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# SEISMIC PROTECTION OF MONUMENTS AND HISTORICAL BUILDINGS BY A SUSTAINABLE REDUCTION SYSTEM OF THE SEISMIC INPUT MOTION (COMPDYN 2013)

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**Abstract.** *The basic idea of the described research work, lies on the exploitation of the incoherency and the out of phase physical phenomenon among the in-coming seismic waves that excite the structures through their foundation. This beneficial function is valid only if the foundation is quite large, stiff and strong along all directions. The ground motion mitigation system consists of a grillage of reinforced concrete beams that are constructed in direct structural contact with its foundation body. The dimensions of the plan of the orthogonal grillage should be longer that the half seismic surface wave's length in order to achieve a substantial reduction. A parametric analysis was carried out considering the foundation as a two dimensional elastic beam and the velocity of the propagation of the ground motion as basic parameters. Two strong ground motions have been used, each one with quite different characteristics compared to the other one. Finally, a combination of 32 cases was examined, and the resulted motions at the center of gravity of the beams as well as the corresponding response spectra were calculated. The ratios of the resulted response spectra over the free field spectral values at various natural periods and for 5% critical damping are calculated, and it was proved that the reduction may reach the 50% of the free field values. The application of the methodology is presented in various case studies of monumental structures such as churches and residential buildings. These results are checked following the FEMA 440 methodology. It was proved that the difference between the two approaches is within engineering acceptable limits given the large amount of parameters involved in the problem.*

## 1 SCOPE OF THE WORK

With the present work it is aimed to document a design methodology in order to reduce the size of the seismic input motion to monuments and historic building structures. This is achieved by creating a new larger, stiffer and stronger foundation which comes in full structural connection with the existing one. In this methodology are not used any devices. The whole reduction is achieved with structural members that provide sustainability to the whole intervention. This methodology might be proved quite useful for monumental structures where there strengthening unless impossible it is at least difficult and expensive. This difficulty is increasing with the increase of the value of the monument under preservation. On the other hand the intervention in the foundation body starting from a certain depth into the ground does not come to any conflict with the Venice Charter, or other international relevant treaties.

Any strengthening methodology is based on the following well known relation well known inequality:

$$S \leq R \tag{1}$$

where:
S is the loading of a structure, and
R is its resistance.

In the case of the earthquake loading, one could better say that S should be the resulting loading from the earthquake response of the structure. In the present communication we are dealing with monumental structures that we are designing into the elastic – linear domain and ductilities or other nonlinearities are not primarily taken under consideration. Therefore, the loading S is rather proportional to the input motion.

Therefore, the problems from Eq.(1) start when this equation is not satisfied, which means that S>R. Instead of increasing the structural resistance R, one could decrease S by a proportional percentage, without touching the structure. Of course, this procedure involves a limitation, since it is not always possible to decrease S as much as is necessary in order to respect Eq.(1). In that case we must increase the resistance R only for the remaining unsatisfied part of Eq.(1). For example, if it is necessary to increase the resistance R by a percentage 50% to meet Eq.(1), and we can reduce the loading S by 40% only, we have to increase the resistance R of the structure roughly by 10% only.

## 2 HISTORICAL RETROSPECT

Since about 50 years ago, the influence of the size of the foundation to the earthquake response of structures has attracted the interest of various researchers, a quite eminent of which was the late professor G. Housner with his work [1]. He observed the reduction of about 50% of the free field earthquake ground motion along the longer and stiffer E-W direction of a building structure, see Fig.1, during the Arvin Tohachapi, 1952 earthquake.Along the more flexible and slender N-S direction of the structure almost not any change was noticed compared to the free field motion. The respective response spectra obtained are presented in Fig. 2.

It is self understood that the seismic response of the building structure ay its basement automatically constitutes its seismic design excitation along the two main directions of the building.

The monolithic reinforced concrete box of the Hollywood Storage building possesses the following fundamental periods: Along the longitudinal E-W direction, T=0.49 sec while along the transverse N-S direction, T=1.20 sec. The distance between the building and parking lot (P.L.) is about 37 m, where the accelerograph is functioning into a light metallic shelter 2 m x

3 m in plan. The record of the latter instrument may be considered as the free field earthquake motion. As it is shown in Fig. 2a, along the longer and stiffer side of the building the reduction of the free field motion reaches 50%. From the same figure one may see that the greater reduction of the response spectra values is achieved for shorter than 1.20 sec periods, compared to those of higher natural periods.

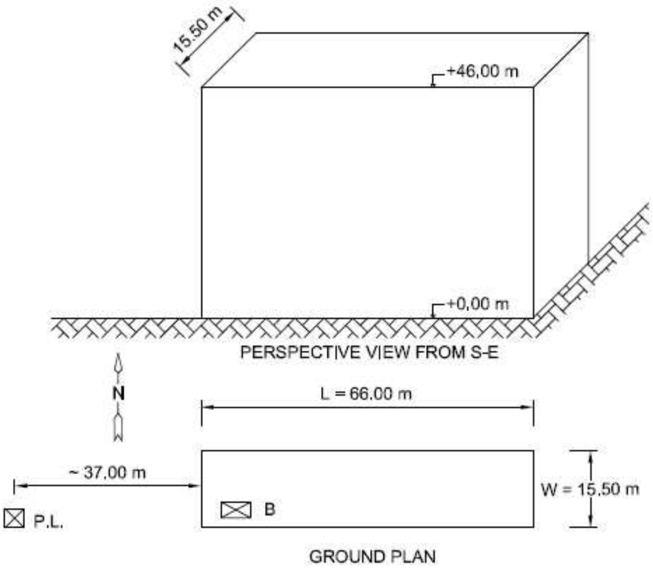


Figure 1: A stiff, large and strong structure of the Hollywood Storage building is instrumented at its basement (B) with a 3-D accelerograph. An other one is recording the free field motion at the parking lot (P.L.), after [1].

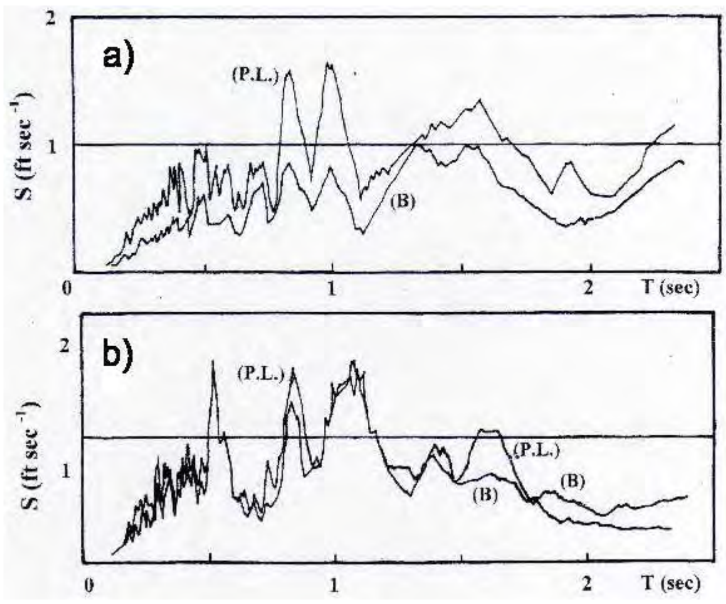


Figure 2: Velocity response spectra for ζ=0% damping in order to compare the ground motion records between the free field (P.L.) and the building response at its basement (B), for (a): along E-W stiff and longer direction and (b): along the N-S shorter and more flexible direction, after [1].

In [1] it is concluded that the reduction of the free field motion along the E-W direction is due to the length of the foundation compared to the half wave length of the seismic waves and, also, due to its high stiffness. Obviously the observed lack of damage is due to the high strength of the structure.

On the other hand, according to the regulations for earthquake resistant structures issued by the Ministry of Construction of U.S.S.R. up to the present [2], [3] long size foundation structures were considered as seismically vulnerable. For this reason limitations for the maximum allowable lengths of the structures and foundations were introduced. In those limitations the seismicity, the structural type and the quality of the material were introduced as parameters. For example, for non seismic regions the maximum length is 150 m independently from the structural type and quality of material. For the lowest seismic zone is 80 m for reinforced concrete frames and for the highest seismic zone the maximum allowable length is reduced to 40 m for simpler structural systems and of lower quality construction materials.

As it is explained, the building length limitations are set in order to avoid misleading or complicated calculations in order to evaluate high tensile and shear stresses in the foundation and to avoid horizontal tensional forces around the vertical axis of the structure. Those observations are documented after strong earthquakes, in which the longer the structure the larger was the damage.

According to [4] it was fully demonstrated by [5] the increase of stresses in a type of a large panel building which reached 1.35 MPa for the tensile stress and 0.74 MPa for the shear stress. In the same publication it was clearly shown that the increase of the length of a building usually will adversely affect its earthquake response. In the same publication it was not explicitly stated but it could be concluded that this is due to the phasing phenomenon of the seismic waves that propagate along the longitudinal direction of the building foundation. A further and more rigorous investigation of this phenomenon can be found in [6], in which it was proved, by using a two dimensional analysis, that important torsional vibrations were created in long buildings excited by seismic waves.

According to the above mentioned, a contradiction seems to exist between the recorded seismic observations at the Hollywood Storage building [1], and the equally well documented analytical calculations and field observations, on which the U.S.S.R. seismic codes since 1957 are based. Actually, there is not any contradiction, since the Hollywood Storage building is constructed out of solid reinforced concrete walls with very limited openings in the façade. At those years the type of structural systems as well as the material used in U.S.S.R., in general, should not be of the type and quality of the Hollywood building. It must be noticed that if the materials of that building were of lower quality, severe seismic damage would unavoidably occur. Therefore, based on the observations of both sides one may conclude that the longer is the foundation structure, the higher are the seismic stresses developed, while smaller becomes the resulting response – input motion to the structure.

In addition to the above mentioned, according to [7], a considerable decrease of the total seismic loads may be expected for building founded on strip foundations of continuous footings. This is compared to the case of the free field motion and the assumption that the ground particles during the earthquake vibrate “synchronously throughout the foundation”. For the first time the averaging concept was introduced by [8] according to the formula shown in Eq. (2):

$$\ddot{a}_b(t) = \frac{1}{L}\int_0^L \ddot{a}(x,t)\,dx \tag{2}$$

where,

$\ddot{a}_b(t)$ is the mean seismic acceleration in the body of the foundation as a function of time (t)

$\ddot{a}(x,t)$ is the point seismic acceleration at the free field along the foundation as a function of position (x) and time (t)

L is the total length of the foundation.

According to Eq. (2) the acceleration $\ddot{a}_b(t)$ will be the design acceleration for the whole structure, and its maximum value is certainly smaller that the maximum value of the free field one.

It is easily understandable that, due to the phasing phenomenon and / or the incoherency of the incident motion at the various points of the foundation body considerable dynamic strains (axial, bending, rotational and shear) may be developed in the body of the foundation. These strains and deformations are increased by a mutual increase of the length of the foundation, up to a certain magnitude. Therefore, it is logical to expect damage in inadequately designed and dimensioned foundations as this is mentioned in [9]. The calculation of those strains and the resulting forces is a rather difficult and time consuming procedure, some times based on various engineering approximation. In case that the various design parameters is difficult to be safely estimated it is better to set limitations related to the allowable maximum length of the foundation as a function of various parameters (seismicity, level of reinforcement and quality of the construction, geometry and mass of the above the foundation structure, stiffness of the foundation, soil conditions, expected motions from far or near field seismic sources).

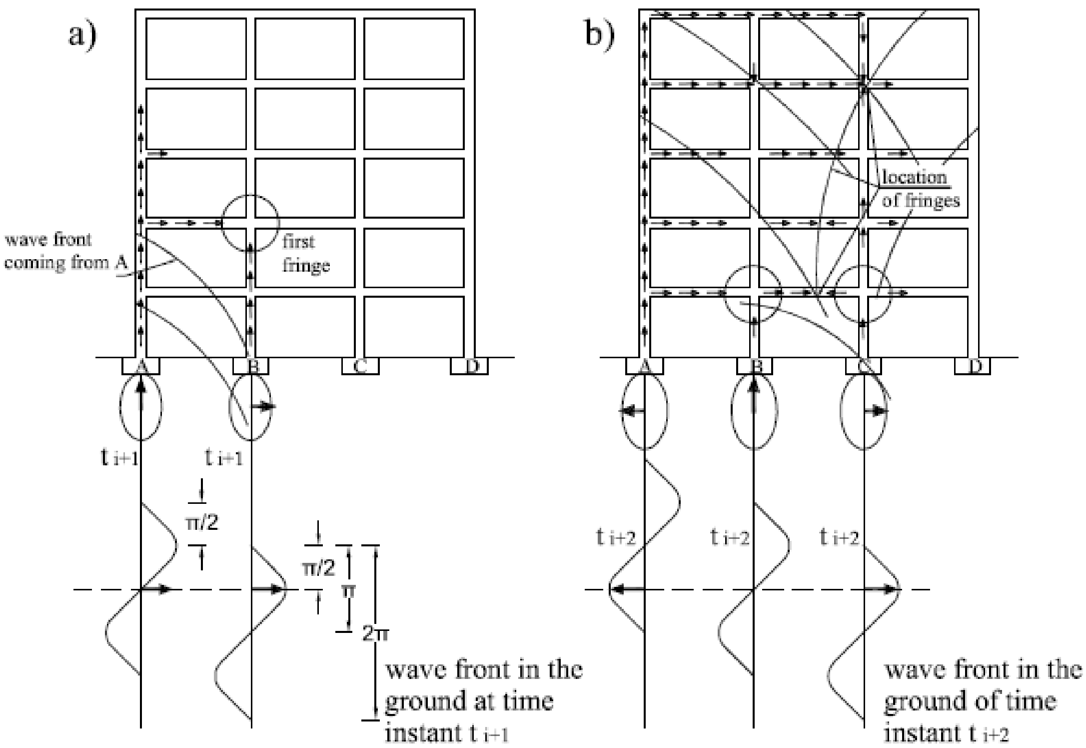


Figure 3: The input motion is inserted at discrete points of the foundation body (A), (B), (C) and (D): a) the earthquake motion, coming from the left of the figure, strikes the left hand parts of the structure first, while the other parts of the structure does not move until; b) the earthquake motion propagates through the foundations (C and D). The various input particle ground motions show a phase lag among themselves.

In the sketch of Fig. 3 the phasing phenomenon is attempted to be presented. The practical meaning of Fig. 3, may be illustrated with the following example. For a length of a building foundation of 45 m and an apparent wave velocity of 150 m/sec, the time required for the seismic motion to cross the foundation from point A to point D, is 45/150 = 0.30 sec. This means that for an oscillation period of the ground particles of the order of 0.60 sec the phase lag between points A and D can reach 180$^o$. In this case, the maximum amplitude of the resulting input motion will be roughly the 0.75 of the free filed one, according to Eq.(2).

A very important contribution to this subject was given by FEMA 440 [10], in chapter 8 where the base-slab averaging and embedment effects are presented and illustrated in its Fig.8.1. The placement of a rather rigid body between structure and ground absorbs all differences of the spatially variable ground motion and produces an averaging effect. The produced motion at the center of gravity of the slab is more uniform, it is filtered and it is less than the free filed motion in which localized maxima are usually observed and recorded, [10].

The relevant design methodology suggested by FEMA 440, [10], is followed in the arithmetic applications given in the relevant case studies presented in section 6 of the present communication.

Following the above mentioned, it is logical to propose that the engineering design of the foundation must be carried out according to the following approach: in its top side is the interactive building, while in its bottom side, the various incident waves with phase lag should be taken under consideration. These additional considerations must be done in order to maintain the current building design process as it is specified by the codes.

Also, it must be mentioned here, that in epicentral regions of mainly shallow focus earthquakes, it is not fully justifiable to use the term “surface wave velocity”, since the surface waves are greatly distorted due to multiple incident wave motions with various reflections and refractions. Thus, “surface waves” although created by the body waves emerging from the source, can not be normally propagated. Instead, the term “apparent surface wave velocity” should rather be used. The well known function $c = \lambda/\tau$ ($c$=wave velocity, $\lambda$=wave length and $\tau$=period) has actually not a pure physical sense, in epicentral regions of shallow focus earthquakes, since no one of these three parameters may belong to one and the same wave. According to various field observations in epicentral regions the apparent wave lengths are rather short compared to the far field ones, where the conditions are more normal and harmonic.

The particles of the ground in Rayleigh waves perform elliptical motions. The longer axis of the ellipse is along the vertical direction and the shorter one is along the horizontal one. The closer to the epicentre, the greater is the ratio between the vertical over the horizontal diameter of the ellipse.

In this way, the seismic motion that excites the foundation of a structure is a motion that results from the convolution of basic motions, plus the initial tectonic motion in the hypocentral region, which are projected on the three perpendiculars to each other axes, horizontal and vertical. This is exactly what accelerographs are recording. Of course, it is useless to say that these instruments record the 3-D true seismic motion of the base on which they are fixed, as for example the two quite different records obtained in [1] and shown in Fig. 2.

## 3 EARTHQUAKE RESPONSE OF BUILDINGS PROVING THE NECESSITY FOR LARGE, STIFF AND STRONG FOUNDATIONS

It is proved by abundant field observations and [11] the great variation of the ground motion and the resulting response of similar structures built on almost identical ground conditions and in a rather close distance among themselves. This means that the spatial distribution of the ground motion presents maxima and minima even along rather close distances, as already mentioned. Also, supporting the point of view of the U.S.S.R. code there are some cases in which the damage refers to structures of rather long dimensions.

According to [12] and as presented in Fig. 4, a “rhythmic” destruction occurred in a case of six similar multistoried buildings, built along a straight line, at equal distances one from the other. These six apartment buildings were identical to each other because they had been constructed with the same design drawings and the same materials, by the same constructor, with the same workmanship and at the same time. In addition to that and although there is no reason to assume any alternation of the soil conditions from one plot to its adjacent one, the damage and collapse occurred selectively at every second building. The difference of the earthquake response between one building and its neighboring is striking, i.e. from a total collapse to partial damage only.

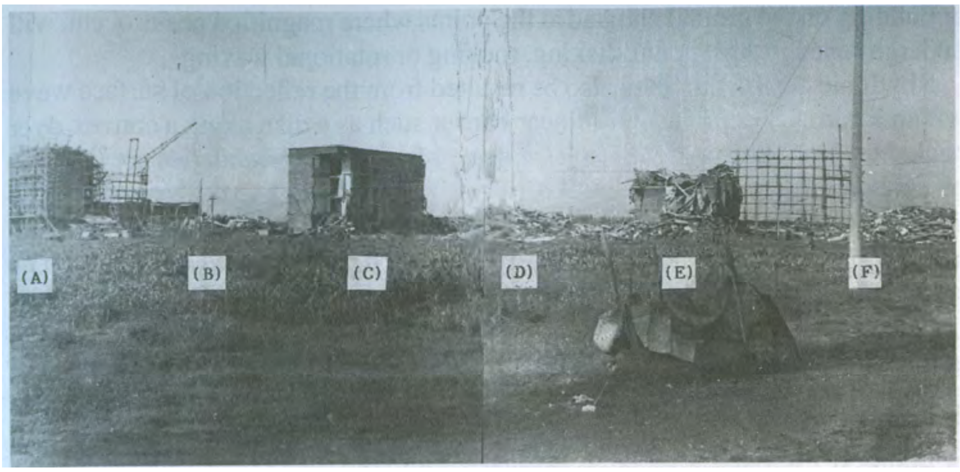


Figure 4: Taken from [12]: Six quite similar buildings responded quite differently from total collapse (B), (D), (F), or partial collapse (E), up to minor damage (A) and (C).

The above mentioned authors reported for the same earthquake, a similar to the above mentioned phenomenon in another location. In that case, six one – storied quite similar houses, located also along a straight line at equal distances suffered, as in the previous case, quite different damage. The first and fourth were safe, while all the rest totally collapsed.

In Fig. 5 are shown three buildings. At the left hand side the first building is located that suffered small damage. The second one suffered some heavier damage, while the third one totally collapsed. All three buildings were identical, constructed at the same time by the same constructor and with the same materials. According to a site inspection carried out by the first author, the soil conditions were all the same, for the three buildings, [13].

Quite interesting is, also, the case shown in Fig. 6, after the Dinar, Turkey, 1995 earthquake, [14]. The two out of five similar four storied apartment buildings totally collapsed, while the other three remained safe. The first author investigated the whole site and concluded that there was not any alternative difference in the underlying soil conditions. The total distance between the left hand side of the first and the right hand side of the fifth building is estimated to be about 120 m. Since all this takes place inside the epicentral region, the apparent surface wave length, is much smaller than the usual, as already mentioned. Therefore, it might be estimated that the apparent surface wave's length is of the order of 50 m. Also, it might be concluded that it was a one pulse damaging shock, that can only be explained with the convolution of the P waves.

In Fig. 7, another similar to the above mentioned rhythmic destruction phenomenon is presented after the Erzincan, Turkey, 1992 earthquake, in which, quite similar buildings within a small distance, with almost the same soil conditions along a street (Fig. 7a) presented a great variation of the observed damage: from almost no damage (Fig. 7b), up to total collapse (Fig. 7c), alternatively, [15].

Considering the above presented five representative cases of rhythmic destruction, it seems logical to support the hypothesis, from a pure engineering point of view, that if the foundations of each one of those building series were connected together rigidly and adequately strongly, collapse would certainly have been avoided. This is because the destructions occurred not due to the increased seismic vulnerability of the specific buildings that were destroyed, but due to the increased destructiveness of the input motion at the specific plots, see also, Eq. (2). Even more, the distance between the various plots is relatively small.

Figure 5: Rhythmic destruction. Three quite similar buildings in close distance to each other, on the same soil conditions, present a different response after the Corinthos, Central Greece earthquakes of 1981. The damage increases from the left hand side of the picture to the right hand side. The third building totally collapsed, [13].

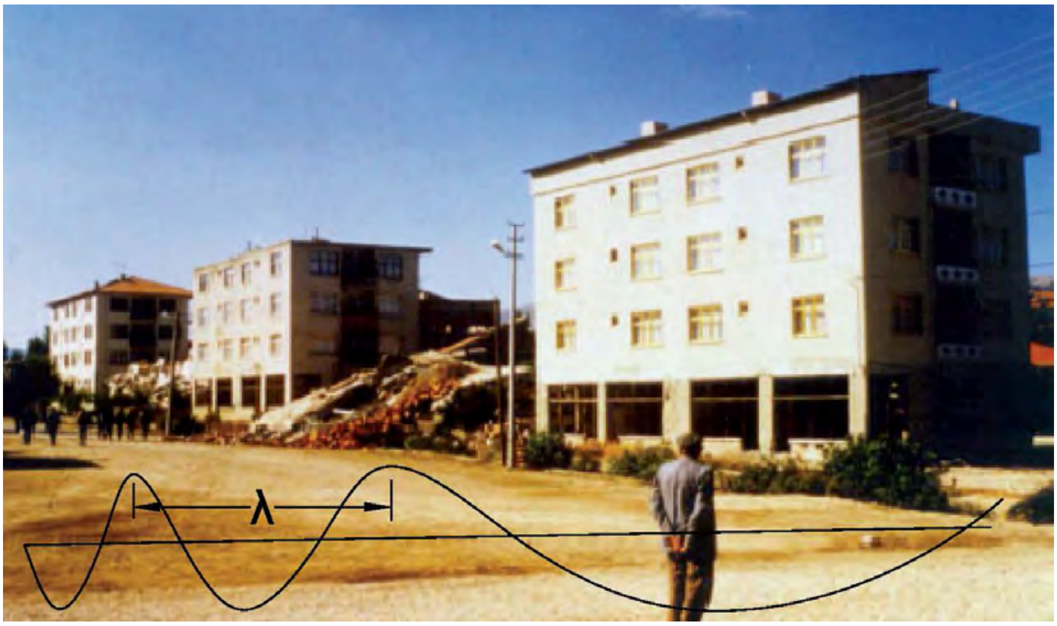


Figure 6: Rhythmic destruction. Two out of five quite similar structures founded on the same soil conditions and constructed by the same constructor totally collapsed after the Dinar, Turkey earthquake of 1995, [14].

The flexibility and strength of the foundation and, further, of the whole building on the vertical plane against the ground deformation due to surface seismic wave propagation, can be checked, during the design, with a simple calculation, as it is schematically shown in Fig. 8. If the stiffness of the foundation is not enough to withstand the subsidence (d) of the ground, the stiffness of the whole building along the vertical plane could be included. As a result, of these calculations the additional member forces and deformations will probably lead to a new dimensioning of the structural members.

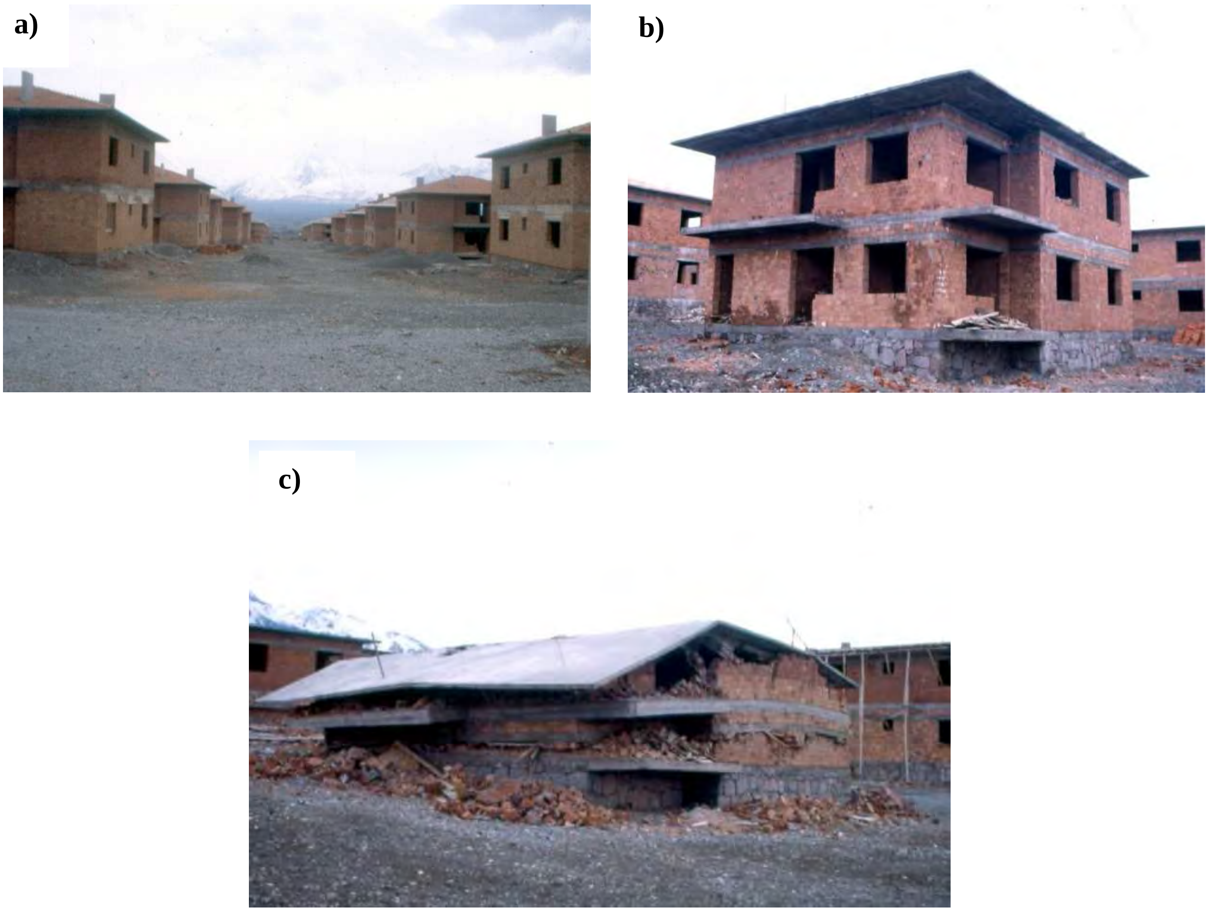


Figure 7: Rhythmic destruction. A series of similar buildings along one and the same street within a rather small distance one to the other (a), suffered damage of quite different intensity alternately. From almost no damage (b), up to total collapse (c). Erzincan, 1992 earthquake, by courtesy of A. Pomonis, [15].

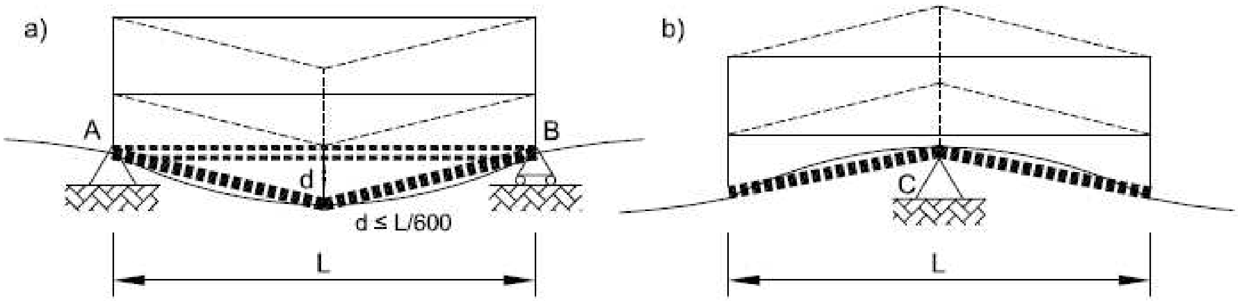


Figure 8: Simple deformational scenario on the vertical plane, may help the design against ground deformation due to the propagation of surface seismic waves. In order to avoid damage also to non bearing elements, the stiffness must be quite high, so that the flexure under its own weight is $d \leq 0.50\ell/300$; (a) a simply supported at A and B massive or Vierendeel beam; (b) a cantilever dual beam supported at C.

In Fig. 9 a partial collapse of an hotel after Egion, Greece, 1995 earthquake, [16] is shown. The building consists out of three parts statically separated by two separation joints of a negligible width. The damage of the remaining two blocks and the remaining part of the third block (Fig. 9b) is rather limited. The ground under the collapsed part did not present any permanent deformation (vertically or horizontally). The combination of the vertical components of the "surface waves" and of the body waves at the site of the collapse, with the flexible and weak foundation along the vertical plane could be an explanation of that type of partial collapse. The fact of the almost total lack of damage of the remaining parts supports this approach. During the same earthquake, many other similar partial collapse, while the rest of the building remained intact, were reported, [16]. The characteristics of the focal mechanism through the body P waves being propagated upwards might have a contribution to this result.

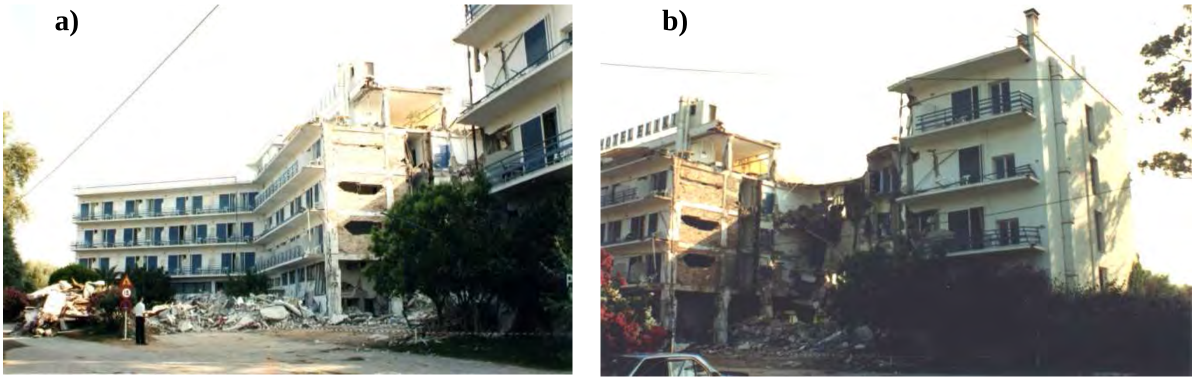


Figure 9: Typical example of partial collapse of buildings after the Egion 1995, M=6.7 earthquake. The rest of the building suffered limited damage, [16], [17].

An other collapse is shown in Fig. 10 after the Tokachi-Oki, 1968 earthquake. This phenomenon might be classified as a rhythmic destruction too, with the same conclusions as mentioned above: if the foundation structure were stiffer and stronger, the collapse would have been avoided.

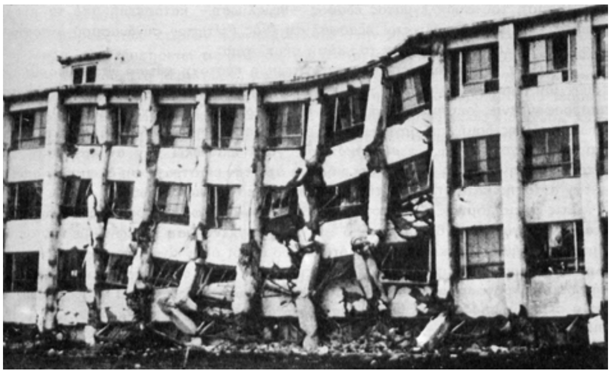

Figure 10: The elongated and without separation joints building suffered partial collapse after the Tokachi-Oki earthquake of 1968, [9].

As a partial practical conclusion of the present chapter is the fact that once the forces and deformations in the foundation body of large dimensions are successfully confronted, the seismic response of the whole structure, or of a group of structures that are bound together through a foundation body, will be less, compared to the case with a foundation of smaller dimensions. According to [18] existing structures separated by construction joints could increase their earthquake resistance just by elastically connecting the above the foundation parts of the structure.

## 4 SEISMIC ARRAYS AND COHERENCY. A BRIEF REVIEW

The subject under consideration is obviously based on the variation of the input motion along the foundation body. This variation is due to the amplitude, the phase difference and the shape of the seismic waves that travel along the foundation of a structure. A good measure of this variation is the coherency between two motions at points i and j. This coherency according to [19] is expressed in the frequency domain by:

$$\gamma_{ij}(\omega) = \frac{S_{ij}(\omega)}{\sqrt{S_{ii}(\omega) S_{jj}(\omega)}} \tag{3}$$

where:
$S_{ij}$ is the cross spectral function and
$S_{ii}$ or $S_{jj}$ are the auto spectral functions.

The coherency is a normalized complex function, [19]. The coherence is identified as the square of the modulus of the coherency i.e. the quantity $|\gamma_{ij}(\omega)|^2$, which as a real number, may vary from zero (non coherency) up to one (complete coherency).

A great help to the subject under consideration was given by the analysis of the strong motion array records obtained from the various strong motion arrays as it is presented by [7], [19], [20], [21], [22], [23] and many other researchers. Besides this, the practical interest lays on the establishing of time histories of strong motion for the seismic analysis of structures with multiple supports, long structures e.tc..

The recorded motions by the SMART 1 array, [19], are dominated by “incoherent energy” at frequencies above 2 Hz when they are averaged over a large distance.

On the other hand as it is concluded by [21] the peak cross correlation values (P) of the accelerogramms, of the El Centro Differential Array may be approximated by the expression:

$$P = e^{-\frac{0.00035 \times L \times c}{\lambda}} = e^{-0.00035 \times L \times f} \tag{4}$$

where:
L is the distance in (m) between the two stations or the foundation length,
$\lambda$ is the apparent wave length in (m),
c is the apparent velocity of the wave propagation in (m/sec) along the ground on an adequate depth below the foundation, and
$f=c/\lambda$ is the apparent frequency in (Hz) of the particle ground motion ($T_g=1/f=\lambda/c$).

Based on Eq. (4) it is concluded by [21], that the frequency (f) and the length of the foundation (L), as far as the reduction of the input seismic motions is concerned, are interchangeable quantities. This means that we can get the same values of cross correlation either by long lengths of foundations or by high predominant frequencies of the seismic ground motion.

Eq. (4) is graphically presented in Fig. 11 for a variety of values of parameters L and f.

A considerable contribution must be attributed to the research work by [24] to a better understanding of the earthquake response of rigid foundations to spatially varying ground motion excitations by analytical approach, producing closed mathematical forms including all basic parameters. In that research work the authors used a massless rigid foundation bonded to a viscoelastic half space, excited by a wave passage and they found reduction of the translational component of the motion of the beam at higher frequencies (>1Hz). Also, they found that torsional and rocking components of the motion were created. They noted that these effects highly depend on the degree of spatial incoherence of the free field motion along the foundation.

It is worth mentioning in this chapter that the in depth study of the effects on structures of the spatially variation of the seismic motion, has very much advanced with the help of adequate analytical evaluation of mainly digital recordings obtained from strong motion arrays on the surface of the ground and in bore holes as well. These well known devices are installed and functioning all around the globe since many years. Eminent research efforts are carried out in addition to the ones mentioned above, as for example: by [22] in which the "differential motion $\Delta_{jk}(t)$", also other physical parameters are introduced and studied as well; by [23] using the strong motion data recorded by dense arrays described the spatial variability of the ground motions and stochastic models were produced using the coherency quantity; by [25] in which the effects of the spatially variation of the seismic input motions on a two – span indeterminate beam are parametrically investigated.

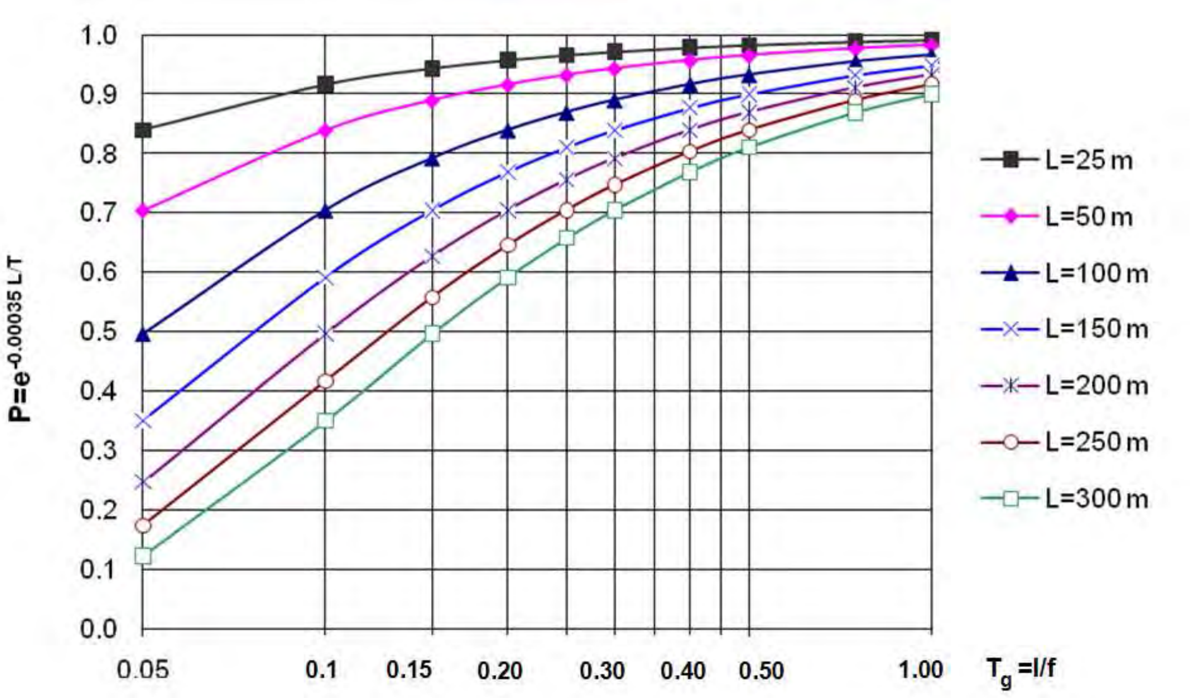


Figure 11: A graph of the function $P=e^{-0.00035\ L/Tg}$, using a variety of values of parameters L and f, (see Eq. 4).

On the other hand, it must be mentioned here, that all these observations might be much more pronounced in epicentral regions of shallow normal or reverse focal mechanisms. This could be attributed more to the fact of the low coherence of the emerging seismic motions, due to multiple reflections and refractions, rather than to the phase lag of the travelling “surface waves”.

In all the above mentioned references it is clearly stated that there is a phase lag among the incident seismic motions along a sizable building foundation as well as other phenomena that lead to reducing the seismic excitation of the structure. On the contrary, [26] the analysis of the earthquake records in 57 building foundations of various sizes, did not verify the decrease of the motion as a function of the size of the foundation body rather than its rigidity. An in depth discussion of the findings of this work should be very interesting, but it is beyond the scope of the present communication.

## 5 A PARAMETRIC EVALUATION OF THE RESPONSE OF A MASSLESS RIGID FOUNDATIONS BEAM

The parameters studied are a combination of the length of the beam, a variety of the traveling velocity of the acceleration time history and a variety of two types of time histories, [17].

With the advancement of computers and computer codes in Earthquake Engineering it became feasible to carry out quickly and reliably series of parametric analyses under various assumptions in order to examine complex phenomena as the one under consideration.

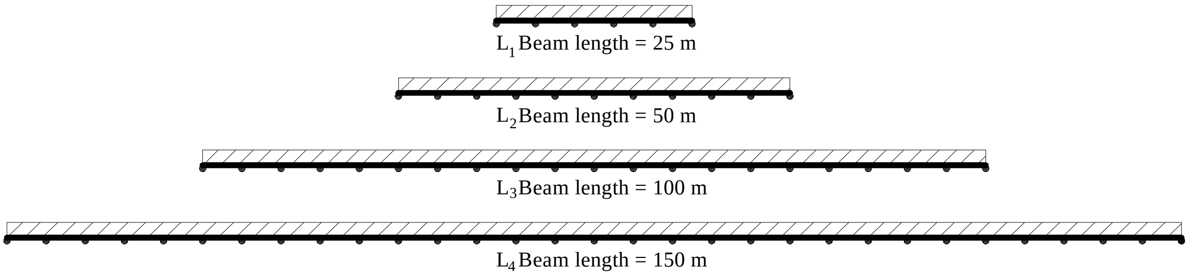


Figure 12: The various beams and points with spring supports – input motions used in the parametric analyses, [17].

As it is shown in Fig. 12 for the parametric study a massless beam out of reinforced concrete with various lengths of $L_1$=25 m, $L_2$=50 m, $L_3$=100 m and $L_4$=150 m as it is shown in Fig. 14, has been used. The rigidity of the beams is a function of their cross section 0.50 m width x 2.00 m height. The beams rest on elastic springs with a stiffness of:

$$K_h = 4.0\ MN/m \text{ and } K_v = 5.0\ MN/m \tag{5}$$

where
$K_h$ is the horizontal stiffness of the springs
$K_v$ is the vertical stiffness of the springs
their modulus of elasticity is E = $2.90 \times 10^7$ kPa and the poison ratio ν = 0.20.

The transient dynamic analysis of the computer code ABACUS has been used throughout the present study, as well as other programs of every day use for minor calculations (response spectra, reinforcement).

The input motion is defined as follows. The time history of an accelerogramm has been selected. The whole accelerogramm is propagated with a certain velocity along the beam and the excitation is applied through the above mentioned supports. The velocity of the propagation of the accelerogramm is selected for the various parametric calculations as follows:

$$c_1 = 100\ m/sec, c_2 = 200\ m/sec, c_3 = 400\ m/sec \text{ and } c_4 = 600\ m/sec \tag{6}$$

For example, for the case of the beam $L_3$=100m and the velocity $c_3$=400m/sec its last support starts to be excited at a time lag equal to Δt=0.25 sec after the excitation of the first support of the beam. The coherency of the two input motions in the case under consideration depends also on the predominant period of the motion, as already mentioned. And according to Fig. 11, the higher is the predominant period the higher is the coherency.

For the needs of the present investigation, the following two input time histories have been selected: The first one is an artificial accelerogramm, the response spectrum of which fits to EC-8 Type 1, ground class A, corresponding to a design acceleration value of $a_g$=0.24g, of a total duration of 2.4 sec, as it is shown in Fig. 13. The second one is the Edessa N – S component due to Griva northern Greece, 21 Dec. 1990, M=5.9 earthquake with max ä=0.10g, recorded by the Greek National Institute of Engineering Seismology and Earthquake Engineering, ITSAK. The epicentral distance is 31 km and the depth of the hypocenter is H< 15km. The record is on rather soft soil conditions.

Therefore it is logical to measure predominant periods between 0.5 sec and 0.7 sec. The component used, is shown in Fig. 14.

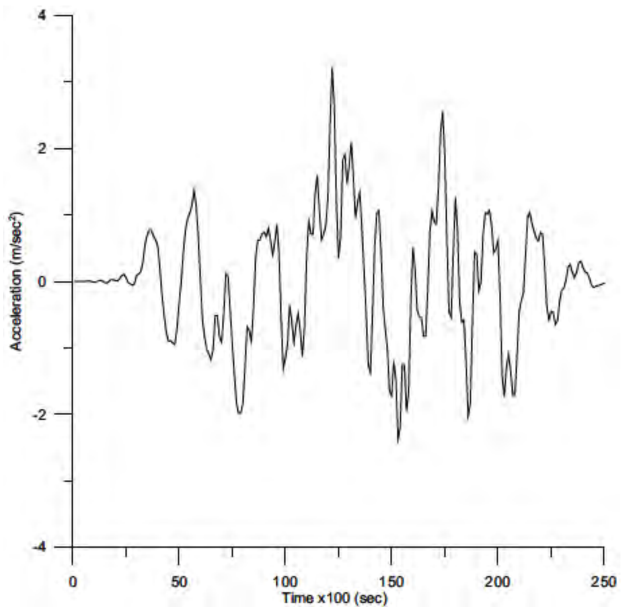


Figure 13: Artificial accelogramm fitting to EC-8 Type 1 response spectrum, ground class A, design acceleration 0.24g.

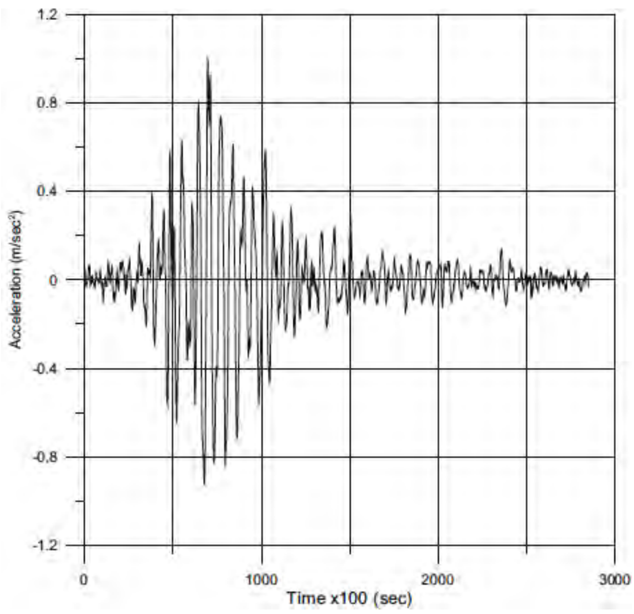


Figure 14: The N – S component of the Edessa, 21 Dec. 1990, M=5.9, northen Greece, Griva earthquake recorded by ITSAK.

In total 34 responses spectra have been calculated (4 seismic wave velocities x 4 beam lengths x 2 types of ground motions = 32 cases + 2 for the free field motions). The obtained horizontal motion at the center of gravity of each beam was used in order to calculate its response spectra for 5% damping ratio. Here, for the sake of brevity, the most representative response spectra are presented. It was proved, as already shown in [6], that the determining parameter among the various response spectra is the ratio $\tau$=L/c sec. Therefore, in Fig. 15 these response spectra, for $\tau_1$=0 sec (is the corresponding to the free field motion), $\tau_2$=25/200=0.125 sec, $\tau_3$=25/100=0.25 sec, $\tau_4$=100/100=1.0 sec and $\tau_5$=150/100=1.5 sec, are depicted for the artificial ground motion.

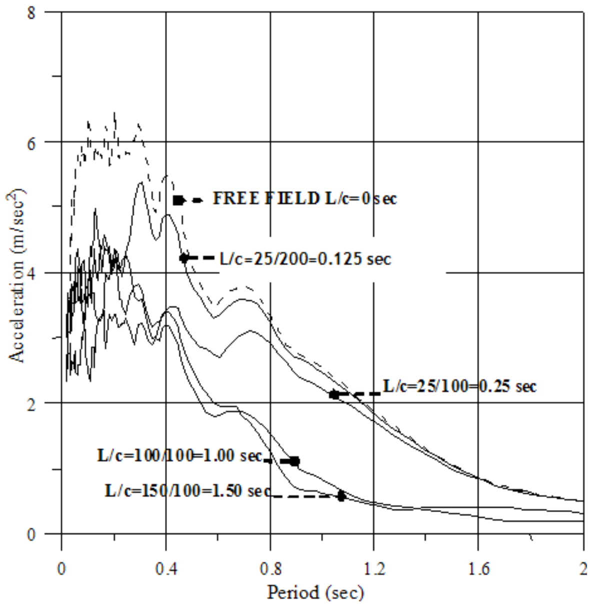


Figure 15: Response spectra at the center of gravity of the beams for the artificial ground motion fitting to EC-8 Type 1, for various τ(sec)=L/c=beam length/wave propagation velocity, and 5% damping ratio.

In Fig. 16 the response spectra, for $\tau_1$=0 sec, $\tau_2$=25/100=0.25 sec, $\tau_3$=150/400=0.375 sec, $\tau_4$=100/100=1.0 sec and $\tau_5$=150/100=1.5 sec, are depicted for the Edessa ground motion.

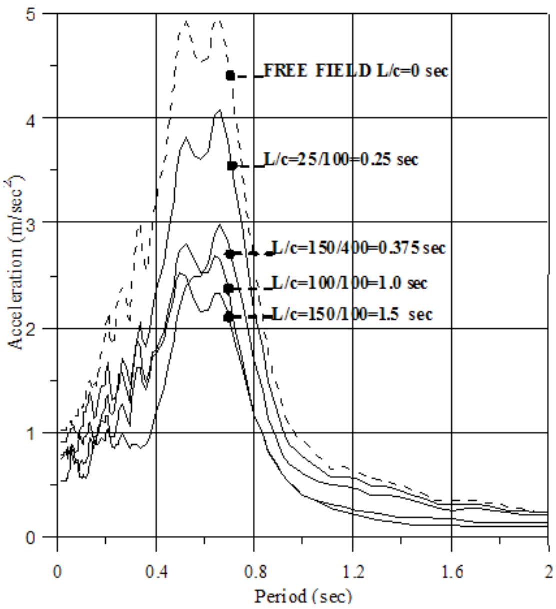


Figure 16: Response spectra at the center of gravity of the beams for the Edessa earthquake, for various τ(sec)=L/c=beam length/wave propagation velocity, and 5% damping ratio.

Although the earthquake excitation records are quite different in shape, duration, and frequency content from the engineering point of view and the resulted response spectra are quite different in shape and magnitude, their resulted reduction is in both cases obvious.

For the discrete set of natural periods of the one DOF oscillator: 0.1, 0.2, 0.4, 0.6, 0.8, 1.0, 1.2, 1.4 and 1.6sec, the ratios of the various response spectra shown in Figs 15 and 16 over the respective values for the free field ground motion are depicted in Fig. 17, always for 5% damping ratio.

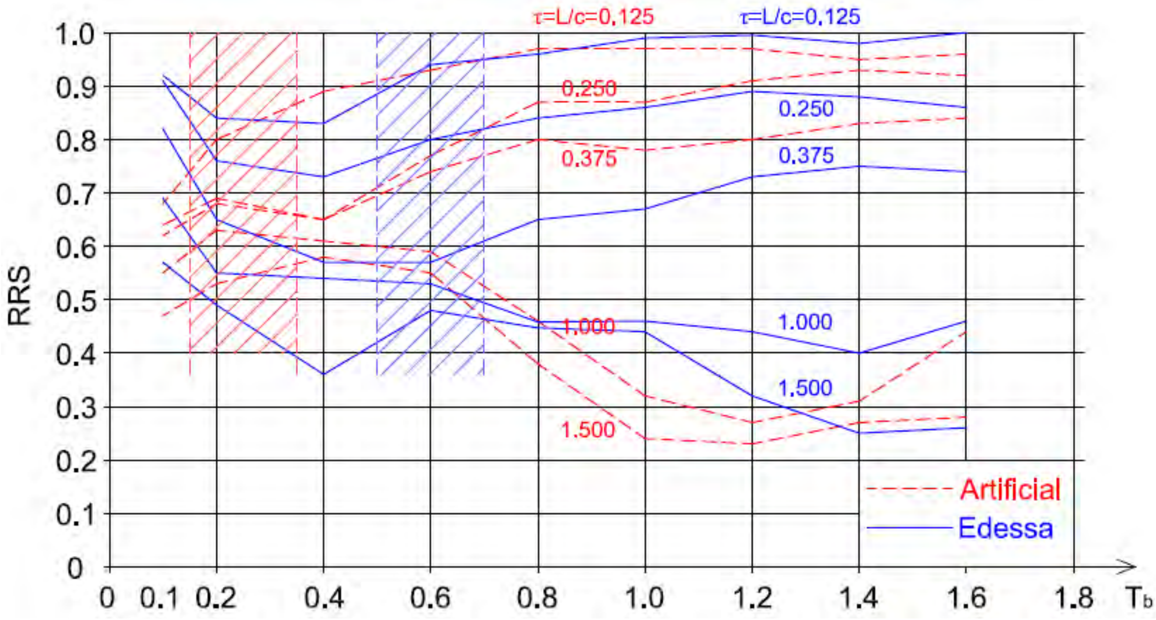


Figure 17: Ratios of Response Spectra (R.R.S.) obtained at the center of mass of the beams over the respective free field ones, all for 5% ratio. The hatched areas indicate zones where the maximum values of the spectra occur.

A lot of results and comments may be drawn based on the findings of Fig. 17. For example the spectral values reduction is extremely high for values of $\tau$=L/c $\geq$ 1.0 sec, for both types of earthquakes, as it is expected.

In Table 1 the ratios of the maximum values of the various response spectra obtained at the beam's center of gravity over the maximum values of the free field motion are shown.

| τ =L / c (sec) | 0.000 | 0.125 | 0.250 | 0.375 | 1.000 | 1.500 |
|---|---|---|---|---|---|---|
| Artificial | 1.000 | 0.840 | 0.760 | 0.680 | 0.630 | 0.580 |
| Edessa | 1.000 | 0.940 | 0.840 | 0.600 | 0.570 | 0.510 |

Table 1: Ratios of Response Spectra (RRS) of the obtained at the center of gravity of the beams maximum response to the free field ones.

It must be mentioned that the max $S_a$ values shown in Table 1, do not occur at the same periods of the one DOF oscillator. This means that the interference of the stiff foundation grillage does not only result to the reduction of the amplitude of the free-field motion but also to the modification of its frequency content. The latter is more pronounced in the case of the artificial ground motion which is characterized with a higher frequency content that the Edessa one.

The calculated stresses are quite high at the beams of Fig. 12 The longer the beam the higher the stresses, but to a certain limit. For example, the stresses for $L_4$=150 m are almost the same with those yielding for $L_3$=100 m. The latter ones are a little bit higher than those for $L_2$=50 m. More specifically, for the $L_3$=100 m beam length, the maximum calculated stresses are the following: Homogeneous axial stress, about 1.0 MPa (tensional or compressional). Axial stress due to the bending only (without the axial stress) 1.5 MPa and shear stress about 0.6 MPa. These stresses calculated after intergration, over the respective cross section area of the beam (part or whole) give the corresponding forces which finally lead to the appropriate dimensioning and reinforcement of the beams. For example, the main reinforcement that will be put equally distributed all around the cross section of the beam 50/200 is 36Ø22 of which

6Ø22 at the top and 6Ø22 at the bottom side of the beam. Stirrups 2xØ12/15. The geometrical percentage of the main reinforcement is about 14‰.

## 6 CASE STUDIES OF SOME MONUMENTAL STRUCTURES

It is obvious that the presented methodology may be applied to existing structures by creating long, rigid and strong beams in close contact with the existing foundation. These beams may function as a two – dimensional close orthogonal grillage in order not only to reduce the input seismic motions but also to comply with the plan dimensions of the building. Besides that, the mentioned beam grillage of large dimensions increases, also, the necessary resistance against torsional phenomena that may be created by the non uniform motion of the ground and by the torsional response of the structure itself during strong earthquakes. Also, the sail grillage, in conjunction with the above standing structure, will withstand the ground deformations of the traveling “surface waves” if adequately designed. The so designed and constructed new foundation body will possess a minor kinetic state compared to the underlying ground. It must also be taken under consideration that the new foundation body possesses a higher mass density than the underlying ground. According to well known principles of physics, the travelling seismic waves from a less dense (ground) will be easier reflected on the denser grillage out of R/C beams. Therefore, a transition – interface zone is naturally formed between the moving ground and the less moving and/or out of phase moving foundation grillage. The height (h) of this transition zone is determined in combination with its quality. A useful parameter is the developed shear deformation $\gamma=\Delta d/h$, that must not create excessive permanent – plastic differential deformations (**Δd)**. The latter one is the maximum difference between the two maximum displacements: do (the original of the ground) and d (the resulting in the center of gravity of the grillage). The thickness of this transition – interface zone, might be of the order of 30 to 60 cm. This is in agreement with the proposed method presented in [27]. This transition – interface zone is a path along which an amount of input seismic energy is going to be absorbed. For this reason, it is indicatively shown the respective sketches of the case studies a gradual improvement of the underlying ground from the base of the zone to the base of the grillage, in the case of soft ground. If the ground is harder, the transition – interface zone might be of smaller thickness and could be constructed out of well compacted, by vibration, angular rubbles of almost equale dimensions (about 5 to 7 cm), as it is also shown in some sketches of case studies.

Some basic parameters for the design, besides the above mentioned, are the seismicity and the characteristics of the anticipated earthquake ground motions, the soil conditions, the relief of the ground surface, the characteristics of the structure to be protected, its dimensions e.tc. It is of basic importance to know, if a basement is needed, what is the available area of the plot of the structure and its geometry. It is self – evident that all the life–line connections of the building with the public utility networks must be constructed resiliently in order to continue functioning even after strong earthquakes during which, considerable deformations between the building and the surrounding may occur.

As it is shown in the sketches of the various study cases that are following, there are three types of the new foundation elements that are provided in each intervention. To the first type belong the main load bearing beams that are forming the grillage. To the second type belong the secondary beams that are provided in order both beams to confine the masonry members of the structure. The two types of beams are connected together, usually with cross beams 40/40 cm at horizontal distances of about 1.50 m one to the other, [28]. The confinement of the masonry elements with these beams is indispensable in order to ensure a uniform response of the whole structure given that under the foundation body of the existing structure there is not any transition – interface zone. On the other hand we want to increase, us much as possible, the areas of those zones. An acceptable lower threshold of the total area of the transition – interface zones is to be at least 2.5 times that of the total cross sections of the load bearing masonry walls of the structures. As far as the calculation of the total area of the transition - interface zone is concerned it will be taken under account only those zones that are in close vicinity to the walls as mentioned above. For this reason, the third type of elements constituting the grillage is provided, which consists of a rather thick (25 cm) R/C slab. Those slabs are functioning as retaining – continuous interconnecting structural elements between beams at their bottom. Clarifications are provided in the details of the sketches. If a non structural element is founded on the ground needs to be fixed to the load bearing walls by an adequate horizontal diaphragm.

In this way we are absolutely sure that all bearing and non bearing parts of the existing structure will follow the response of the grillage in case of an earthquake.

For a preliminary design of the case studies presented below two methodologies are followed. The one is based on the procedure already exposed in the present communication and the other follows the FEMA 440, chapter 8 methodology [10]. In general, we prefer to have a rectangular shape of the foot print of the grillage. Nevertheless, if the dimensions of the grillage are large enough, i.e. several times the length of the anticipated seismic waves, as this was realized at Ano Liossia city for the reconstruction of 1,500 houses after the destructive earthquake of 1999 and as it is presented in [29], the said requirement for a rectangular shape of the foot print of the new foundation, is not necessary.

### 6.1 Case study No 1

In Euboea, Greece, the Prophet Elias church was "strengthened" by applying the already exposed methodology. The plan of the church with the proposed new bean's grillage with dimensions 75m x 70m is shown in Fig. 18a, while a construction detail is shown in Fig. 18b.

The mean plan dimension of the realized grillage is $L = \sqrt{L_x \times L_y} = 72.50\text{m}$.

The ground quality is medium, of a predominant natural period $T_g = (T_{1g} + T_{2g})/2 = (0.15 + 0.60)/2 = 0.375\,\text{sec}$ (according to EAK-2000 code), and the shear wave velocity is c=300 msc$^{-1}$. The quantity $\tau = L/c = 72.5/300 = 0.24\,\text{sec}$.

These ground conditions are more consistent to the artificial earthquake fitting to EC-8 Type 1, ground class A. Using Table 1 and after interpolation, one may find RRS=0.77 for the maximum spectral values. The natural period of the building fixed on rigid base is $T_b$=0.25 sec. By using the respective curves shown in Fig. 17, and after double interpolation for $\tau$=0.24 sec and $T_b$=0.25 sec one may find $RRS_b$=0.69.

The thickness of the transition – interface zone can be calculated with the use of adequate computer codes. But for a preliminary design one could estimate it as follows: The achieved reduction of the motion between free field and R/C grillage is of the order of 30%. In engineering terms of the design input acceleration, this means 0.30x0.24g=71.0 cmsec$^{-2}$.

With a mean predominant period $T_g$=0.375 sec, the differential displacement is $\Delta d=71.0/(2\pi/0.375)^2=0.25$ cm. The normalized shear deformation that can be developed within the transition zone without creating accumulative shear deformations for the specific material quality and compaction, is $\gamma=5.10^{-3}$, [30]. Therefore, $h=\Delta d/\gamma=50$ cm.

On the other hand, following FEMA 440, chapter 8 procedure for the present case study, we find for the foot print of the foundation dimensions 70x75 m$^2$=230x249 ft$^2$.

Effective foundation size $L = \sqrt{230 \times 249} = 239\text{ft}$.

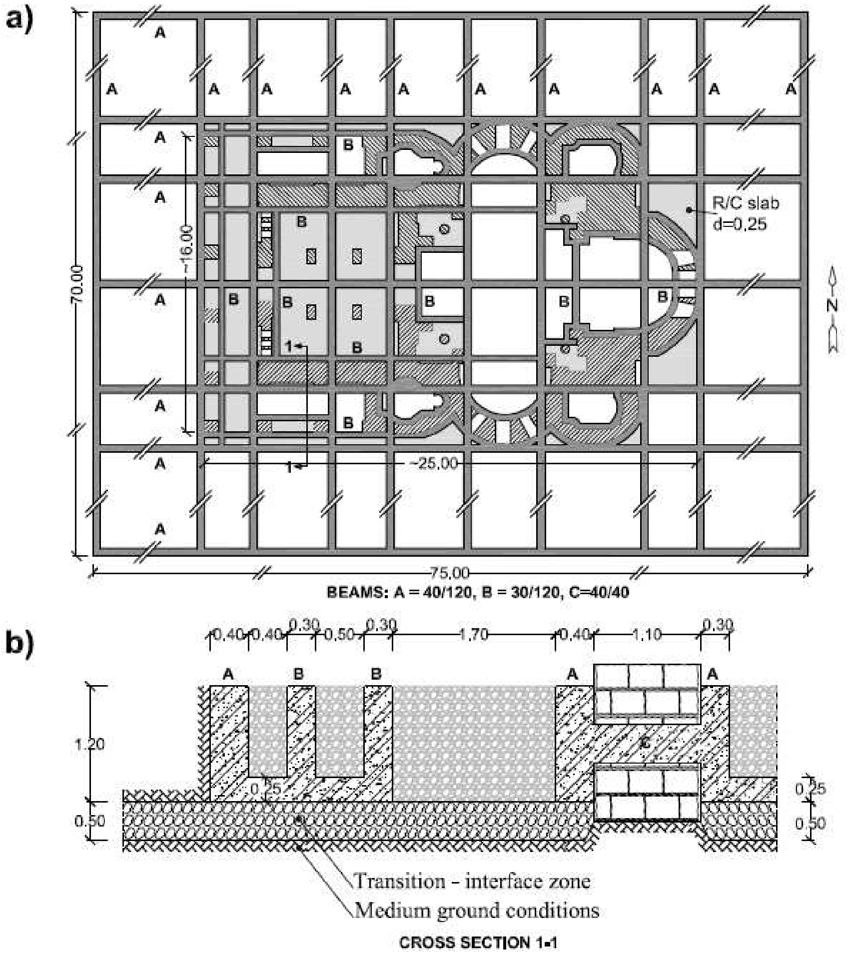


Figure 18: The Prophet Elias, a church at Euboea "strengthened" by applying the methodology exposed in the present communication, (a) general plan; (b) detail at cross section 1-1.

The Ratio of the Response Spectrum (RRS) due to base-slab averaging, according to FEMA 440, chapter 8, formula (8-1), for $T_b$=0.25 sec:

$$RRS_{bsa}=1-(L/T_b/14{,}100)^{1.2}=1-[(239/0.25/14{,}100)^{1.2}=1-0.27=0.73 \quad (7)$$

An other parameter being considered by FEMA 440, closely related to the subject under consideration is the effect of the foundation embedment, treated under the relationship:

$$RRS_e = cos[\frac{2\pi e}{T_b nc}] \quad (8)$$

where:

e is foundation embedment (ft)

c is the shear wave velocity at a depth L under the foundation ($ftsec^{-1}$)

n is the shear wave velocity reduction factor for the design ground acceleration, according to FEMA 440, Table 8-1.

The total foundation's grillage thickness from its base up to the surface of the ground is about e=9 ft. The shear wave velocity reduction factor for the expected PGA=0.24g (according to the 2000 Greek code for design and construction of earthquake resistant structures - medium seismic zone) is n=0.68.

Therefore applying Eq. (8), and for e=9 ft, $T_b$=0.25 sec, n=0.68 and c=300 $msec^{-1}$=984 $ftsec^{-1}$, one may find $RRS_e = \cos[\frac{2\times\pi\times 9}{0.25\times 0.68\times 984}] = \cos(0.34) = 0.94$.

The total RRS value is:

$$RRS = RRS_{bsa}\ x\ RRS_e = 0.73\ x\ 0.94 = 0.69 \qquad (9)$$

Comparing the obtained two values: a) Using the present methodology: $RRS_b$=0.69, and b) Using the FEMA 440: RRS=0.69, one may conclude that the coincidence of the resupts between the two approaches is quite incidental taken under consideration the abundance of parameters involved in a so complicated phenomenon; we could just accept a good agreement between the results of the two approaches. Nevertheless, the achieved reduction of the earthquake design motion is of the order of 30%.

### 6.2 Case study No2

Near the city of Alexandroupolis the church of St. Panteleimon was "strengthened", by applying the already exposed methodology. The problem in the case under consideration is that the available area of the plot of the church is rather limited along N-S direction. The finally applied dimensions of the grillage are 65.0 m x 45.0 m, as shown in Fig. 19a. In Fig. 19b a detail of cross section 1-1 is shown. The ground is soft with a predominant period $T_g$=(0.20+1.20)/2=0.70 sec, (according to EAK 200 code). The shear wave velocity is c=150 $msec^{-1}$. The mean length of the foot print of the grillage is $L = \sqrt{65\times 45} = 54.0m$. The quantity τ=L/c=54/150=0.36 sec.

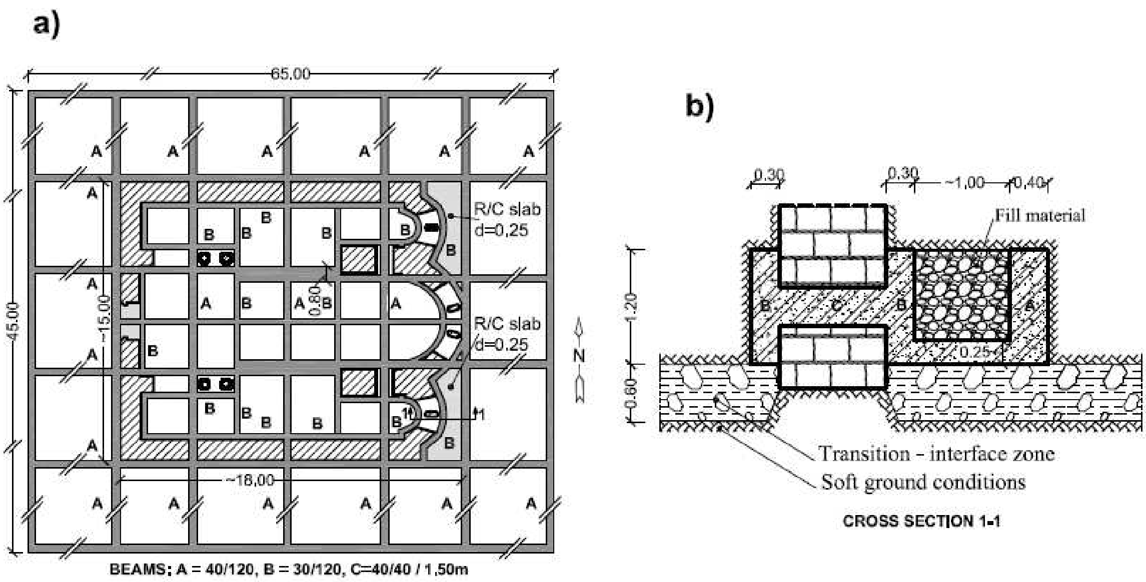


Figure 19: The St. Panteleimon church close to Alexandroupolis, "strengthened", by applying the methodology exposed in the present communication; (a) general plan; (b) detail at cross section 1-1.

The anticipated type of motion is more consistent with the Edessa earthquake time history. The natural period of the fixed on the base structure is $T_b$=0.23 sec. By using the respective data in Fig. 17 we may obtain $RRS_b$=0.62.

On the other hand, using the FEMA 440 procedure we may find, with an embedment e=7 ft $RRS=RRS_{bsa}xRRS_e$=0.79x0.84=0.66.

The achieved reduction of the input motion is of the order of 35% as a mean value.

## 6.3 Case study No3

The plan of the private monumental building in Epirus is shown in Fig. 20, with the beam grillage as designed, based on the present communication. The dimensions of the R/C grillage is 45.0 m x 45.0 m.

The ground conditions are soft to medium, $T_g$=(0.2+0.8)/2=0.5 sec (according to EAK 2000 code). The shear wave velocity is c=200 msec$^{-1}$. The parameter τ=L/c=45/200=0.225 sec. The anticipated type of motion must be something between the Edessa and the artificial ground motions. The fundamental period of the building fixed on its base is $T_b$=0.30 sec. The building possesses a cellar. The whole embedment height is e=12 ft.

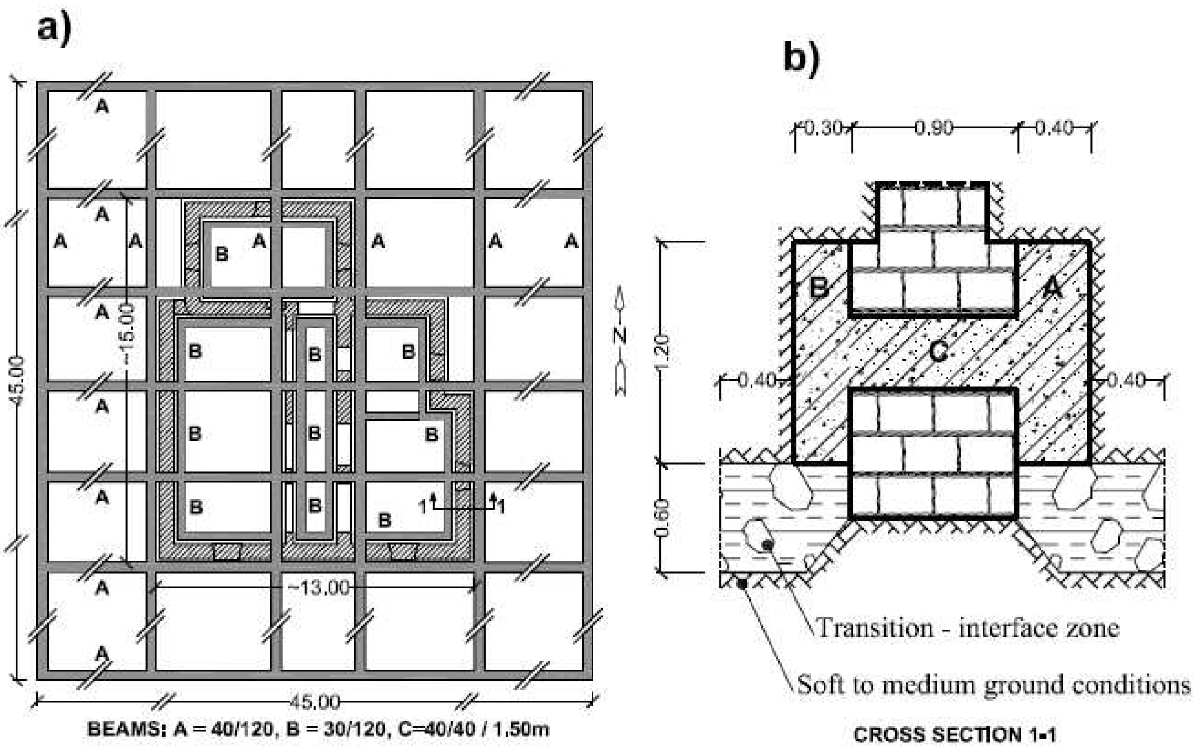


Figure 20: A monumental private building in Epirus, "strengthened" by applying the methodology exposed in the present communication; (a) general plan; (b) detail at cross section 1-1.

The RRS for the Edessa earthquake is $RRS_{bED}$=0.76.

The RRS for the artificial earthquake is $RRS_{bART}$=0.71.

Then by an engineering judgment, considering the mean value we obtain: $RRS_b$=0.74.

By using the FEMA 440 procedure, we may find:

RRS=$RRS_{bsa}$x$RRS_e$=0.88x0.85=0.75. The reduction of the anticipated input seismic motion is of the order of 25%.

Other study cases related to monumental and new structures are presented in [17].

# 7 CONCLUSIONS

- A sustainable seismic input reduction system is presented, in which there are not employed any kind of seismic isolation or seismic input energy absorbing devices. The methodology is based on simple principles of engineering mechanics that can be adjusted to any site condition.

- The system may be applied to monumental structures, such as churches, houses and existing buildings in general, in which an intervention to the superstructure might be difficult or even impossible, due to Venice Charter or other international or national relevant treaties. On the other hand interventions in the foundation body could be carried out in a more effective and conventional way.
- The resulted seismic input motion reductions after the application of the system to several case studies were checked following the FEMA 440, chapter 8 procedure. The obtained differences between the two approaches were either null or negligible.
- The reduction of the input motion following the presented methodology may reach 50% of the free filed motion. Therefore, this finding could be used by various means, one of which is as the following example: if the existing earthquake resistance of a monument is keeping behind the nominal by 50%, it might be possible to make it earthquake safe by a structural seismic resistance upgrading of 10% only, and by applying the present methodology that will result to a 40% reduction of the seismic input motion. It is self understood that the required dimensions for the foundation grillage will be available in the plot of the structure.
- The proposed system should be also applied to structures that may possess the adequate earthquake resistance, but they must be secured against future much stronger earthquakes.
- The main parts of the system are: a) an orthogonal grillage of R/C beams that must be longer than the half of the anticipated seismic wave length. These beams are stiff and strong enough in order to withstand the resulting member strains and stresses. The beams must be in structural contact with the existing foundation of the structure; b) a transition – interface zone. In this zone the differential motion between the grillage and the ground must be compensated without creation of permanent and uncontrollable deformations. Using simple engineering mechanics the thickness and the quality of this zone may be defined. As a general guideline a thickness of the order of 30 to 60 cm might be enough for an appropriate material of the zone and adequate dynamic compaction of it.
- In the most of the cases various problems in the foundation body of monumental structures are quite often reported. In those cases the proposed system might be proved extremely effective for curing the said problems of the existing foundation.
- The longer are the dimensions of the grillage, the higher the developed strains and stresses are in the beams. For the examined four beam lengths $\ell_1$=25.0 m, $\ell_2$=50.0 m, $\ell_3$=100.0 m and $\ell_4$=150,0 m the largest member strains and stresses were developed in the $\ell_3$=100.0 m. This loading level was a little bit higher than that developed in the $\ell_2$=50.0 m beam.

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