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A Comparative Earthquake Analysis of Structures by a Wave Propagation Method

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Abstract

The excitation and response of structures in epicentral regions of shallow focus earthquakes is quite complicated in which the horizontal and the vertical ground motions are almost simultaneous. In these cases the response of a structure cannot be simulated by a single time history that horizontally and uniformly strikes it.

In the present communication it is tried to simulate the input motion by a transient ground motion, in the form of a wave that is traveling along the foundation of the structure with a certain group velocity. The ground particles perform oscillations in the vertical plane, that results from the convolution of horizontal and vertical components of a recorded ground motion. The motion, from the footings of the columns is propagated into the body of the structure as longitudinal and shear waves, producing displacements and member forces.

This type of excitation is applied, with a variety, at a time, of group wave velocities 200 and 800 m/sec, predominant frequencies of particle oscillation 20, 5, 3, 2 Hz and a ratio of vertical to horizontal particle acceleration is equal to V/H=0.59, 2.0 and 3.0. The reference time history is that of the Cephalonia 2014 ground acceleration record, while the amplitudes and the predominant frequencies are scaled accordingly. For each one of these types of the horizontal only component time histories, the respective acceleration response spectra are defined and, for the structures mentioned below, the respective structural response is calculated. Each one of these corresponds to the respective reference structural response, and the displacements of various levels and member forces at selected critical cross sections are called “reference values”.

In order to illustrate the effect of the wave type excitation and the resulting structural response, a set of two pairs (a three and an eight storied) of reinforced concrete plane frames are selected. The two pairs are identical to each other except for the clear beam span that in one set is 4.0 m and in the other is 8.0 m. In the same levels and cross sections, with those mentioned above, of the load bearing system, the resulting displacements and member forces are measured, that are called “test values”. The ratios between the “test” and the respective “reference values” are calculated and presented graphically in well understandable charts.

From the comparison between “reference” and “test values” it is concluded that the said ratios may reach very high values.

Keywords: Earthquake analysis of structures; wave propagation in structures; epicentral regions of shallow earthquakes; vertical earthquake component

1. INTRODUCTION AND SCOPE OF THE INVESTIGATION

The earthquake response of a structure may be evaluated by various methods of loading. For example, one is the method by a simultaneous excitation of its base, as this is schematically shown in Fig.1a. This method is usually described in existing seismic codes, that, in general, is followed by Engineers for the seismic design of structures. Nevertheless, an other much simpler method is by applying a horizontal loading specified by forces or displacements along the height of the structure, that it is also foreseen by codes, mainly, for the design of simple structures.

A methodology that is used in the present communication is by a wave propagation procedure, in which the excitation of the base is progressing in space and time. Namely, the particles of the ground, perform the motion as it is described by the X, Y and Z components of the strong motion records that is propagating with a certain group wave velocity along the base-foundation of the structure. In the analytical procedures performed, the excitation of the structure is resulting by considering a displacement wave that is propagating into the body of the structure introduced to it successively through its footings. This procedure is schematically shown in Fig. 1b. An almost similar methodology by considering a phase lag of the excitation between the various supports, is usually applied for the seismic design of long span bridges. The interest of researchers on the subject has been expressed in the past [1, and references there in]. The relevant investigation for long buildings was achieved with the use of analytical closed form solutions for the case of incident monochromatic, plane SH seismic waves.

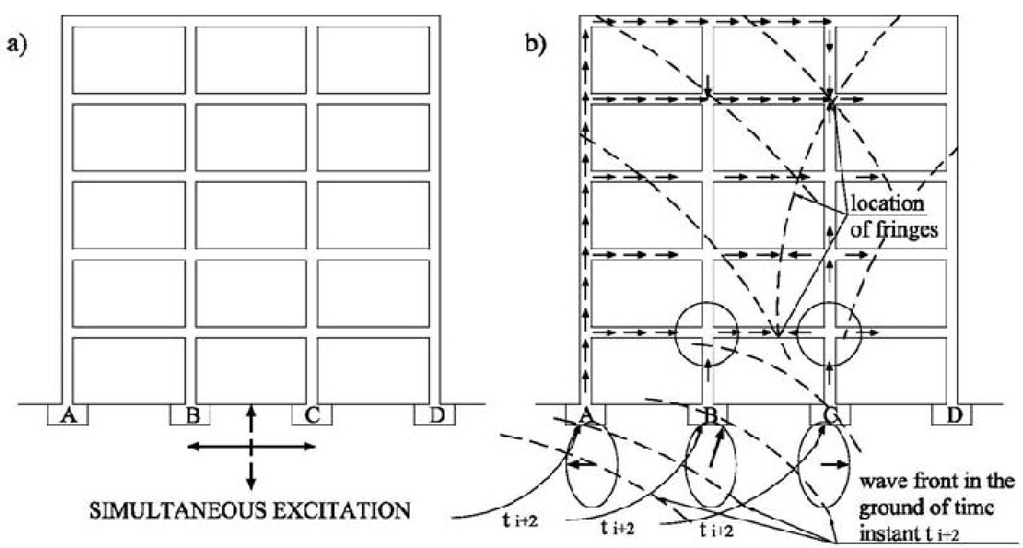

Fig. 1 – Comparison of the response of structures resulted by two different seismic loading methods. (a) By a simultaneous excitation of the base of the structure, following the modal response spectrum method. (b) By a wave propagation procedure, where fringes – convolutions of strains in various locations of the structure may occur. In the figure, the time instant t_i indicates the time when the first column footing is stricken by the wave front.

The scope of the present communication is to present a comparison of the response and the loading of structures that are resulting between two different methods of seismic excitation and analysis, namely: the first method is the well known modal response spectrum dynamic analysis, and the second one is the wave propagation methodology as it is applied here in.

It is deemed that the modal response spectrum dynamic analysis method is one of the most commonly used methods for the earthquake analysis of structures. On the other hand, it was deemed that the wave propagation procedure may produce results that are closer to the real earthquake loading and response of structures.

For the analytical procedure the N160 and Z components of the ARGA_20140203_0309 records obtained at 11.0 km epicentral distance, during the Cephalonia earthquake of February 3rd 2014, compiled by [2], were used. The focal depth was 8 km. Their acceleration, time histories and the respective total acceleration response spectra for 5% damping are respectively shown in Fig. 2. The predominant frequency of the N160 component is found to be f_1=2.0Hz.

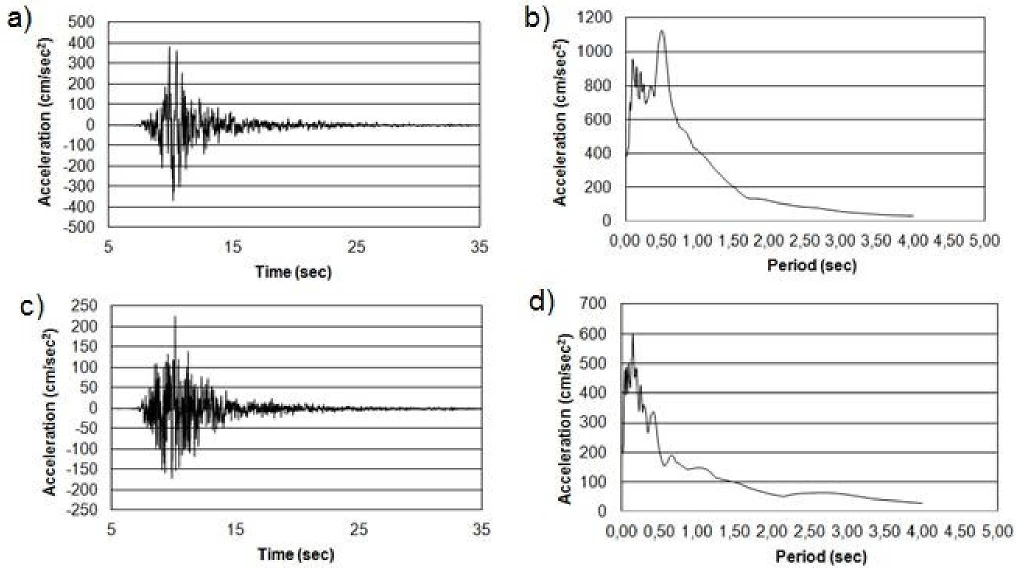

Fig. 2 – Corrected acceleration time histories [1] and their total response spectra for 5% critical damping recorded at Argostoli, Cephalonia, during the Cephalonia earthquake of February 3rd 2014. (a), (b) the N160 component and (c), (d) the Z component.

The recorded ratio (V/H) of the respective peak ground accelerations is $\max(a_V)/\max(a_H)=2.264(\text{msec}^{-2})/3.832(\text{msec}^{-2})=0.59$.

2. INVESTIGATED PARAMETERS

In order to carry out the comparison, the same building models were used, applying either the modal response spectrum analysis method, or the wave propagation procedure. Further, the influence of the following parameters were investigated. These parameters are separated in two main groups. The first is related to input motion characteristics (three parameters) and the second one is related to building frame characteristics (two parameters).

2.1 Input motion characteristics

a. Influence of the wave propagation velocity. For the wave propagation procedure the ground motion is translated along the footings of the structure, with a group wave velocity equal to c_1=200 (msec^{-1}) and c_2=800 (msec^{-1}).

b. Influence of the predominant frequency of the natural horizontal and vertical time histories. The ground motion was modified in order to possess predominant frequencies (f), f_2=3Hz, f_3=5Hz and f_4=20Hz. In all these artificial time histories the values of the peak ground accelerations were kept constant with those of the prototype ground motion (with f_1=2Hz). On the contrary, for the velocity ($v_i(t)$) and the displacement ($d_i(t)$) values of the time histories, the simple frequency time scaling rule was applied, so that:

$$v_i(t) = v_1(t) \bullet (f_1/f_i), \text{ for } i=2,3,4 \tag{1a}$$

$$d_i(t) = d_1(t) \bullet (f_1/f_i)^2, \text{ for } i=2,3,4 \tag{1b}$$

and the duration (D_i) of the respective ground motions becomes:

$$D_i = D_1 \bullet (f_1/f_i) \tag{1c}$$

c. Influence of the ratio of the Vertical over the Horizontal (V/H) peak ground accelerations. As it is already mentioned, the ratio of the Vertical over the N160 Horizontal acceleration was V/H=0.59. The epicentral distance of the accelerograph that recorded the said motions is about 11.0 km. Therefore, the instrument is located in the outer margins of the epicentral region, given the focal depth of the earthquake to be 8.0 km. In smaller epicentral distances the said ratio V/H becomes higher. There are cases [3], [4], [5], in which the V/H ratio may reach much higher values. Therefore, it was decided to investigate the influence of higher values of the V/H ratio, namely $(V/H)_2$=2.0 and $(V/H)_3$=3.0, besides the initial one of $(V/H)_1$=0.59.

2.2. Building frame type characteristics

In order to facilitate the analytical work carried out in the present investigation, it was decided to use instead of 3D building models, 2D frame models, assuming that a building consists out of a set of similar parallel plane frames and the seismic excitation takes place within the vertical plane of the frame.

a. Influence of the number of stories of building frames: Two types of building frames have been selected possessing identical plans and load bearing systems, but with different number of stories. The one is a three storied and the other is an eight storied, as shown in Fig. 3. The clear beam span is in both frames equal to 10.0 (m), and the clear height of each storey is 2.40 (m).

b. Influence of the beam span: An other set of two building frames that are identical with those described above, except for the clear beam span that is now equal to 4.0 (m), as it is shown in Fig. 3.

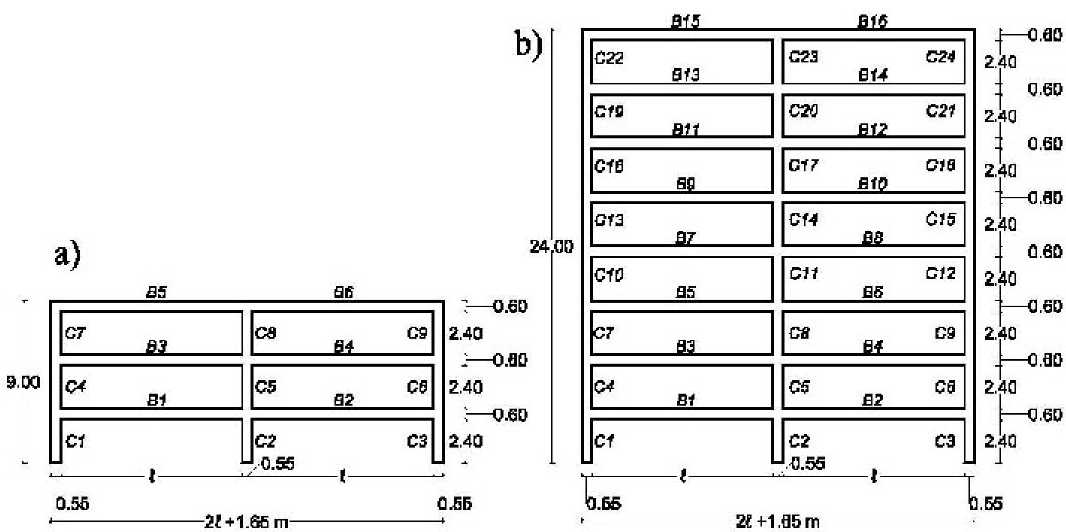

Fig. 3 – (a,b) A three and eight storied building frame, where ℓ (m) is the clear beam span. There are two sets of buildings. One with ℓ=10.0 (m) and another with ℓ=4.0 (m). The alphanumeric indicators show the column (C) or beam (B) number, to which the comparison of the developed member forces is carried out (see Figs 4,5,6,7).

Given that the above mentioned varying parameters apply to four different structures, as already shown in Fig. 3, the total “test” analyses reach the number of 4(building frame types) x 2(levels of group wave velocities) x 4(different predominant frequencies) x 3(different V/H ratios) = 96 cases. To this number it must be added the modal response spectrum (dynamic) analyses for the “reference” calculations that reach the number of: 4(building frame types) x 4(different predominant frequencies) = 16 cases. Due to the huge amount of data that was collected from the 96 + 16 = 112 dynamic analyses, only a few of the most important ones are presented in supervisory charts for clearer understanding due to space limitation of the present communication.

3. MODELLING AND ANALYSIS

The whole procedure for the wave propagation analysis was carried out with the use of the computer code Abacus [6]. For the excitation of the structure, the input motion must be a displacement time history, that resulted after double integration of the acceleration, using the code [7].

The time that is required for the input wave motion to travel from the center of one support to the next one is equal to $(L_{cl}+0.55)/c_i$, where L_{cl} is the clear beam span in (m) and c_i is the group wave velocity.

The analysis was computed using finite element procedures according the ABACUS code, as already mentioned, applying the explicit dynamic method.

In the explicit dynamic method, at every time step, the dynamic equation of equilibrium is solved:

$$m\ddot{u} + d\dot{u} + ku = p(t) \tag{2}$$

The time step of the analysis is calculated using the Courant [8] criterion:

$$\Delta t = b/c \tag{3}$$

where b is the characteristic length of the element and c is the sound speed for shell elements:

$$c = (E/(\rho(1-\nu^2)))^{1/2} \tag{4}$$

where E is the Young's modulus, ρ is the density of the mediums and ν is the Poisson's ratio.

For stability reasons the final time step is calculated at 1.09E-05 (sec). The structural damping ratio for the earthquake action is taken equal to 5% in all cases.

The distributed load on the beams, including the self weight is calculated at 90.4 kN/m.

The material of the frames is considered linear elastic with properties E=41 GPa, $\rho=2500$ kg/m^3 and $\nu=0.20$. The additional load of the beams is modeled by proper mass scaling of the upper line of shell elements at every beam.

The discretization of the columns is a grid of 0.05x0.05 (m^2) elements and the discretization of the beams is a grid of 0.05x0.10 (m^2) elements, where the 0.05 (m) edge is along the vertical axis of the beams.

Each storey has a total height of 3.00 (m), the cross section of the columns is 0.55x0.55 (m^2) and the cross section of the beams is 0.40x0.60 (m^2).

Every building frame is symmetrical along the vertical central axis and possesses three columns and the same beam length per storey, as it is shown in Fig. 3, therefore, the seismic response resulting from conventional calculations produces symmetrical loading - deformation. The duration of the analysis is extended beyond the termination of the ground motion for a time window equal to three times the fundamental period of each building frame in order not to ignore any value of the response that could contribute to the research.

The analyses for the "reference" cases were carried out using a "commercial" code following a conventional procedure. Actually, in order to get reliable results, that are worth to be classified as the "reference" ones, a few cases were selected and analyzed by two "commercial" computer codes. The results were found almost identical. Finally, for the needs of the present investigation and for the analyses of at least 16 cases, as mentioned above, the computer code: ScadaPro16 [9] was used, following the "modal response spectrum analysis" as this is described by [10].

4. PRESENTATION OF RESULTS

The huge volume of the results is in part presented in an illustrative way so that to be easily understandable. For this reason in Fig. 3 the reference key alphanumerics that indicate the location where the comparison is carried out in column (C) or beam (B) cross sections, are shown. In Figs 4, 5, 6 and 7 the ratios between the resulted member forces N, M, V, after the application of the wave propagation procedure ("test" cases) over the respective "reference" ones N_{ref}, M_{ref}, V_{ref}, that resulted after the application of the modal response spectrum analysis at the same structural member are shown, for each one of the investigated four building frame types. All these ratios refer to the absolute maximum values of each member force, that was calculated, but the sign of the resulted ratio was maintained in the respective charts. Therefore, one may notice in those charts plenty of negative values of ratios. This fact possesses a major importance in cases where column axial forces are compared, namely in Figs 4a,b,c, Fig. 5f, Figs 6a,b,c,f, Figs 7b,c. Statically, it is worth mentioning that the most cases of this fact is for the low rise building frames, and for smaller beam spans.

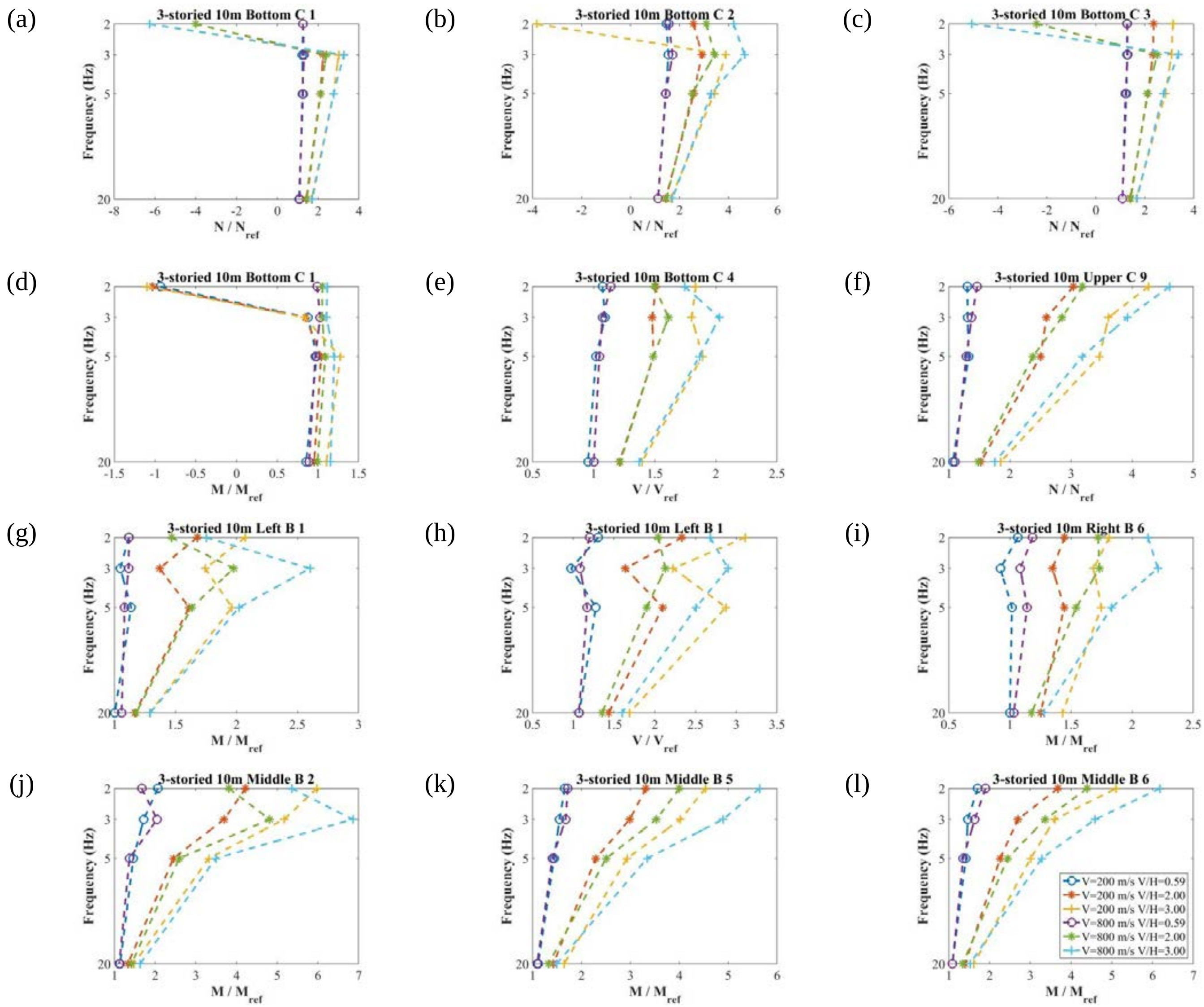

Fig. 4 – Ratios of Axial Forces (N) over (N_{ref}), Moments (M) over (M_{ref}) and Shear Forces (V) over (V_{ref}), developed at various locations of the three storied building frame, with a clear beam span of 10.0 (m). The key alphanumeric indices indicate the location of the column (C) or beam (B) cross section, and they are respectively shown in Fig. 3.

The columns (C_1) and (C_3) in the examined four building frame types are symmetrical, and their overall seismic response should be symmetrical too, as already mentioned. In order to illustrate how these structural members are responding to each one of the said two methods of analysis, the charts (a) for (C_1) and (c) for (C_3) are compiled in Figs 4 to 7. Statistically, the graphs in all these charts are not at all the same, although refer to symmetrical members, except for Fig. 5 which refers to the eight storied building frame with 10.0 (m) clear beam span, in which the graphs possess an increased similarity. This observation yields for the "test" only cases since the respective seismic loading resulting from the modal response spectrum analysis the – "reference" cases – gave symmetrical results.

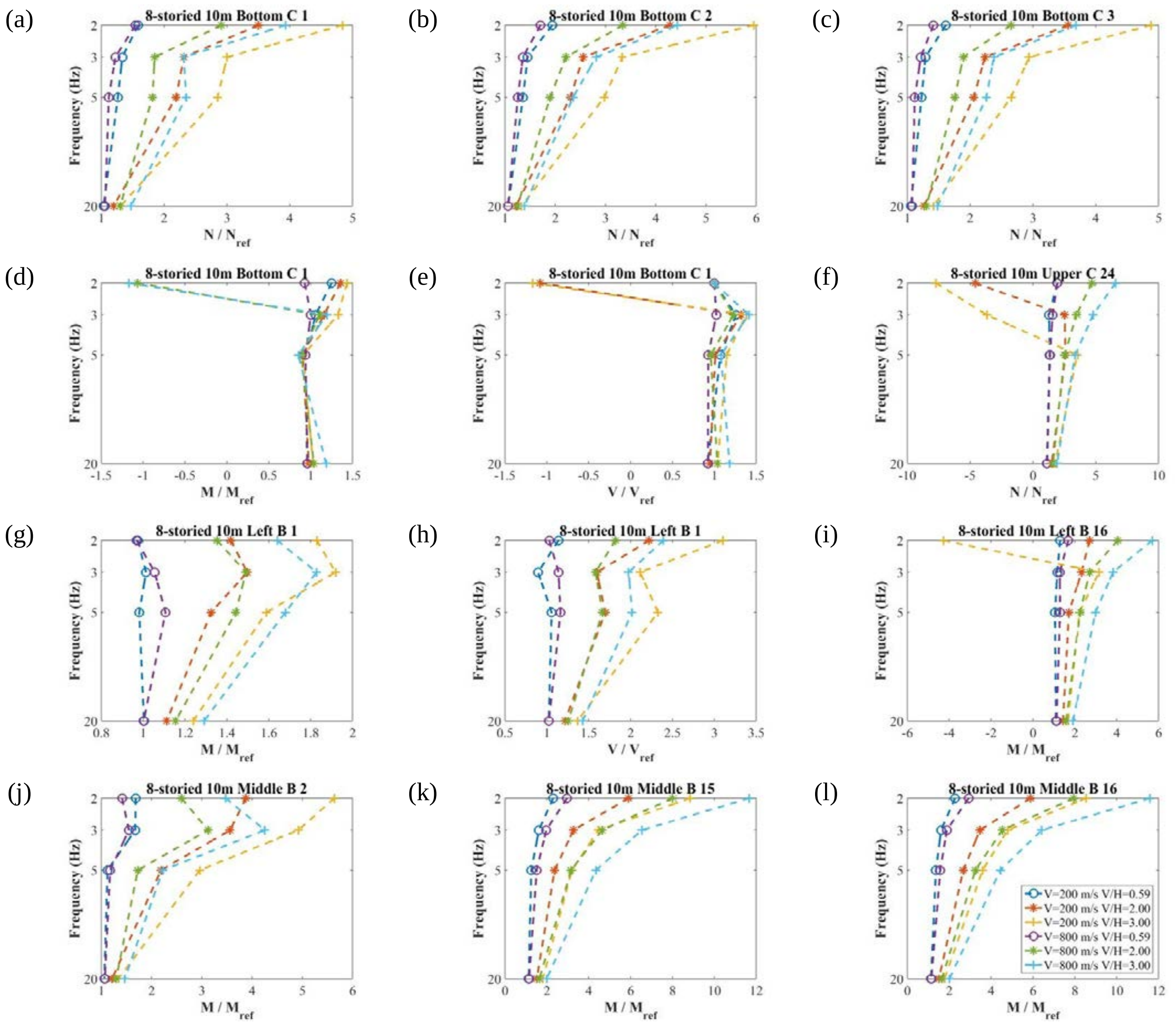

Fig. 5 – Ratios of Axial Forces (N) over (N_{ref}), Moments (M) over (M_{ref}) and Shear Forces (V) over (V_{ref}), developed at various locations of the eight storied building frame, with a clear beam span of 10.0 (m). The key alphanumeric indices indicate the location of the column (C) or beam (B) cross section, and they are respectively shown in Fig. 3.

In Figs 8a,b,c, the ratios of the eccentricities $e/e_{ref}=(M/N)/(M_{ref}/N_{ref})$ of the three lower columns are shown for the three storied building frame with 4.0 (m) clear beam span, while in Figs 8d,e,f, the ratios are for the eight storied building frame with 10.0 (m) clear beam span. The calculation of the said ratios is carried out for the whole duration of the analysis, that is extended after the termination of the ground motion for a time window equal to three times the fundamental period of the building frame, as already mentioned. Especially, in order to serve this particular case, the response of the "reference" building frames was carried out following the in time integration method.

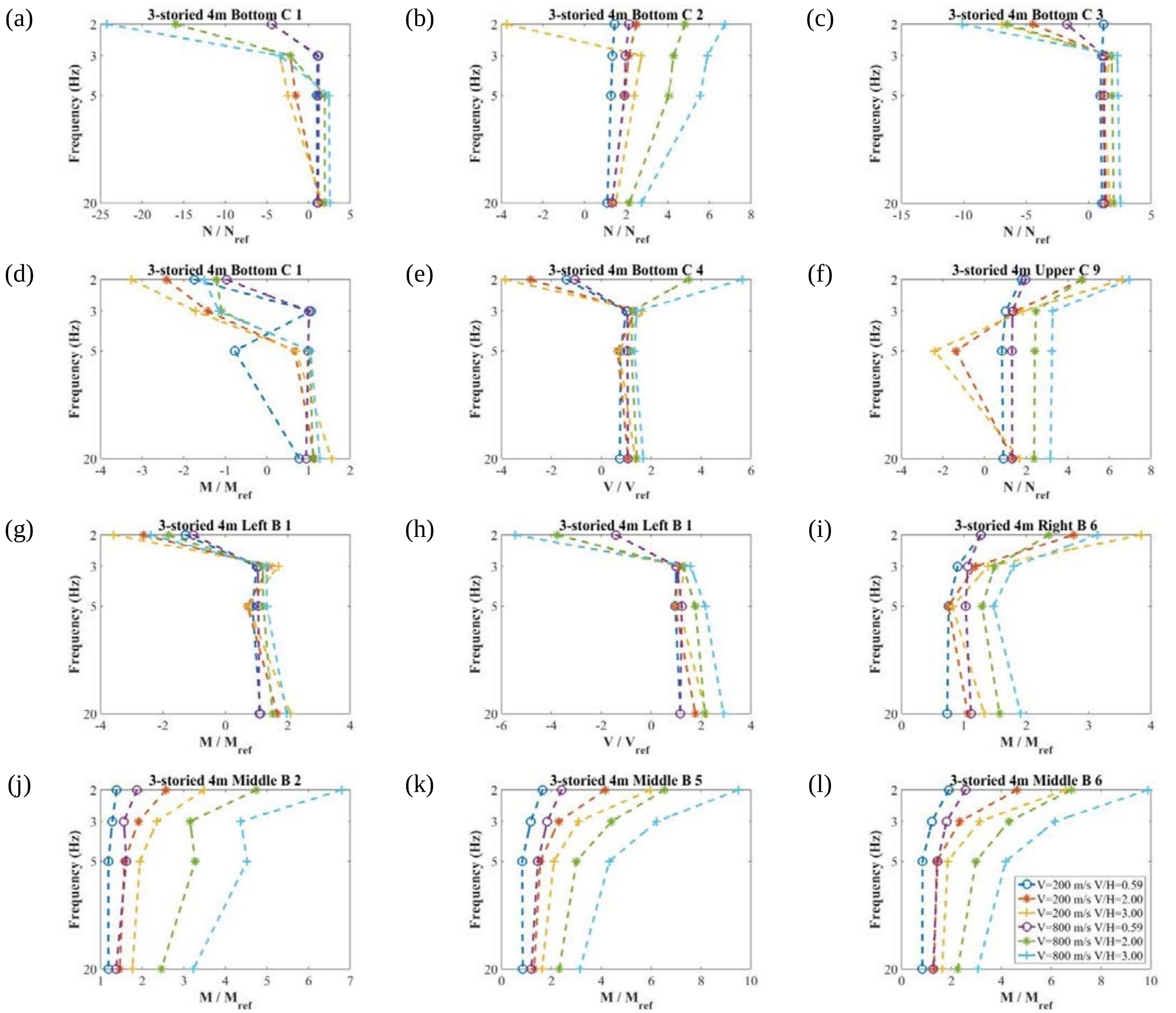

Fig. 6 – Ratios of Axial Forces (N) over (N_{ref}), Moments (M) over (M_{ref}) and Shear Forces (V) over (V_{ref}), developed at various locations of the three storied building frame, with a clear beam span of 4.0 (m). The key alphanumeric indices indicate the location of the column (C) or beam (B) cross section, and they are respectively shown in Fig. 3.

It is worth mentioning here, that for the calculation of the ratios e/e_{ref}, pairs of eccentricities in which $e_{ref} \geq 0.05$ (m) were only taken under consideration. Otherwise, the resulted eccentricity at the respective time instant was rejected and the ratio was not calculated. Therefore, the extremely high values of the ratios e/e_{ref}, that one may notice in Fig. 8 are mainly due to the respectively very high values of the eccentricities $e=M/N$ of the "test" cases and are not due to very small values of $e_{ref}=M_{ref}/N_{ref}$. This finding is extremely important, and it might be attributed to very low and negative values of the column axial forces (N) that result after the application of the wave propagation procedure.

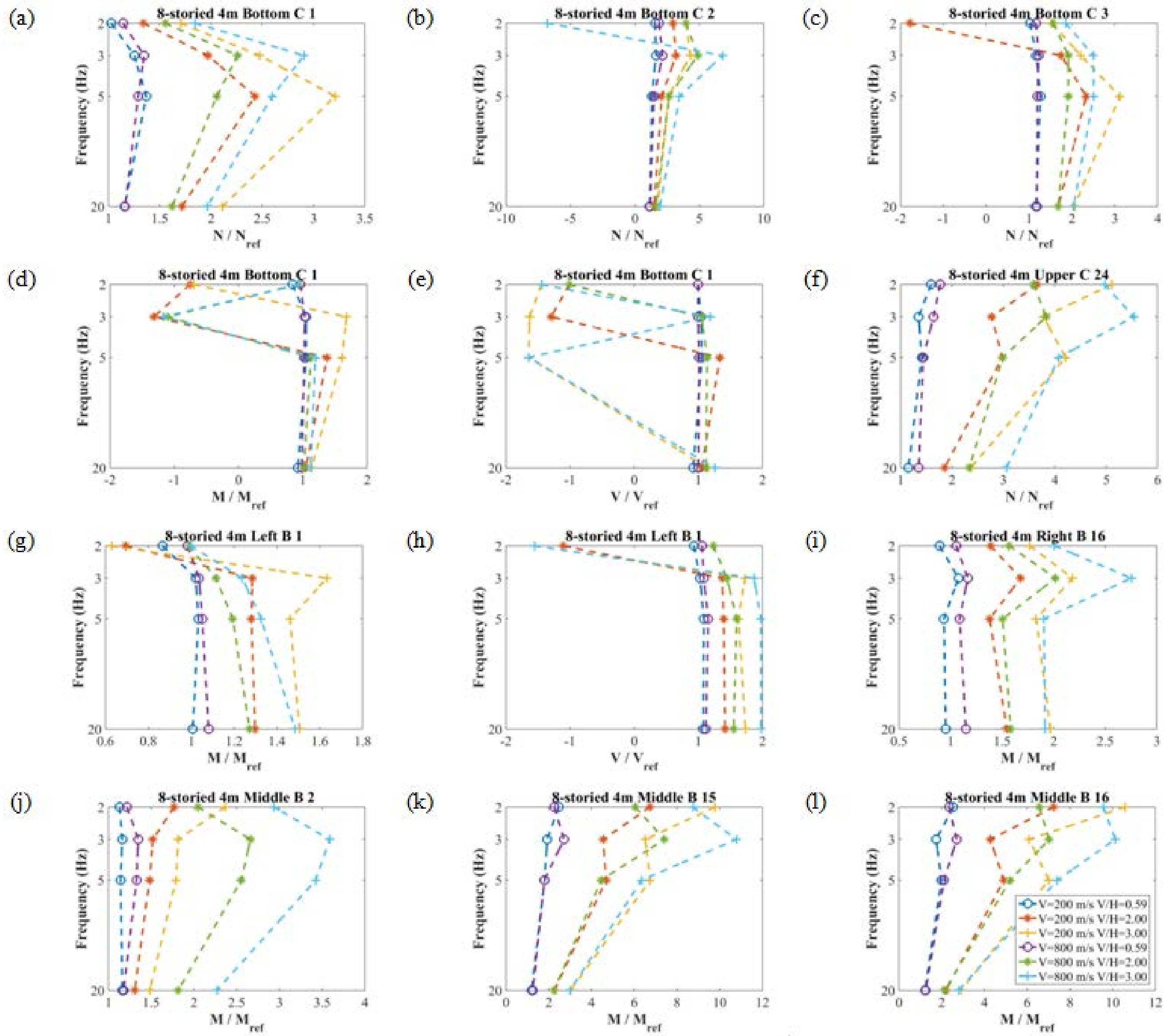

Fig. 7 – Ratios of Axial Forces (N) over (N_{ref}), Moments (M) over (M_{ref}) and Shear Forces (V) over (V_{ref}), developed at various locations of the eight storied building frame, with a clear beam span of 4.0 (m). The key alphanumeric indices indicate the location of the column (C) or beam (B) cross section, and they are respectively shown in Fig. 3.

As it is schematically shown in Fig. 1b, it was initially estimated that fringes – convolutions of strains may occure in various cross sections of the "test" building frame. Up to the moment that the present communication is under compilation, it stood impossible to determine, in advance, the location of those cross sections. This was proved to be valid for all "tested" building frame types, after the application of the wave propagation procedure. In full disagreement to this finding is the practice followed by the most of the existing "commercial" computer codes, that "by default" provide member forces at standard cross sections as those are the top/bottom of columns and left/middle/right of beams. Furthermore, the same notice is yielding also, for the geometry of the reinforcement at those as already mentioned, standard locations of the load bearing members. Nevertheless, "commercial" codes usually reflect specific official specifications.

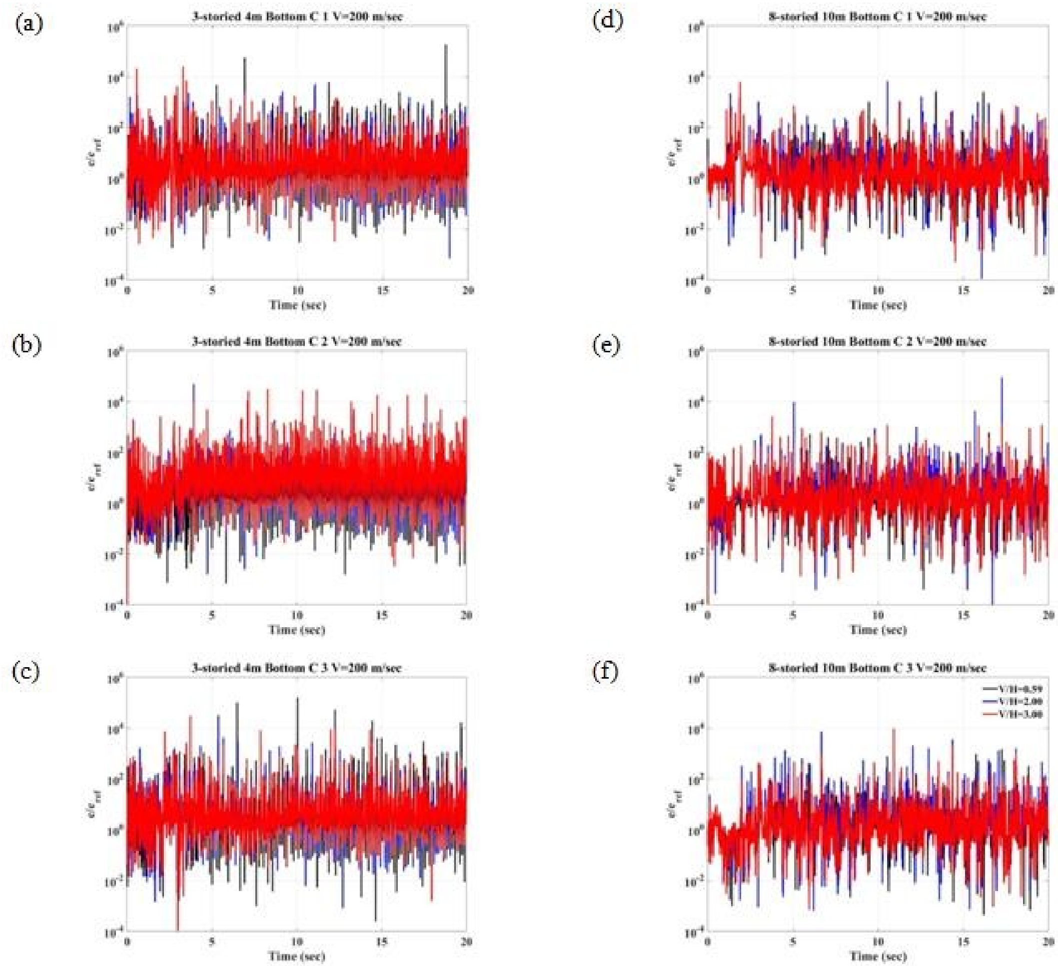

Fig. 8 – Ratios of the eccentricities $e/e_{ref}=(M/N)/(M_{ref}/N_{ref})$ at the bottom of the three lower columns during the whole time of analysis. (a), (b), (c) Three storied building frame with 4.0 (m) clear beam span, and (d), (e), (f) eight storied building frame with 10.0 (m) clear beam span.

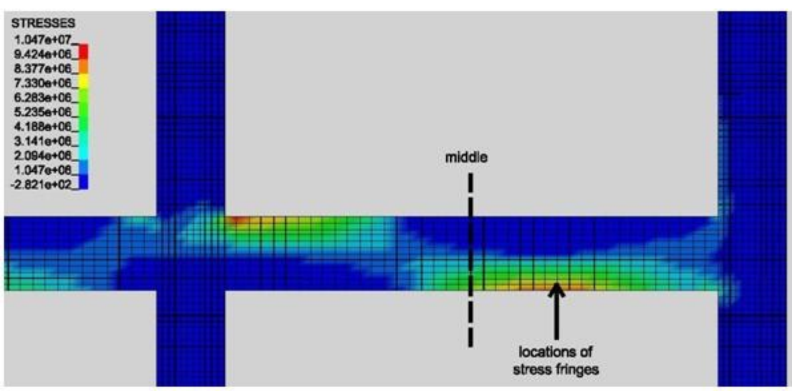

Fig. 9 – A representative image of the created fringes – convolutions of stresses at any location of beams or columns after the application of the wave propagation procedure, at the "test" building frames.

As a final product of the present investigation, is the finding shown in Fig. 10a, in which the three storied building frame with 4.0 (m) clear beam span is deformed in this way. The input motion parameters are V/H=3.0, predominant frequency f=2Hz and front wave velocity c_2=800 (m/sec). Actually, the presented deformation is, in the specific time instant, when the building frame is in the air, during its downwards free fall, before its impact onto the ground. This may happen due to the vertical excitation that is higher than gravity

(=3x0.3832(g)=1.15(g)). On the other hand the lower parts of the basement exterior columns, fixed into the ground, are ejected outwards, as it is usually observed in epicentral regions of shallow earthquakes, and shown in Figs 10b up to e. This outwords ejection might be justified as the result of a static reaction of the lower part of the column. Comparing the finding shown in Fig. 10a with the observations from near field earthquakes, one may conclude that the wave propagation procedure produces results that are closer to the real earthquake response of structures.

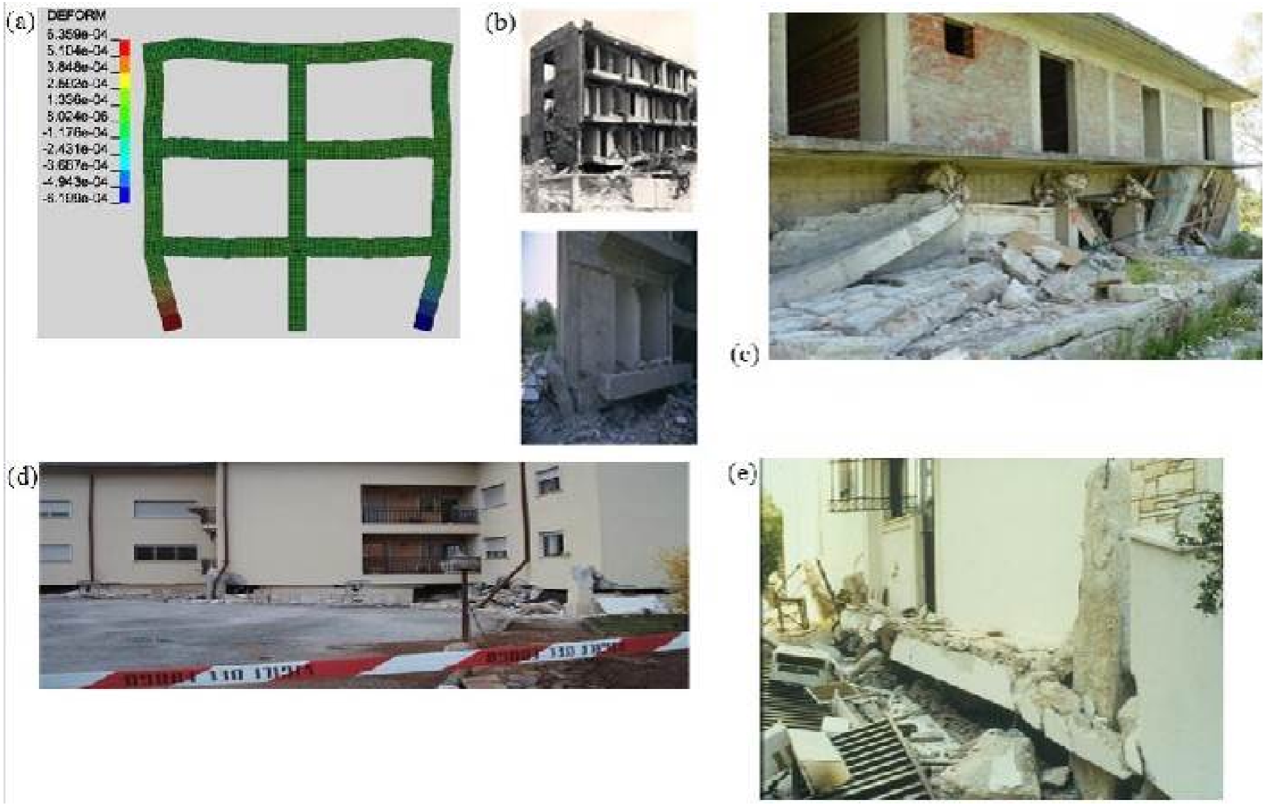

Fig. 10 – (a) A sample of the deformation of the three storied building frame for V/H=3.0, c_2=800 (m/sec), and 4.0 (m) clear beam span. (b) A brand new building damaged after a very close to the structure shallow, M=4.3, 1974 Rion, Patras, Greece, shock. Note the outwards ejection of the lower parts of the basement exterior columns. (c) Similar to Fig. 10b is observed after the 2014 Cephalonian, Greece, earthquakes. (d) Similar case was observed also, after the 2009 L' Aquila, Italy, earthquake. (e) Repetition of the same story after the 1999 Parnitha, Athens, Greece, earthquake.

5. CONCLUSIONS

The conclusions refer to the results obtained be the wave propagation procedure compared to the modal response spectrum analysis.

In symmetrical structures the results substantially differ from the rule of symmetry, due to the way of the excitation of the structure is performed. Thus, at the first column from where the wave motion is introduced into the body of the structure, tensional forces are developed. This is yielding for all V/H ratios. These tensional forces are developed immediately after the input motion meets the first column. The tensional forces that reach the value of 15 to 25 times the respective "reference" compressional ones yield, for the cases of 4.0 (m) clear beam span. For the 10.0 (m) span the respective values are smaller. For V/H=2.0 the tensional forces are 4 times, and for V/H=3.0 the value is 6.5 times the "reference" compressional forces.

Since, from a state of compressional forces in columns another state that of tensional forces is developed, the resulting effects are catalytic: The M/N eccentricities reach tremendous values, the friction on the basement is highly reduced, and some types of base isolation systems may not function properly.

The location of the maximum member forces may not be at the well know positions specified by official recommendation, namely at the top and bottom of columns and at the left, middle and right of beams.

The variation of the response of the beams possesses a special importance in our investigation for the cases of V/H = 2.0 and 3.0, for which the maximum moments may reach 8 times that of the "reference" ones and to be of opposite sign as well.

All parameters examined possess a significant influence on the results of the comparison, namely that of the height of the building frame, that of the frequency of the exciting input motion, that of the beam span, that of the V/H ratio and that of the front wave propagation velocity of the ground motion.

The wave propagation procedure produces results that are closer to the real earthquake response of structures as this is observed in epicentral regions of shallow focus destructive earthquakes.

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