 \cdot \Delta f_1{}^2 + \\ &+ c_4 \cdot \Delta f_1 + c_5 \cdot \Delta f_2{}^2 + c_6 \cdot \Delta f_2 + c_7 \cdot \Delta\xi^2 + c_8 \cdot \Delta\xi \end{aligned} \qquad (5)$$

The contribution of the weight of each single instrumental parameter on the maximum analytical inter-storey drift is estimated by the following expression:

$$W_i = \frac{|F_i|}{\sum_i |F_i|} \qquad (6)$$

In case of existing structure, if there are not available data recorded on the structure during some previous earthquake, it is possible to build a nonlinear numerical model for the considered structure. Taking into account the soil characteristics it is possible to study in a statistical way the dynamic response of the structure using as input several accelerograms statistically possible for the considered site. From these analyses it is possible to evaluate the (c_1,…,c_8) coefficients using the proposed procedure.

EXPERIMENTAL TESTS

The results of two extensive experimental programs of dynamic tests carried out on two similar 1/4 scaled 3-D R/C models, derived from a prototype building designed for gravity loads only, have been considered. The first 4-storey model was tested at the Structural Laboratory of the University of Basilicata in Potenza, within the POP project (Dolce et al., 2005), through mono-axial shaking table test (Figure 4).

The second model was a 3-storey specimen (Figure 4), with infill panels, tested on the 4x4m 6-DOF shaking table facility of ENEA Casaccia (Rome) within the TREMA project (Dolce et al., 2006).

The correlation analyses presented here were carried out considering, for the POP model, the structural response to both natural and artificial mono-directional earthquakes, while natural bi-directional earthquake response was considered for the TREMA model.

The natural seismic input used in both projects was the Colfiorito record of the Italian 1997 Umbria-Marche earthquake, while the artificial seismic input considered was derived from the response spectrum provided by Eurocode 8 for soil type B. Both acceleration profiles were scaled down in time by a $Sc = \frac{1}{\sqrt{\text{Scale Factor}}}$ factor, for consistency with the scale of the model.

In the POP Project, for the tests in the fixed-base configuration here considered, the effective peak acceleration of the table was progressively increased from 0.05g up to 0.35g. All floors displacement were measured through Temposonic digital transducers, fixed to an external steel reference frame. The floor accelerations were acquired through a system of horizontal servo accelerometers. In the tests for the TREMA project, the effective peak acceleration of the table was progressively increased from 0.04g up to 0.23g. In this case, the top floor the accelerations were acquired through 3 horizontal servo accelerometers and their measures were recomposed in the centre of mass. Figures 5 and 6 show the maximum experimental inter-storey drifts obtained for both POP and TREMA experimental tests vs each considered instrumental parameters. The maximum inter-storey drift Δ_{max} is nonlinearly dependent from the maximum top acceleration A_{max}, as shown by Figures 5 and 6, and with just a low dispersion factor of the experimental points. Indeed, other parameters Δf_1, Δf_2 and $\Delta\xi$ (Figs. 5 and 6) show a non linear dependence on the maximum inter-storey drift Δ_{max}, more evident for $\Delta\xi$ and Δf_2, and a dispersion degree considerably bigger than A_{max}. It is interesting to notice that both frequency variations Δf_1 and damping variation $\Delta\xi$ assume values different from zero for Δ_{max} greater than about 0.4-0.5%. This value represents the threshold of structural damaging.

The analytical inter-story drift evaluated by the nonlinear regression analysis, as summarized by equation 5, was compared to the experimental results and shown in Figures 9 and 10. Then, for each specimen was determined a single vector of characteristic parameters ($c_1,\ldots,c_8$) starting from all experimental data relative to examined structures. Figure 9 shows the experimental vs analytical maximum inter-storey drifts for POP model. In this case the correlation factor between the two parameters ($R^2 \approx 1$) demonstrates the good estimation of maximum drift furnished by the proposed formulation as a function of all considered parameters. In Figure 7 the weight of each single instrumental parameter used by non linear regression to estimate the inter-storey drift are reported. Maximum top acceleration A_{max} provides the most important contribution to Δ_{max}, especially for low intensity earthquake. In this case and for the POP model the frequency variation Δf_1 accounts for 20÷25%. For increasing seismic intensity, and therefore for damage cumulating, the contribution of two previously parameters is getting down, to advantage of Δf_2 and $\Delta\xi$.

A different trend was observed for TREMA out-comes, probably because of the presence of masonry infill and of damage suffered by model in previous tests. In any case the analytical formulation was able to evaluate correctly the maximum inter-storey drift.

NUMERICAL SIMULATIONS

A parametric numerical simulation study was planned in order to understand the influence of the seismic input and of the most significant geometrical, mechanical and regularity parameters on the weight of the each instrumental parameters for the damage detection. A huge amount of numerical data are now being processed. The first results here analyzed have been obtained by considering a non linear numerical model of a R/C regular building, five floor, four frames along the longitudinal direction (X) and three frames in the transverse direction (Y), having a rectangular plan, 15x12m.

The considered structure has been designed following the criteria of the Italian seismic code (OPCM 3431/2005) for high ductility class (CDA), high seismic intensity (PGA 0.35g) and for soil type A. The height of each storey is 3m, for a total height of the building equal to 15m. A software based on non linear finite element (SAP2000 non-linear) has been used to model the 3-D structure (*Wilson 2002*). Beams and columns have been modelled with frame elements, assuming 20 MPa cylindrical strength of concrete and 430 MPa yield strength of steel. In order to simulate a structural non linear behaviour during a strong ground motion, link elements and plastic hinges have been added at the end of beam and column elements respectively. Link elements have a Takeda hysteretic behaviour, while plastic hinges have an axial load-dependent one. A total of 168 non-linear bi-directional dynamic analyses have been carried out using, for each categories of seismic zones (from ag=0.05g up to 0.35g) and for soil type A and B,C,E (S=1, 1.25), seven different pairs of natural accelerograms, selected in the European Strong Motion Database, compatibles with the Italian OPCM 3431/05. For soft soil, D type, seven artificial accelerograms pairs, having the spectrum compatible with Italian seismic code, have been considered. Figure 11 shows the response spectra of each group of seismic inputs.

Figure 12 shows the maximum numerical inter-storey drifts *vs* all the considered parameters. These parameters were evaluated by the displacement and acceleration histories of the master joint at each floor and for all considered earthquakes. The maximum inter-storey drift Δ_{max}, depicted in Fig. 12, is still linear dependent from maximum top acceleration A_{max}, where the linear correlation coefficients between maximum drift and each parameter are provided. Moreover in the diagram of Figure 12 an evident low dispersion of the results can be observed. Other parameters Δf_1, Δf_2 and $\Delta\xi$ (Figs. 12b, c and d) show a dispersion of results bigger than A_{max}, while frequency variation Δf_2 and damping variation $\Delta\xi$ confirm the same trend observed for experimental correlation analyses, in particular, that a drift of about 0.5% represents a damage threshold for R/C buildings.

For numerical model a regression analysis, considering the results of all numerical simulations, has been made. Therefore, a single vector ($c_1,\ldots,c_8$) of constitutive constant of the considered structure has been carried out. In Figure 13 the maximum numerical and analytical inter-storey drifts are compared. As for experimental tests, also in this case the analytical procedure (eq. 5) provides a good estimation of the drift values, as testified by a correlation vector $R^2\approx 1$. Using this procedure the vector ($c_1,\ldots,c_8$) is independent from input and soil type.

In Figure 14 the weight of each single parameter on analytical evaluation of the maximum drift with equation 5 are reported. The non linear analyses confirmed the trend observed with experimental results. For low values of PGA and for all soils the maximum top acceleration A_{max} provides the more consisting contribution on Δ_{max}, while for PGA increasing the frequency variation Δf_2 and $\Delta\xi$ become more important. Δf_2 maintain the same importance.

In order to validate the proposed method, the results obtained from our procedure were compared with the results obtained using another damage index: Park and Ang damage index (Park and Ang, 1985 and 1987).

As shown in Figure 15 there is a good correlation between Park Index and analytical maximum drift obtained with the proposed procedure.

DISCUSSION AND CONCLUSION

The damage assessment methodology proposed in this paper can detects and quantify the damage suffered by a R/C building using only few instrumental parameters measured on the top of the monitored structure (Level I methods). The damage is expressed in terms of maximum inter-story drift (damage indicator) evaluated through a non linear regression analysis, starting from parameters above defined.

Several outcomes obtained from experimental and numerical tests allowed to calibrate and verify the methodological approach.

This approach can result useful especially for high excitation level and, considering his low cost, it should favour the possibility to increase the number of monitoring strategic structures.

The first analyses of the results show that, for regular buildings, the only case investigated up to now, the maximum value of the top floor acceleration is the most strictly related parameter to the maximum inter-storey drift along the building height. The other parameters become important for high PGA values, when a strong non linear behaviour is activated.

In case of infilled frames it is reasonable to assume that parameters related to frequencies and damping variations should have a weight greater than those obtained for classical frames. In order to estimate the influence of soil on the dynamic response of the structure, further studies are necessary to estimate the influence of frequency variation of soil during the strong motion.

ACKNOWLEDGEMENTS

This study was developed within the project DPC-RELUIS 2005-2008 (Research Line 9) funded by the Department of Italian Civil Protection.

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TABLE AND FIGURE CAPTION

Table 1: Correlation between damage index and damage level [Park and Ang, 1985 and 1987]

Figure 1: Example of application of STFT applied on a non-stationary signal

Figure 2: (a) Application of logaritmic decrement method on a recorded signal. (b) Statistical distribution of values.

Figure 3: Scheme of approach used in the method.

Figure 4: Pictures of POP and TREMA Models

Figure 5: POP. Correlation among maximum inter-story drift and instrumental parameters.

Figure 6: TREMA. Correlation among maximum inter-story drift and instrumental parameters.

Figure 7: POP Model: weight contribute for each instrumental parameter vs experimental test.

Figure 8: TREMA Model: contribute for each instrumental parameter vs experimental test for both X and Y directions.

Figure 9: POP Model: experimental vs analytical maximum inter-story drift

Figure 10: TREMA model: experimental vs analytical maximum inter-story drift for both X and Y direction

Figure 12: Correlation among maximum inter-story drift and instrumental parameters.

Figure 13: Experimental vs analytical maximum inter-story drift for soil A-BCE-D.

Figure 14: Weight contribute for each instrumental parameter vs experimental test.

Figure 15: Correlation between analytical maximum drift and damage index evaluated by Park and Ang procedure

TABLE 1

Damage Index	Damage Level	Building Appearance	Reparability
>1.0	Collapse	Local or global collapse of building	No
0.4-1.0	Heavy	Extensive Cracks of concrete and buckled bars	Hardly Repairable
< 0.4	Moderate	Extensive Cracks; expulsion of concrete from critic zones of elements	Repairable
---	Low	Few cracks distributed on building; expulsion of cover concrete from columns	Easily Repairable
---	Weak	Few Cracks	Total

FIGURE 1

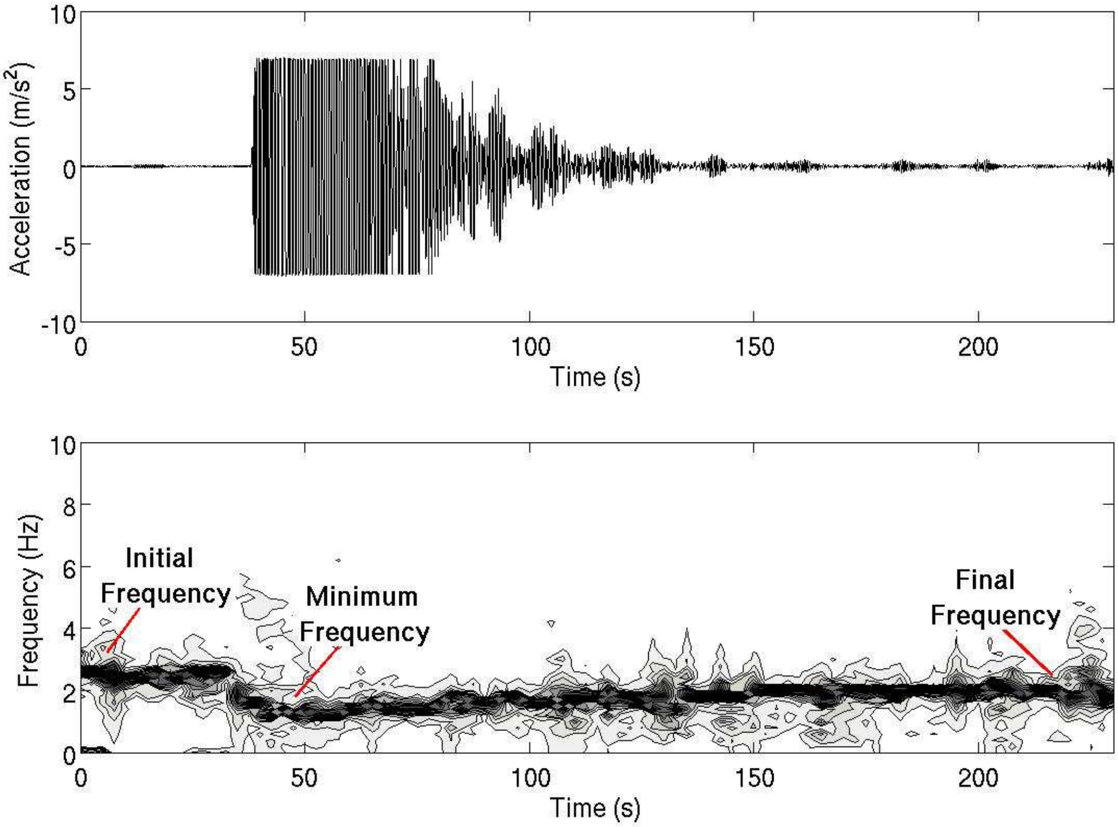
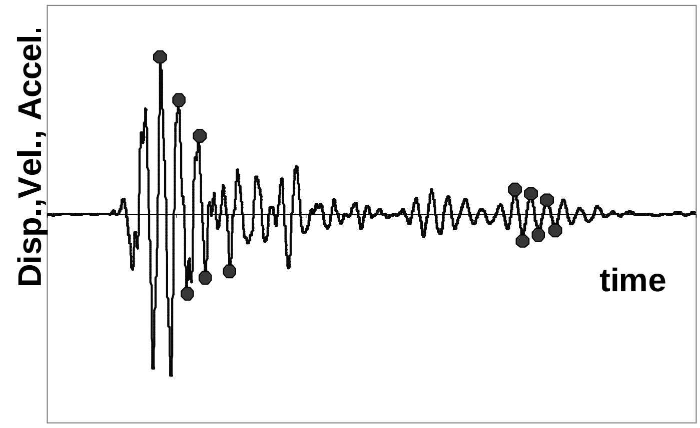
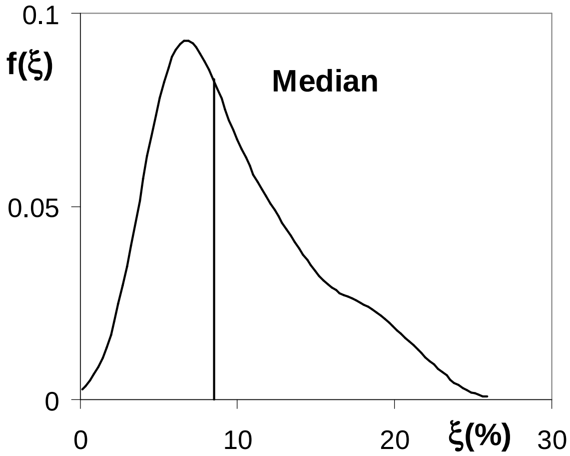


FIGURE 2



a)



b)

FIGURE 3

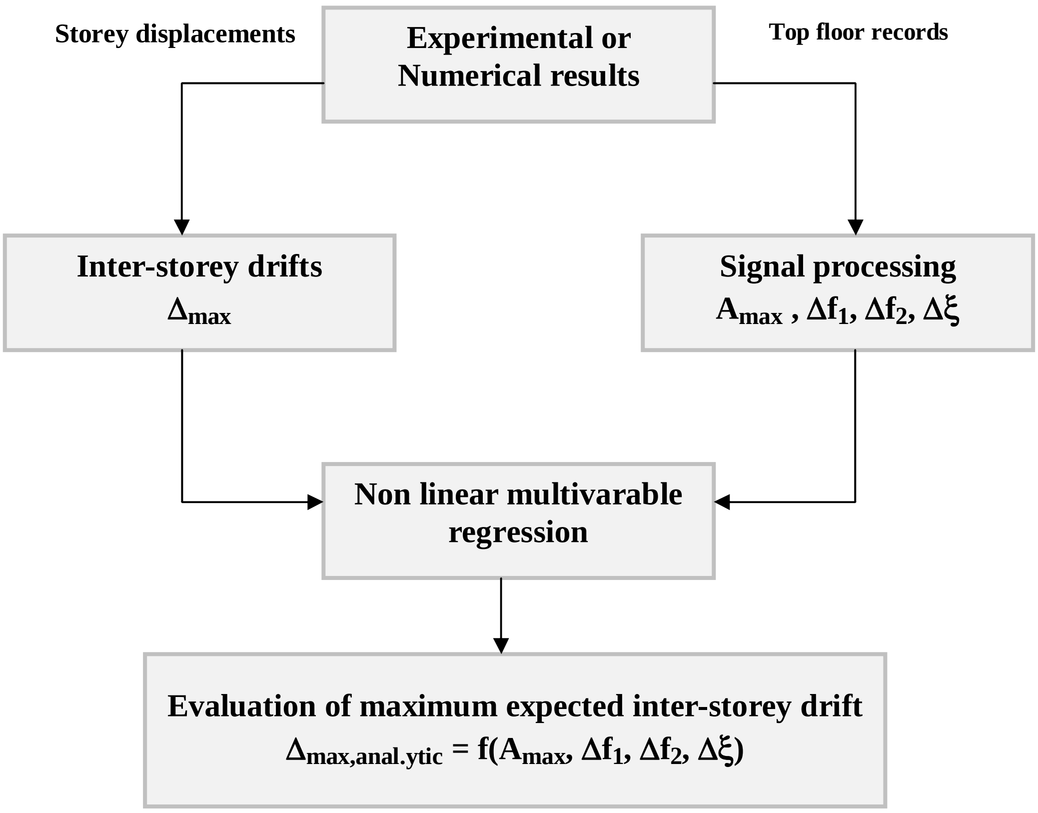
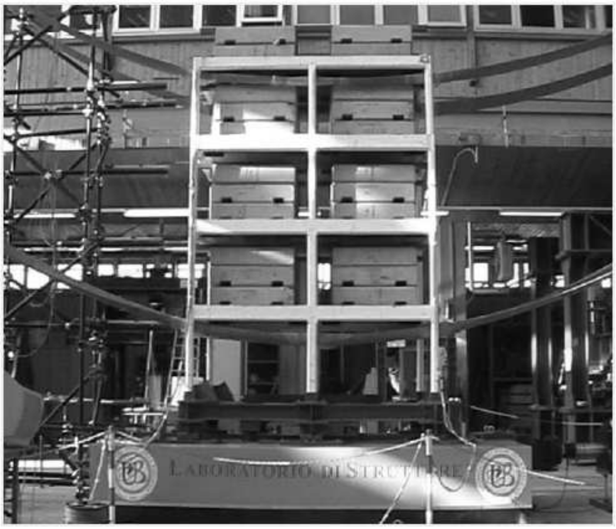
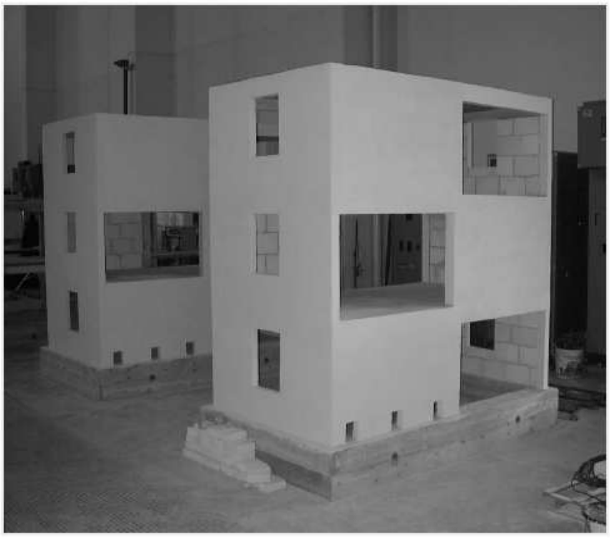

Storey displacements
Experimental or Numerical results
Top floor records
Inter-storey drifts
Δ_{max}
Signal processing
A_{max} , Δf_1, Δf_2, $\Delta\xi$
Non linear multivarable regression
Evaluation of maximum expected inter-storey drift
$\Delta_{max,anal.ytic} = f(A_{max}, \Delta f_1, \Delta f_2, \Delta\xi)$

FIGURE 4



POP Model



TREMA Model

FIGURE 5

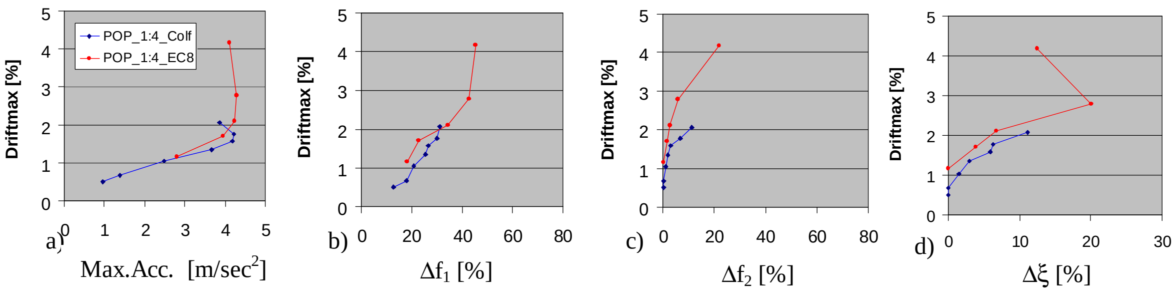


FIGURE 6

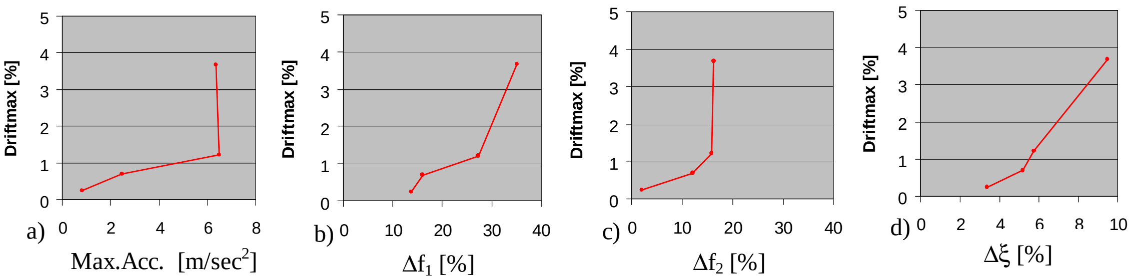


FIGURE 7

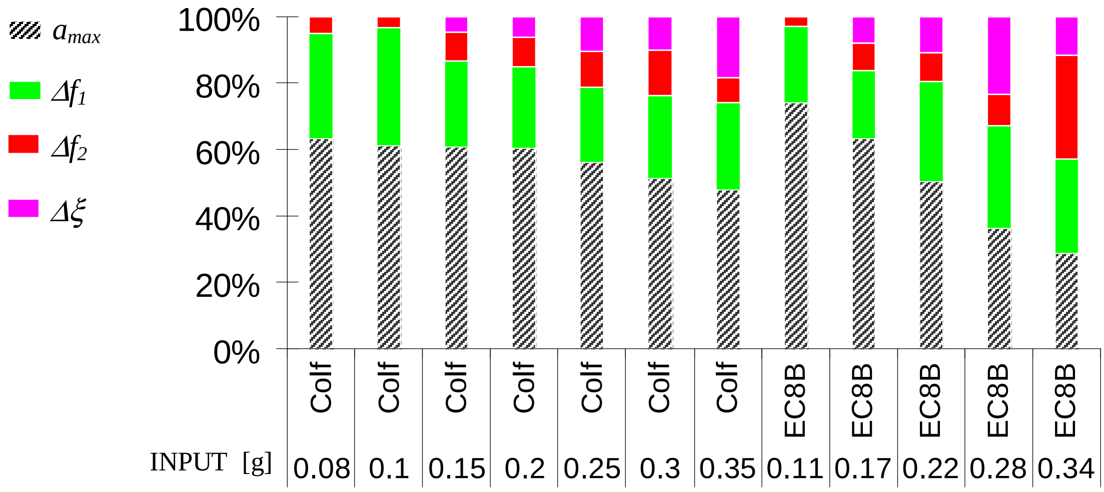


FIGURE 8

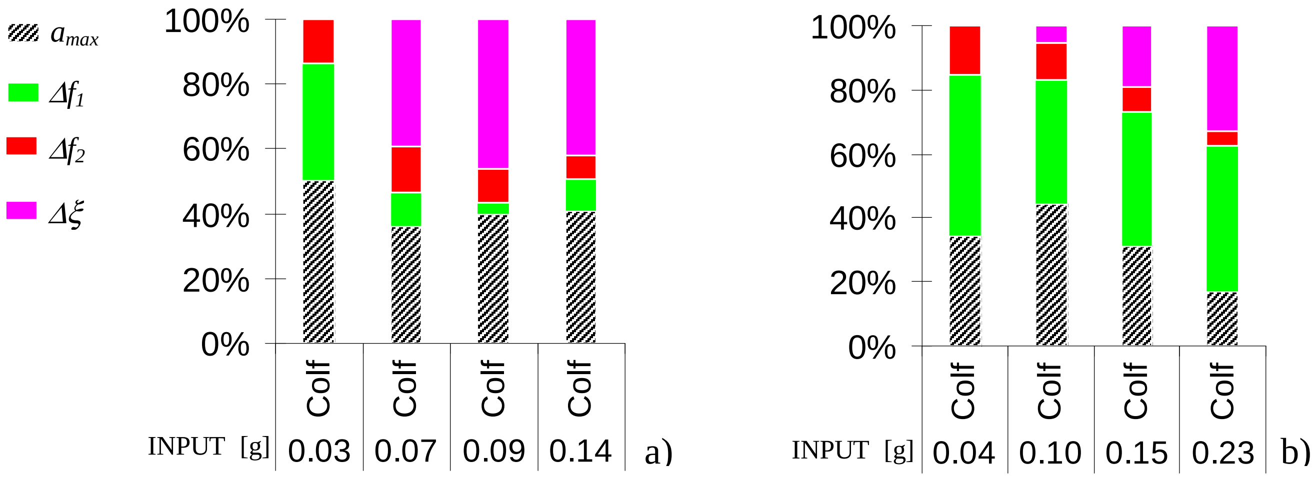


FIGURE 9

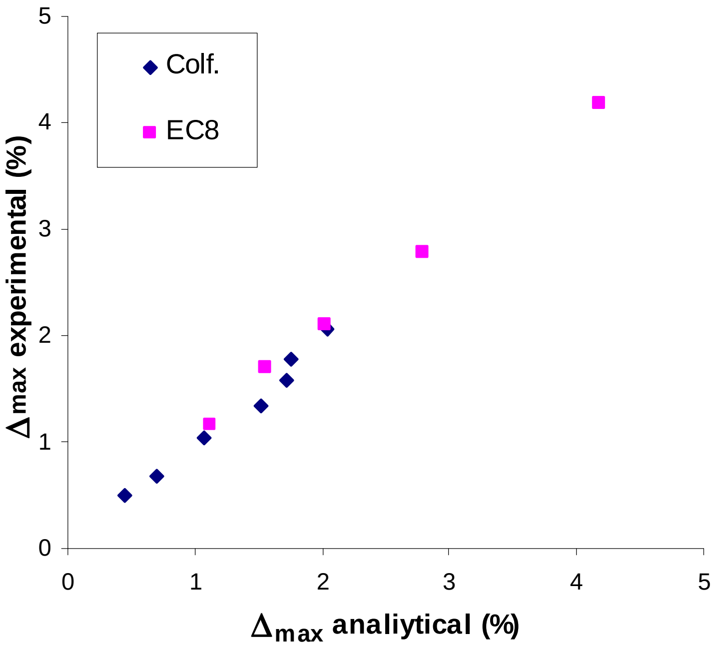
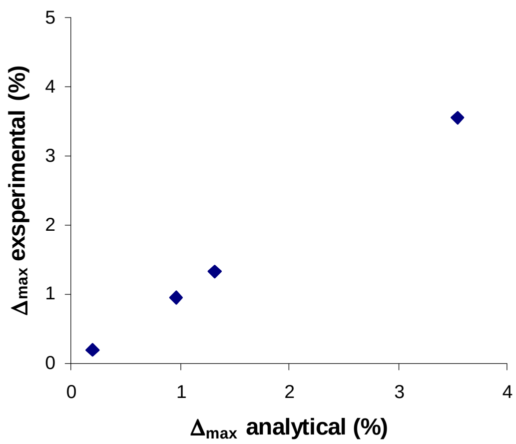
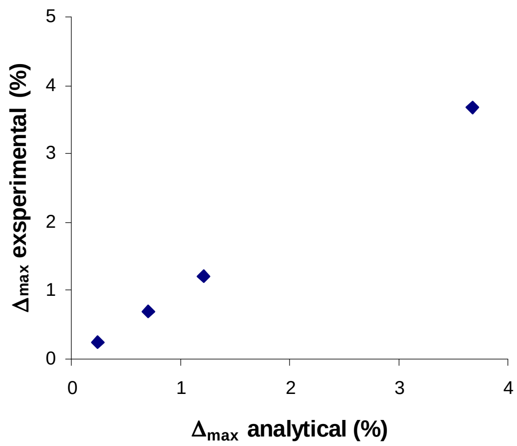


FIGURE 10

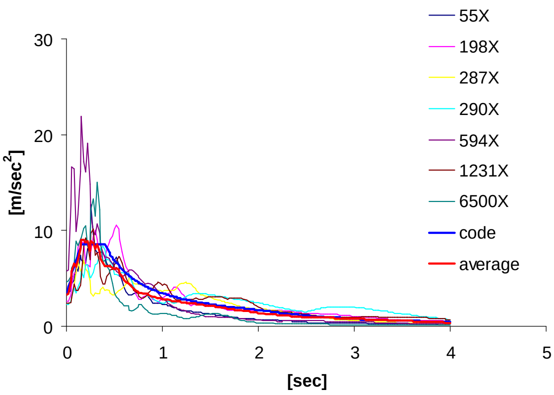


Dir. X

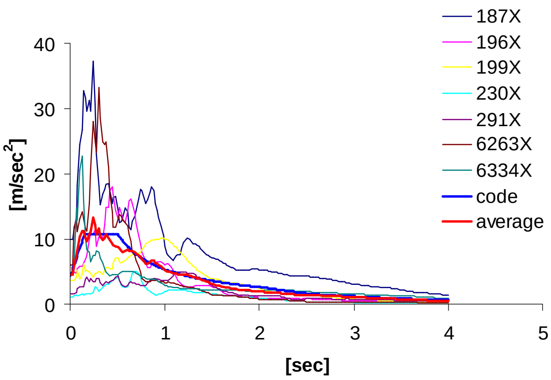


Dir. Y

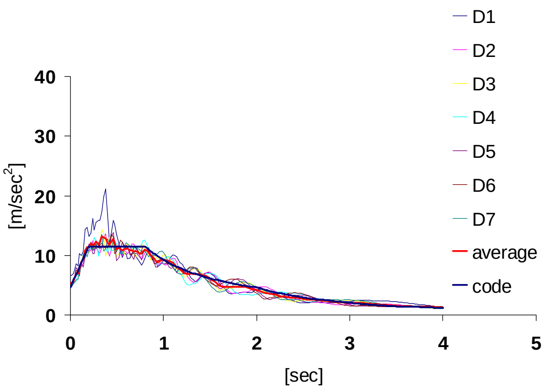
FIGURE 11



55X
198X
287X
290X
594X
1231X
6500X
code
average
[m/sec2]
[sec]



187X
196X
199X
230X
291X
6263X
6334X
code
average
[m/sec2]
[sec]



D1
D2
D3
D4
D5
D6
D7
average
code
[m/sec2]
[sec]

FIGURE 12

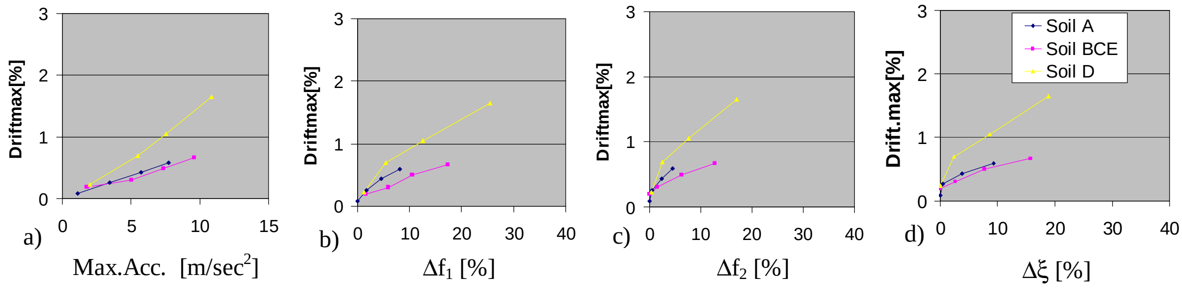


FIGURE 13

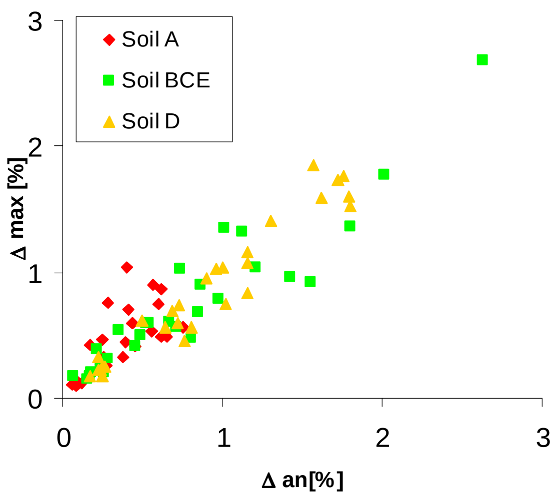


FIGURE 14

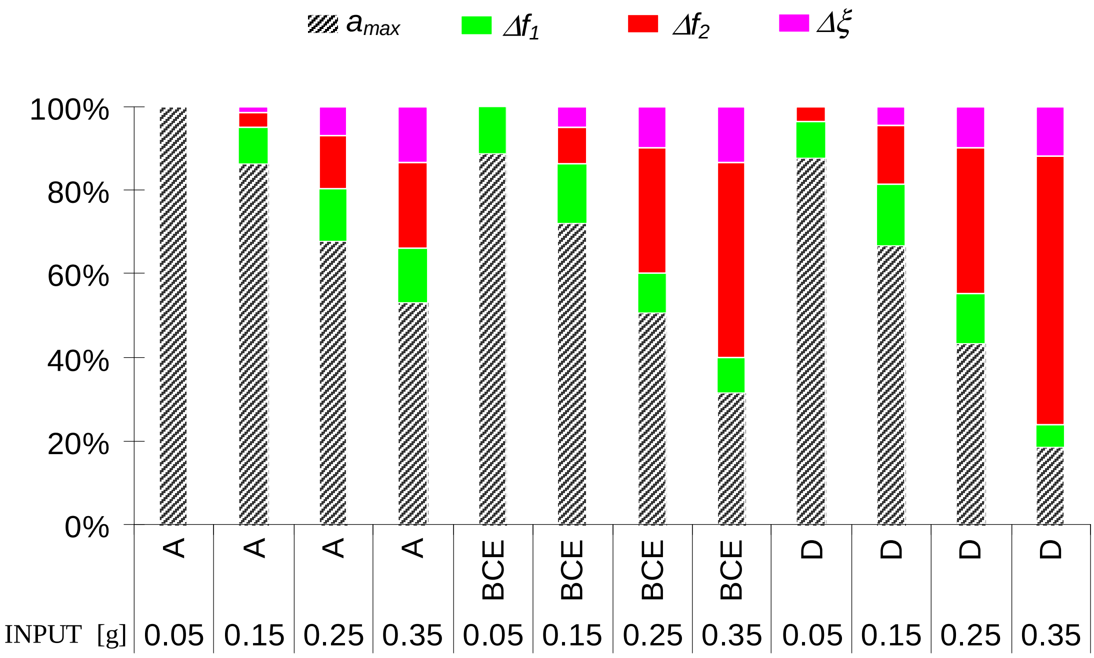


FIGURE 15