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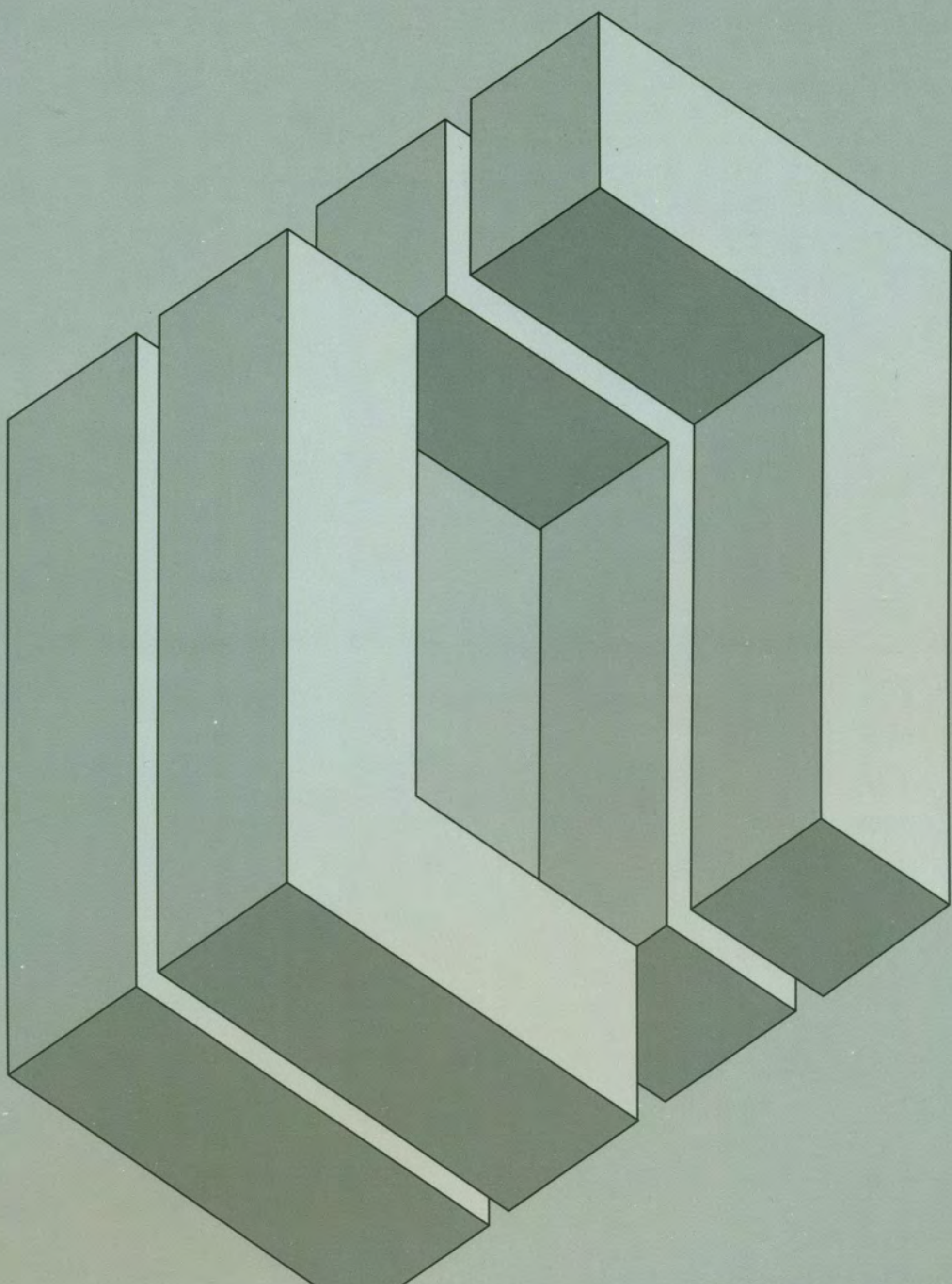

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A sustainable seismic input reduction system for monuments, for existing and new structures by creating large, stiff and strong foundations – practical applications

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SUMMARY – *A system is described, aiming at reducing the seismic excitation of structures, based on pure structural solutions. The basic idea is to exploit the phase lag of the incident seismic waves along the foundation and, accordingly, to design it in order to possess the adequate stiffness and strength. The longer the foundation is, the larger the phase lag becomes. It is, therefore, well understandable that under this requirement, foundations of the maximum possible length must be designed. The presented methodology might be proved quite valuable for existing structures and especially for monuments, where, in most cases, it is not possible to proceed to the necessary strengthening interventions in the structure above its foundation. As a technical support of the present investigation, the size of the foundation as a two dimensional elastic beam and the velocity of the propagation of the ground motion are examined as basic parameters. Two strong ground motions have been used, each one with quite different characteristics compared to the other one: an artificial time history of rather high frequency, fitting to EC8, Type 1, ground class A and a natural ground motion of the Edessa, Greece 1990, M = 5.9, earthquake. The Edessa earthquake is characterized by much longer predominant periods of vibration compared to the artificial one. Various lengths of the foundation beam have been examined in combination with the velocity of the propagation of the ground motion along the longitudinal direction of the beam. The achieved motions at the center of gravity of the beam as well as the pertinent response spectra are calculated. These spectra are compared to the free field ones. At the beginning of the paper, it is tried to explain the inconsistency between macroseismic observations and earthquake code requirements concerning the effects of the size of the building foundation. At the end of the paper, the results of the described methodology are demonstrated in several practical case studies.*

Keywords: seismic isolation; foundations; strengthening; monuments; existing buildings; new buildings.

1. Introduction: Setting the Subject and Historical Review

The scope of the present communication is the clarification of the effect of the foundation size to the earthquake response of the above standing structure and in the case of beneficial effects, what should be the practical engineering measures that must be taken.

The relationship below is very well known:

$$S \leq R \qquad (1)$$

where:

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

In any case of application, either to a new or to an existing structure, this relationship can be satisfied by the following two ways, or by a combination of both: (a) by increasing the resistance (R) of the structure, or (b) by decreasing the loading (S) of it. The present communication is dealing with the second way and especially with reducing the effective input seismic motion.

The influence of the size of the foundation to the earthquake response of the structure has attracted, for more than 50 years ago, the interest of various researchers.

Housner /14/ observed and explained the reduction of up to 50% of the free field motion along the longitudinal (longer and stiffer) direction of a building during the Arvin Tehachapi earthquake of 1952, see Fig. 1. Along the transversal (shorter and more flexible) direction of the building no change was found in the building motion compared to the free field one.

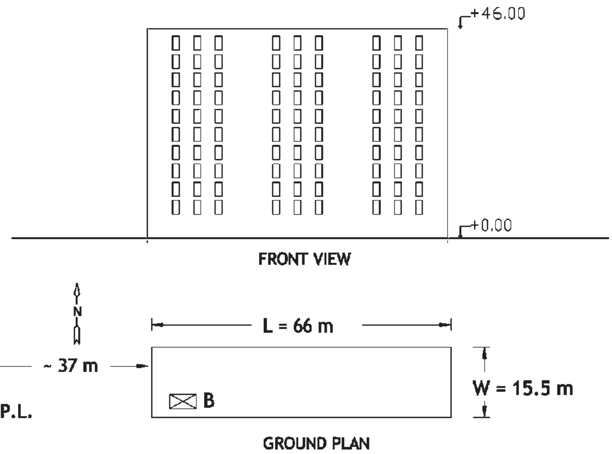

Fig. 1. An *accelerograph functioning on the basement (B) of the Hollywood storage building and another functioning on the base of the parking lot (P.L.) recorded the earthquake ground motion, /14/.*

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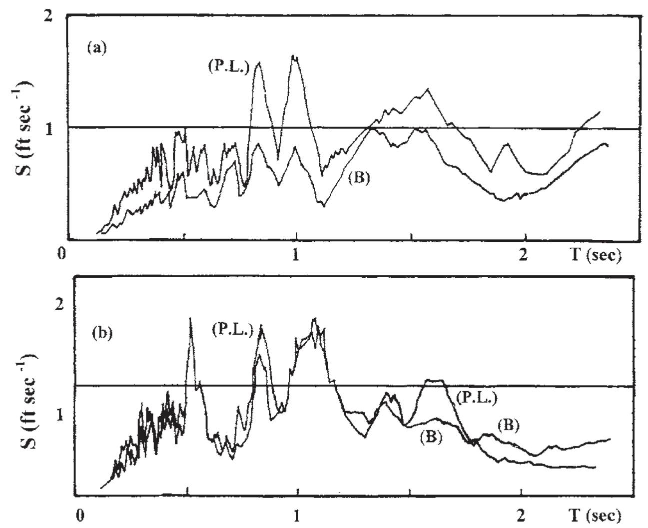

Fig. 2. *Comparison of acceleration response spectra for ζ = 0%; a): along E–W direction, b): along the N–S direction, see Fig. 1, /14/.*

Actually, that case study concerns the Hollywood Storage building which is a quite large (length L = 66 m, width W = 15.5 m, height H = 46 m) and almost monolithic concrete box. Its fundamental periods are 0.49 sec along the E–W direction and 1.20 sec along the N–S direction. An accelerograph functioning at the basement of the building recorded the input motion of the said earthquake. Relatively close to this building (at a distance of about 37 m), another accelerograph was functioning on the base of a small (2 m × 3 m), light metallic shelter, in the parking lot. The response of the second instrument may be considered as the free field motion. The mentioned comparison was carried out with the use of the respective ζ = 0% damping response spectra as it is shown in Figs. 2a and 2b for the E–W and N–S directions respectively. The said up to 50% reduction of the free field motion is valid for smaller than 1.2 sec periods of the spectrum, while for greater periods the difference is smaller. According to the investigation /14/ the obvious reduction is due to the length of the foundation along the longitudinal direction compared to the half wave length of the seismic motion and its high stiffness. As already mentioned, along the perpendicular direction the foundation length is much smaller and the building is much more flexible. This was an indication that the longer the foundation, the smaller the seismic excitations introduced to the building is.

Nevertheless, at the other part of the world, larger foundations were considered as possessing higher earthquake vulnerability. Actually, in the building code for earthquake resistant design of the Ministry of Construction of U.S.S.R. up to the present /18/, /19/ was introduced a limitation for the maximum allowable length of the foundation, depending on the seismicity of the region and the quality and type of the structure. The higher the seismicity, the smaller the length and the stiffer should be each statically independent structural unit. For example, the limitation starts with foundation lengths of no more than 150 m (for non seismic regions) and ends to no more than 40 m (for the highest seismic intensity zone of 9 MSK and worst quality of building walls).

This limitation was set in order to avoid complicated or misleading calculations and the complex evaluation of high tensile and shear stresses in the foundation as well as torsions around the vertical axis of the structure and resulted from macro seismic observations of damage after strong earthquakes. According to these observations the longer the building the larger was the observed damage.

In /23/, the increase of stresses in a type of large panel building was demonstrated by presenting the results of the calculations by E.S. /17/. According to these calculations, using the maximum allowable by the code lengths in the building dimensions, considerable stresses could be created along the longitudinal direction due to earthquake, as high as 1.35 MPa for the tensile stress and 0.74MPa for the shear one. In the same work it was stated that "the increase of the length of a building usually will adversely affect its earthquake response". In the same work the causes of these relatively high stresses are also pointed out in a scheme. It was not explicitly written 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.

For the purpose of the present communication it is adopted that the particle motions taken under consideration for the earthquake resistant design are not discriminated among the orthogonal projections (vertical and horizontal) of the various types of surface and body waves separately. Only the resultant of the motion of the particles, horizontal and vertical is of engineering interest for the time being. Nevertheless, for the sake of completeness in Figs 3a and 3b, the oscillation of the ground particles that produces the propagation of the ground motion is depicted. The particles do not perform a symmetrical to the zero axis oscillation that creates tensional and compressional strains, but oscillate only within the compressional field. This happens because soil materials cannot, in general, transmit tensional strains.

The initial position is marked as i or, after a certain distance j. The maximum translation is, as already mentioned, into the compressional field from i to i″ or, further, from j to j″, and the oscillation takes place from i to i″ and then from i″ back to i, or from j to j″ and from j″ back to j. According to this concept, the final horizontal and vertical motions under and close to the building foundation, are shown in Figs 3c and 3d respectively. At these points of the ground, various forces and strains are concentrated during an earthquake from both the ground motion and the response of the building as well.

Here, a contradiction seams to exist between the documented observations by /14/ and the equally well documented observations and analytical procedures, on which the U.S.S.R. seismic code of 1957 and its later versions are based, resulting in the limitation of the maximum length of buildings and foundations. Obvi-

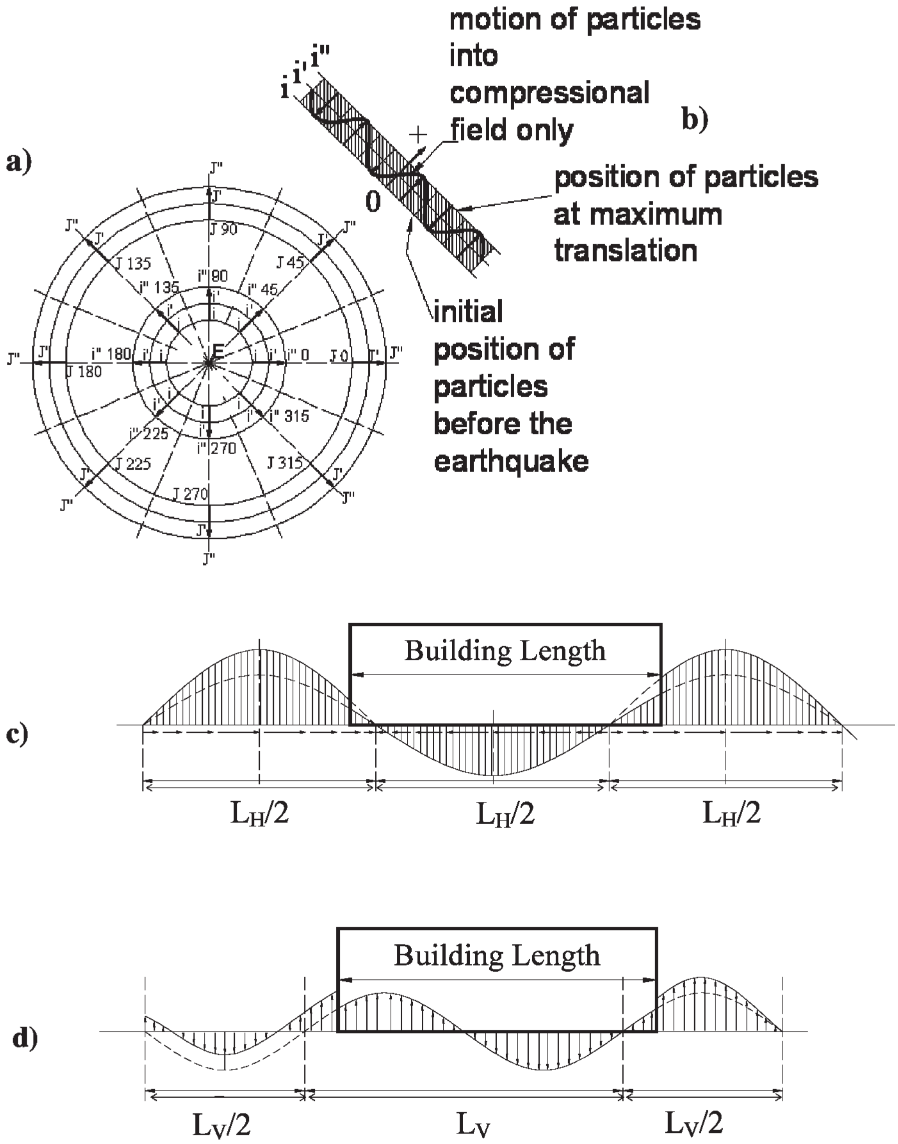

Fig. 3. *Two types of earthquake motions possess an engineering interest under the foundation of a structure, the vertical and the horizontal one: a) for both types the particles from the initial position (i) and further (j) are pushed compressionally to the maximum translation (i″) or (j″), the numbers indicate the angle at the direction of wave propagation from the epicenter E; b) the particle motion takes always place into the compressional field; c) and d) horizontal and vertical particle motion respectively. The motions at the corners of a building present a singularity.*

ously, this is due to the increase of their seismic vulnerability with the increase of their geometrical plan dimensions. However, a more careful study of the Hollywood Storage building reveals that the building is constructed out of solid reinforced concrete walls with very few openings in the facades. Therefore, in spite of its 46 m height, its natural period is only 0.49 sec along the longitudinal direction. This is an exceptional case, which obviously is not related, in any case, to the structures that the U.S.S.R. code of 1957 and its later editions, were based on. We may find in U.S.S.R. long and/or large buildings, built in those years, but not so stiff and strong as the one in Hollywood, as already discussed. If the Hollywood building were much more flexible along its longitudinal direction it would follow the differential seismic deformations of the ground. The usual building materials (for example unreinforced rubble stone masonry), in general, cannot withstand safely these differential deformations. If, on the other hand, the Hollywood building were stiff but not strong enough in order to adequately withstand the forces created by the differential movement of the ground under the non deformable foundation body, damages would also be inevitable.

Nevertheless, later, as it is explicitly stated by /24/, for some buildings erected on strip foundations of continuous footings, considerable decrease of the total seismic loads may be expected, compared to the values that result from the assumption, that the ground particles during the earthquake, vibrate "synchronously throughout the foundation". In /24/ was mentioned that in this case for engineering design purposes the averaging principle might be valid, by which the mean acceleration of the foundation body is:

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

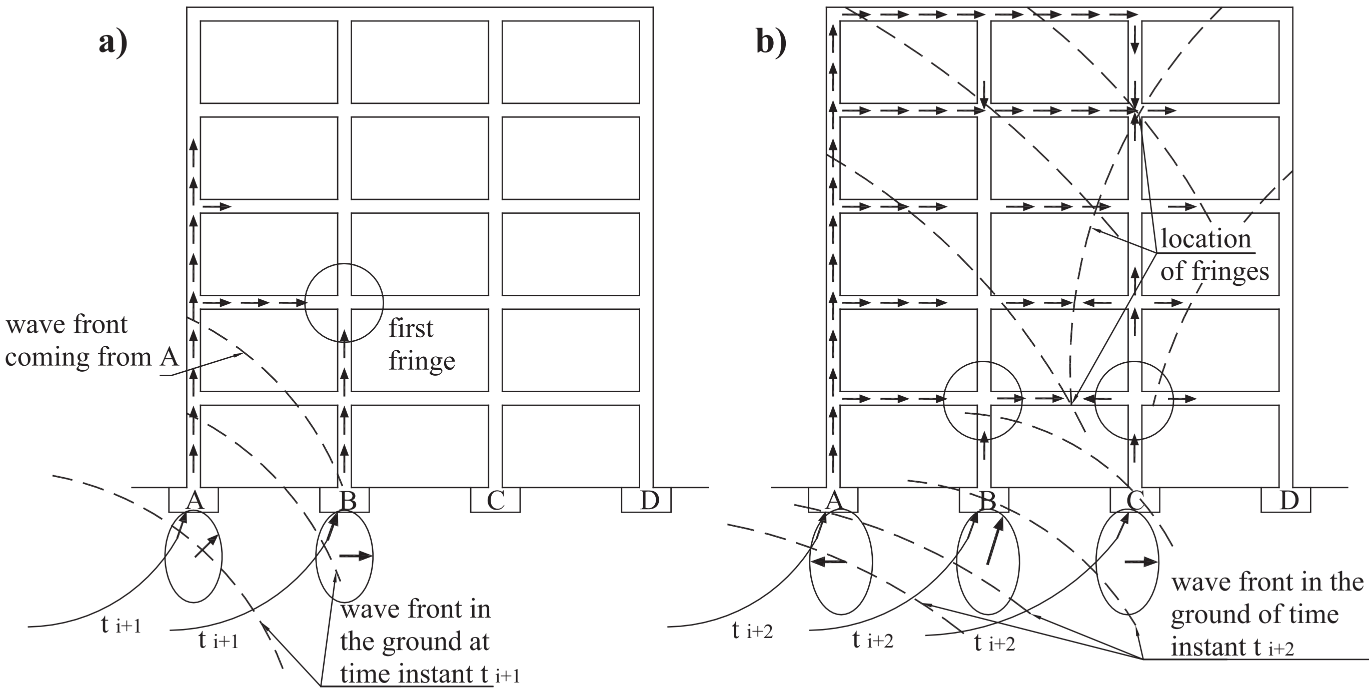

Fig. 4. *The actual earthquake excitation of a structure is realized through the excitation of discrete points of its foundation (A), (B), (C) and (D): a) the earthquake motion that is coming from the left of the figure strikes the left hand parts of the structure first, the other parts of the structure does not move until; b) the earthquake motion propagates through the foundations (C and D). The various inputs show a phase lag among themselves.*

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 local 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.

It is obvious that in Eq.(2) the max $\ddot{a}_b(t)$ is smaller than any local max $\ddot{a}(x,t)$, especially when the building's geometry in plan is sizeable.

One may easily understand that, due to the phasing phenomenon and / or the incoherency of the exiting motion at the various input points of the foundation body, that are also not easily calculable, considerable dynamic forces (axial, moments and shear) may develop in the body of the foundation. These forces are increased by a mutual increase of the length of the foundation, up to a certain magnitude. Therefore, it is logical to expect damages in inadequately designed and dimensioned foundations, /2/. In this case it is acceptable 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).

In the sketch of Fig. 4 the various phasing phenomena is attempted to be approximately presented. In order to illustrate the practical meaning of Fig. 4, the following example may be presented. For a length of a building foundation of 30 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 30/150 = 0.2 sec. This means that for an oscillation period of the ground particles of the order of 0.4 sec the phase lag between points A and D can rise to 180º. In this case, the mean input motion of a large category of earthquakes and building structures will certainly be less than the maximum local one. For example, in the case under consideration, the amplitude of the resulting input motion will be roughly the 0.75 of it (according to Eq.(2)).

A further and much more rigorous investigation of this phenomenon can be found in /27/, in which it was proved, by using a two dimensional analysis, that important torsional vibrations were created in long buildings excited by seismic waves.

It is worth mentioning here, that the widely used term "effective ground acceleration" should rather be substituted by the "maximum mean ground acceleration along the foundation" of a structure as given in Eq. 2. As a logical result of the above, the engineering calculations for the earthquake resistance of the above the foundation structure should be based on the latter quantity which is a function not only of the characteristics of the ground motion itself, but also of the length of the foundation, the apparent wave velocity and the distance from the epicenter. The engineering design of the foundation must be carried out in a completely different 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. This must be done in order to maintain the current building design process specified by the code.

It must be mentioned here, based on Figs 5b and 5c that in epicentral regions of mainly shallow earthquakes, it is not fully justifiable to use the term "sur-

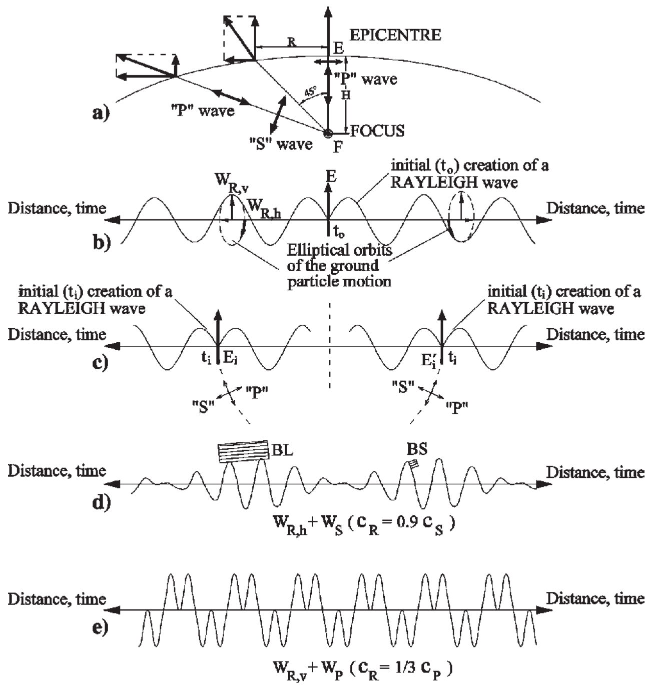

Fig. 5. *The final motion on the surface within the epicentral region, results from the convolution of the reflected and refracted body waves along with the already produced surface waves, as well as any direct motion resulting from the source mechanism, path scattering and seismic energy emission peculiarities. The area with radius R = H, for shallow earthquakes, may constitute, in general, the epicentral region. Large building BL oscillates less than a small one BS in* **d**). *The Rayleigh waves are initially created at point E, t_0,* **b**). *Continuously, new Rayleigh waves are created at points E_i, t_i and E'_i, t_i,* **c**), *due to the vertically emerging body waves P;* **d**) *horizontal and* **e**) *vertical component applied mainly in far field.*

face wave velocity", because the surface waves are greatly distorted due to multiple incident wave motions with various reflections and refractions, and thus, surface waves although created by the body waves emerging from the source, can not be propagated. Nevertheless, in the epicentral regions we have an ostensible propagation of surface waves. Instead, the term "apparent surface wave velocity" should rather be used. The well known function $c = \lambda/T$ (c = wave velocity, λ = wave length and T = period) has actually not a pure physical sense, in epicentral regions, since no one of these three parameters may belong to one and the same wave. This function $c = \lambda/T$ could be used, but only after the above mentioned note and always using the characteristic of "apparent". 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 harmonic. It must be mentioned here that Fig. 3a and Fig. 4 are similar to each other expressing the same kinematic phenomenon (Fig.3a is in plan, Fig. 4 is in elevation).

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 diameters 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 epicentral region, which are projected on the two perpendicular to each other planes, horizontal and vertical. As it is shown in Figs 5d and 5e the resulted horizontal and vertical motions are a superposition of the horizontal projections of the Rayleigh ($W_{R,h}$) and Shear (W_S), and the vertical projections of the Rayleigh ($W_{R,V}$) and P (W_P) waves respectively. Due to the vicinity of the R and S wave velocities, the form of the resulted wave in Fig. 5d is obvious. On the contrary, the P wave velocity is much higher than that of R waves (in Fig. 5e the plot is for $C_p = 3C_R$). In these schemes one has to add the focal – fault displacements (horizontal and vertical) which could be considered, relatively of a quite long period.

One may understand, after the above mentioned, that the phenomenon of the motion at a point in the epicentral region is quite complicated and accelerographs record only the final motion of the respective point, as this is realized along the three axes. They do not record the motion of a surface or of a rigid body with certain dimensions.

Theoretically, (see Fig. 5), the projection of the focus F on the surface of the earth E, in the case of shallow earthquakes, if considered as a single point, it preserves the peculiarity to respond as a singularity. According to /20/ the stresses at that point of the action of a point force become infinite. If the point E is enlarged (being equally stiff as a point in its center), this "point" E presents a minor phenomenon of singularity under the action of the same point force. The similarity between the epicentral point E and the singularity point is obvious, and it is just a matter of scaling. In both cases the results are very intense (a very high seismic intensity for the first case and, as already mentioned, infinite stresses for the second one). However, the level of the intensity of both results, does not have much causative relation to the magnitude of the causing parameter (earthquake magnitude M, or magnitude of the applied concentrated point force, respectively) since in both cases the respective seismic and stress intensities reach their maximum possible values. Nevertheless, for both cases, the magnitudes of the surface of the isoseismals or the isostresses can have a causative relation with the magnitude of the earthquake or the applied concentrated point force, respectively. This similarity refers to the vertical earthquake component, /3/ and /7/.

The most important practical conclusion of the mentioned in the present chapter, is that once the forces and deformations in the foundation body of large dimensions have been successfully confronted, the seismic response of the whole structure will be less, compared to a building with a foundation of smaller dimensions.

2. Post earthquake field observations of building response

The field observations of earthquake damage, presented in the following, aim to illustrate the fact of

Fig. 6. *Rhythmic destruction. Three quite similar buildings in close distance to one another, 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 one. The third building totally collapsed, /9/.*

Fig. 7. *Rhythmic destruction. Two out of five quite similar structures founded on the same soil conditions and constructed by the same contractor totally collapsed after the Dinar, Turkey earthquake of 1995, /11/.*

considerable variations of the seismic response of similar structures that exist in rather close distances. These reported variations cannot by any means be explained, by the rather simple and trite case of different soil conditions from one plot to the other. Also, there are some cases in which the damage refers to structures of rather long dimensions. For example, according to /28/ 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, damage and collapses 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.

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 along a straight line at

a)
b)
c)

Fig. 8. *Rhythmic destruction. Similar buildings along one and the same street within a small distance a), suffered damage of quite different intensity alternately. From almost no damage b), up to total collapse c). Erzincan, 1992 earthquake, by courtesy of Dr. A. Pomonis, /29/.*

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. 6 are shown three buildings. At the left hand side is located the first building that suffered light 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 by the author, the soil conditions were the same.

Quite interesting is, also, the case shown in Fig. 7, after the Dinar, Turkey, 1995 earthquake, /11/. The two a)

b)

Fig. 9. *a) The two storied and rather stiff warehouse in the epicentral region of the Parnitha, Athens, Greece, M = 5.9 earthquake, 7 Sept 1999, suffered extended damage at the second storey. b) The adjacent beams at the roof of the second storey suffered severe damage due to impact between themselves. The beams, due to the impact, were also pushed towards the outside part of the building, /25/.*

out of five similar four storied apartment buildings totally collapsed, while the other three remained safe.

The author investigated the whole site and concluded that there was not any alternative difference in the underlying soil conditions. The 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, the distance of 120 m is comparable to the apparent surface wave's length.

In Fig. 8, 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. 8a) presented a quite great variation of observed damage: from almost no damage (Fig. 8b), up to total collapse (Fig. 8c), alternatively.

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 these building series were connected together rigidly and adequately strongly, collapses would certainly have been avoided. This is because the destructions occurred

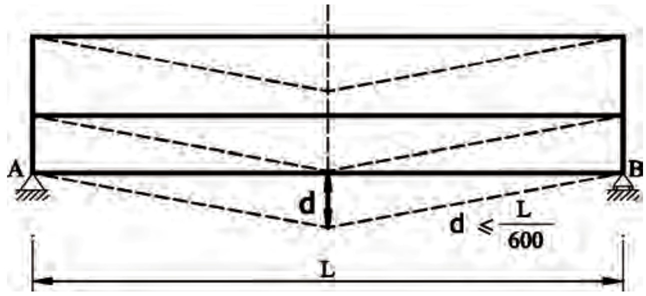

Fig. 10. *The building is considered as adequately stiff along the vertical direction, to avoid damage to the non bearing walls, due to subsidence of the ground at the center, if the flexure f under its own weight is less than 0.5L/300 when freely supported at its two ends A and B*

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).

In Fig. 9a, a two storied warehouse building is shown after the earthquake of Athens in Greece, on 7 September 1999. The total length of the building is 96.5 m, divided into three almost equal parts. These three parts are separated by two expansion joints of a width of about 4-5 cm and they are statically independent. The width of the building is 30.90 m while its total height is about 12.0 m. The building is a shear type one, rather stiff. The main source of damage could be attributed to the combination of the vertical component of the seismic motion along with the surface seismic waves at the foundation level, and the flexibility of the building – foundation body along the vertical direction. This resulted to the severe impact of the roof beams between the adjacent parts of the building, as it is shown in Fig. 9b. The presented damage in Fig. 9b are symmetrical along both construction joints.

The flexibility of the foundation and, further, of the whole building along the vertical direction, due to a subsidence of the foundation at its center, can be checked, during the design, with a simple calculation, as it is schematically shown in Fig. 10. If the stiffness of the foundation is not enough to withstand the subsidence (d) the stiffness of the whole building along the vertical direction 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.

The pictures shown in Fig. 11 present views of a hotel that suffered partial collapse during the Egion, Greece, 1995 earthquake, /10/. The building is consisted out of three parts statically separated by two separation joints. The width of these joints is negligible or non existent. The damage of the remaining two blocks and the remaining part of the third block (Fig. 11b) 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 direction could be an explanation of that partial collapse. The

a)

b)

Fig. 11. *During the Egion 1995, Greece, M = 6.1 earthquake the building suffered rather limited damage in all its three blocks except the partial collapse of the third block, located at the right hand side of the picture, /10/.*

Fig. 12. *Partial collapse of the building after the Tokachi-Oki earthquake of 1968, /2/.*

almost total lack of damage of the remaining parts supports this approach. During the same earthquake, many other similar partial collapses, while the rest of the building remained intact, were reported, /10/. The focal mechanism through the body P waves might have a contribution to this result.

The case of partial collapse is observed also during many other destructive earthquakes around the globe as, for example, the case shown in Fig. 12 after Tokachi-Oki, 1968 earthquake. This phenomenon might be classified as a rhythmic destruction too, with the same conclusions as mentioned above that if the foundation structure were stiffer and stronger, the collapses would have been avoided.

3. **A brief theoretical background and references to studies of seismic arrays**

It is obvious that the whole under consideration subject is based on the variation of the input motion along the foundation body. This variation is due to the amplitude and the phase difference 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. According to /6/ this coherency in the frequency domain is expressed by:

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

in which:

– S_{ij} is the cross spectral function and

– S_{ii} or S_{jj} are the auto spectral functions.

According to /6/ the coherency is a normalized complex function. The coherence is identified as the square of the modulus of the coherency i.e. the quantity $|\gamma_{ij}(\omega)|^2$, which as 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 records obtained from the various strong motion arrays as it is presented by /1/, /6/, /21/, /22/, /30/, /31/ 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 etc.

According to /6/ the recorded motions by the SMART 1 array at frequencies above 2 Hz are dominated by "incoherent energy" when they are averaged over a large distance.

It is interesting to mention here the conclusions by /21/ according to which the peak cross correlation values (P) of the accelerogramms, of the El Centro Differential Array may be approximated by the expression:

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

where:

– L is the distance in (m) between the two stations or the foundation length,

– λ is the apparent wave length in (m),

– c is the apparent velocity of the wave propagation in (m/sec) along the foundation, and

– $f = c/\lambda$ is the apparent frequency in (Hz) of the particle ground motion ($T = 1/f = \lambda/c$).

From this equation, 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, which means that we get the same values of cross correlation either by long lengths of foundations or by high frequencies of the seismic motion.

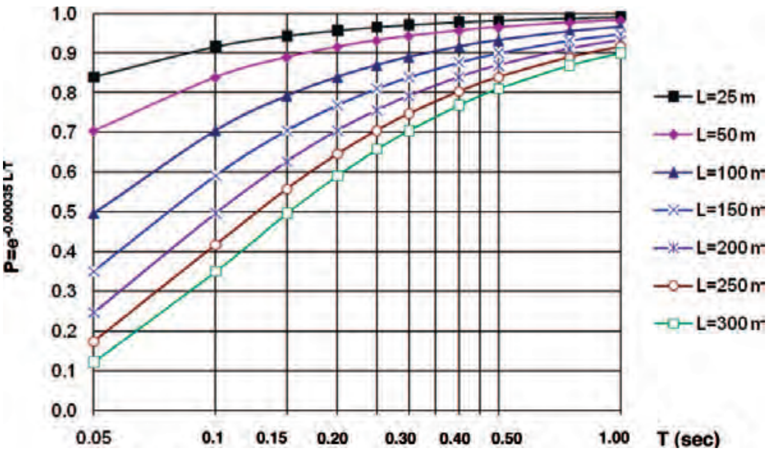

Fig. 13. *Plot of the function $P = e^{-0.00035\ L/T}$ according to /21/.*

This finding of the function (P) by /21/ is illustrated in Fig. 13. For example, we could have the same maximum value of the cross correlation function, P = 0.70, between the two incident acceleration time histories at the beginning and at the end of the foundation body of a length (L), with the following combinations of the predominant period of the ground motion (T) over the length (L):

T(sec)/L(m) = 0.05/50, or 0.1/100, or 0.15/150, or 0.2/200, or 0.25/250, or 0.30/300 (since for all these six combinations the ratio L/T is constant).

The research work /16/ contributed considerably to the 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 /30/ in which the «differential motion $\Delta_{jk}(t)$», also other physical parameters are introduced and studied as well; by /31/ 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 /13/ in which the effects of the spatially variation of the seismic input motions on a two – span indeterminate beam are parametrically investigated.

Nevertheless, it must be mentioned here, that all these observations might be much more emphasized in epicentral regions of shallow normal or reverse faults. 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, see Fig. 3 and Figs. 5b and 5c.

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, /15/ 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

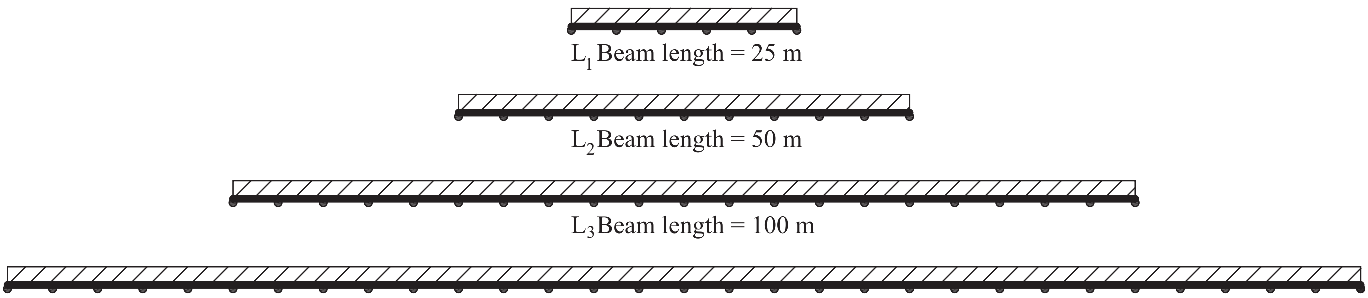

Fig. 14. *The various beams and points with spring supports – input motions used in the parametric analyses.*

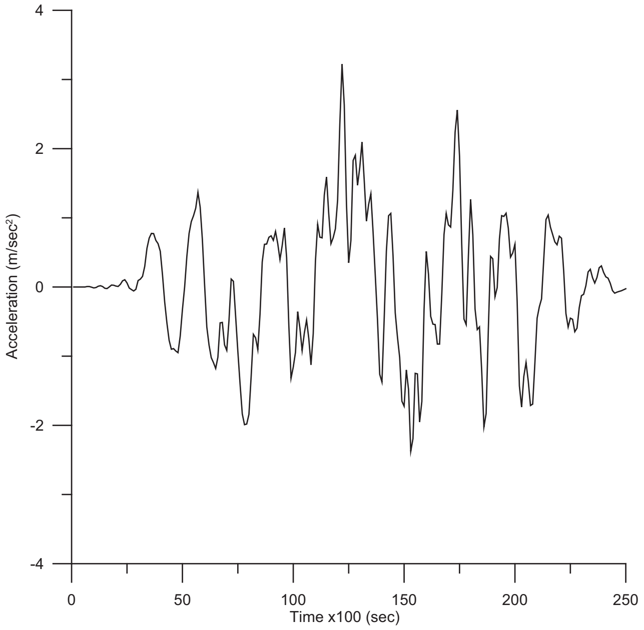

Fig. 15. *Artificial accelogramm fitting to EC 8 Type 1 response spectrum, ground class A, design acceleration 0.24.*

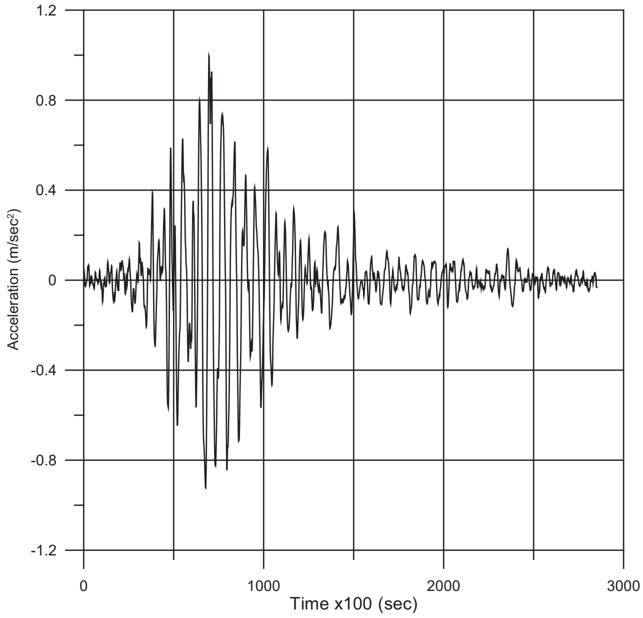

Fig. 16. *The N – S component of the Edessa, 21 Dec. 1990, M = 5.9, northern Greece, Griva earthquake recorded by ITSAK.*

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.

4. Calculation of some indicative response spectra as a function of the length of a massless foundation and the velocity of wave propagation

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.

The motion that is recorded by an accelerograph is the convolution of the particle motion of the ground along the respective three axes (X,Y,Z).

For the parametric study a massless beam 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. Its rigidity is a function of its cross section 0.5 m width x 2.0 m height. The beam rests on elastic springs with a stiffness of

$$K_h = 4.0 \text{ MN/m and } K_v = 5.0 \text{ MN/m} \qquad (5)$$

where

– K_h is the horizontal stiffness of the springs
– K_v is the vertical stiffness of the springs
– their modulus of elasticity is $E = 2.9 \cdot 10^7$ kPa and the poisson ratio is $\nu = 0.2$.

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. This ground motion acts horizontally along the longitudinal direction of the above mentioned beam. The whole accelerogramm is propagated with a certain velocity along the beam and the excitation is applied through the above mentioned horizontal beams. The velocity of the propagation of the accelerogramm is selected for the various parametric calculations as follows:

$$c_1 = 100 \text{ m/sec}, \ c_2 = 200 \text{ m/sec},$$

$$c_3 = 400 \text{ m/sec and } c_4 = 600 \text{ m/sec} \qquad (6)$$

For example, for the case of the beam $L_3 = 100$ m and the velocity $c_3 = 400$ m/sec its last support starts to be excited at a time lag equal to $\Delta t = 0.25$ 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.

On the other hand, this way of excitation may be compatible to the excitation resulting from the horizontal component of the emerging body waves, schematically shown in Fig. 4c. The point of emersion coincides with the discrete supports of the beams, shown in Fig. 14.

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 $a_g = 0.24$ g, of a total duration of 2.4 sec, as it is shown in Fig. 15. The second one is the Edessa N – S component due to Griva northern Greece, 21 Dec. 1990, M = 5.9 earthquake with max $\ddot{a} = 0.1$g, 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 < 15 km. The record is on rather soft soil conditions.

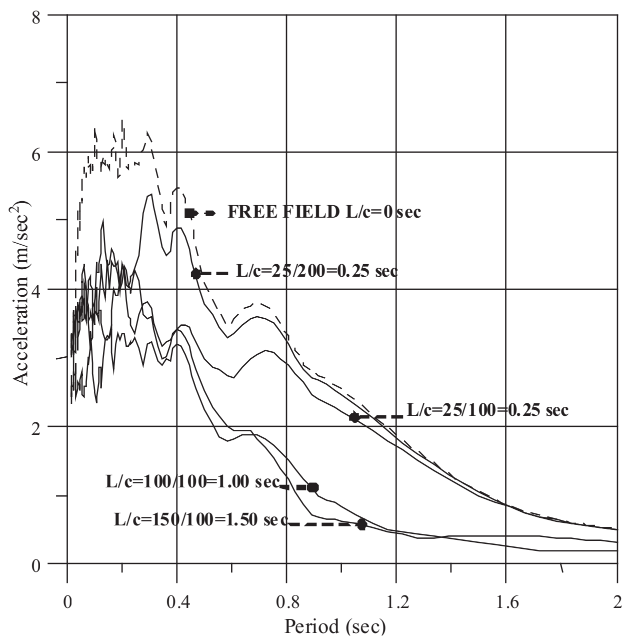

Fig. 17. *Response spectra of the artificial motion fitting to EC8 Type 1, for various τ (sec) = L/c = beam length/ wave velocity, and 5% damping ratio.*

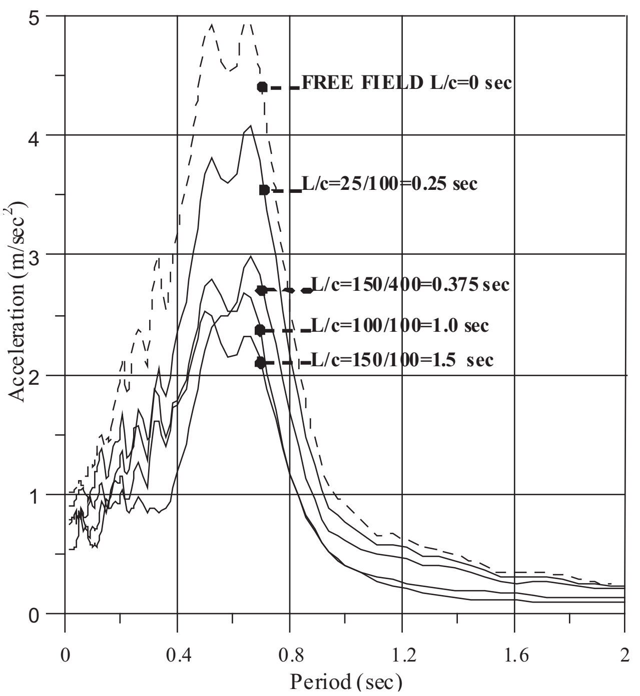

Fig. 18. *Response spectra of the Edessa earthquake, for various τ(sec) = L/c = beam length/ wave velocity, and 5% damping ratio.*

Therefore it is logical to measure predominant periods between 0.5 sec and 0.7 sec. The component used, is shown in Fig. 16.

For each type of excitation four wave velocities, for four beam lengths = 16 combinations of beam lengths and wave velocities have been analysed. Combined with the two accelogramms the total number of the analyses, presented here, reached 32. 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 /24/, that the determining parameter among the various response spectra is the ratio $\tau = L / c$ (sec). Therefore, in Fig. 17 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 presented for the artificial ground motion.

In Fig. 18 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 presented for the Edessa ground motion.

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. Namely, for the two earthquakes and the various $\tau = L / c$ values, the following ratios of the obtained maximum response spectra values to the free field ones are approximately as given in Table 1.

As it is shown in Table 1, the reduction of the response spectra values for the two earthquakes (artificial and natural) is up to 50 % compared to the free field values, /24/.

Table 1. *Comparison of the ratio of the obtained at the center of gravity of the beams maximum response spectra to the free field ones.*

τ = L/c (sec)	0.0	0.125	0.25	0.375	1.0	1.5
Artificial	1.0	0.84	0.76	–	0.66	0.50
Edessa	1.0	–	0.84	0.60	0.50	0.50

The maximum calculated stresses in the second beam ($L_2 = 50$ m) of Fig. 14 were the following: Homogeneous axial stress, about 1.0 MPa (tensional or compressional). Axial stress due to bending, 1.5 MPa and shear stress 0.6 MPa. These stresses calculated after intergration, over the respective cross section area of the beam, 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 is 24Ø25, with an additional reinforcement 15Ø25 at the top and 15Ø25 at the bottom side of the beam. Stirrups 2xØ12/15. The geometrical percentage of the main reinforcement is 2.7%.

5. **Application of the findings on some case studies**

The so created long, rigid and strong beams can also be constructed and function as a two – dimensional close orthogonal grillage of beams in order not only to comply with the plan dimensions of the buildings but also, to reduce the input seismic motions along the other directions too. Besides that, a grillage of beams of large dimensions increases, also, the necessary resistance against torsional phenomena that may be created

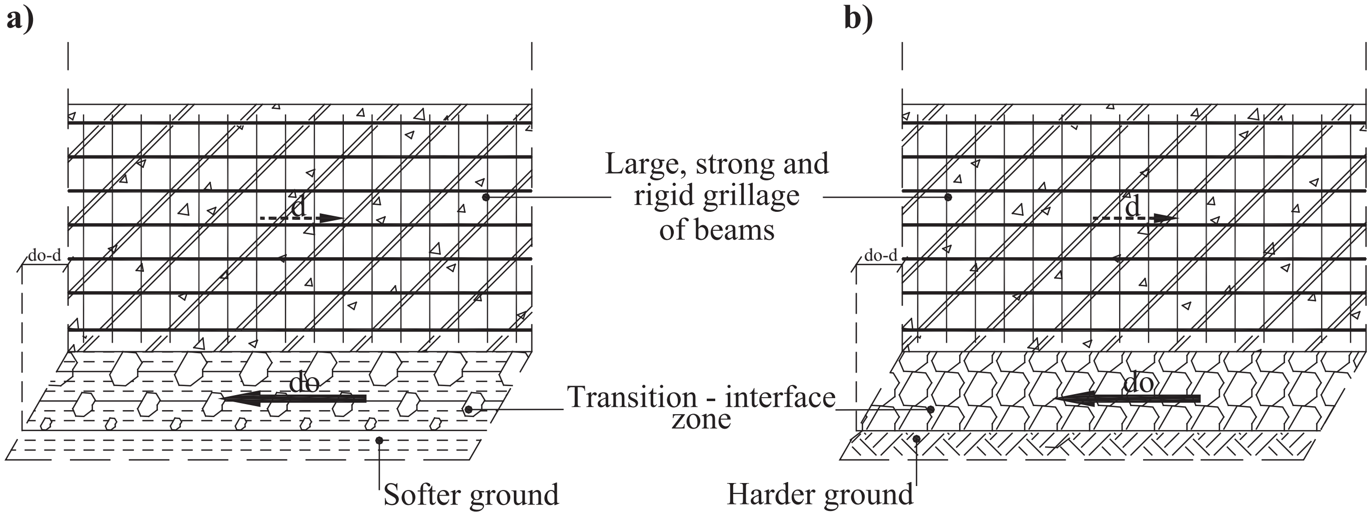

Fig. 19. *Cross section of the general set up with the transition – interface zone between the ground and the rigid grillage of beams; a) the underlying ground is rather soft and the transition zone can be created with the same material, but with a gradual improvement of the ground from the depth to the base of the grillage, using rubbles thrusted into the soft ground; b) the underlying ground is hard and the transition – interface zone is out of equigranular rubbles dimensioning 5-7 cm.*

by the non uniform motion of the ground and by the torsional response of the structure itself during strong earthquakes. If the above mentioned are correct and applied in practice, the so formed foundation body will possess a minor kinetic state compared to the underlying ground. Therefore, a transition – interface zone will be naturally formed between the moving ground and the less moving and/or out of phase moving foundation grillage. Instead of relying on any shapeless formation of this transition zone under the foundation body it would be better this zone to be adequately defined and designed. This is in accordance with the proposed method presented in /4/. This transition – interface zone is a path along which an amount of input seismic energy is going to be absorbed. The thickness and characteristics of this transition – interface zone will result from an adequate design. Nevertheless, a sliding of the grillage on the transition zone should, in general, not be accepted. For this reason, it is indicatively shown in Fig. 19a a gradual improvement of the underlying ground from a certain depth up 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 equal dimensions (about 5 to 7 cm), as shown in Fig. 19b. The thickness of this transition – interface zone, might be of the order of 30 – 50 cm, always depending on 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).

Based on the findings and proposals of the present work, one may create, according to the requirements of a specific application, the appropriate combination of the various parameters involved. First of all there are two basic categories of applications: one is the case of existing structures or monumental buildings isolated or in conglomerates. The other one is the case of new structures (hospitals, museums and buildings of increased importance) and when we are obliged to erect structures in regions of seismically adverse conditions, as for example in regions that have been devastated after destructive earthquakes. Other basic parameters, 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. 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.

5.1. *Application No 1*

During the Egion (northern Peloponnesus) earthquake of 1995, /10/ a church erected during the decade of 1890 was severely damaged. The church is a Greek national monumental treasure. The damage occurred mainly along the perimetric walls, which support arches and domes as a roof construction. The four central main pillars suffered almost symmetrical compression damage at their base. The created damage can be classified as one or two degrees of intensity before collapse. Based on the findings already mentioned in the previous chapters, a grillage was proposed to be constructed in contact with the existing foundation of the church. This grillage is made by reinforced concrete beams 40 x 80 cm^2, as main beams and 30 x 80 cm^2 as secondary ones, as shown in Fig. 20, with a length as long as the plot of the site permits (overall dimensions about 55 m along E – W and about 35 m along N – S direction). The stiffness and the strength of the grillage were analytically calculated. The construction of the grillage would be carried out successively, by digging trenches up to a little bit below the bottom of the exist-

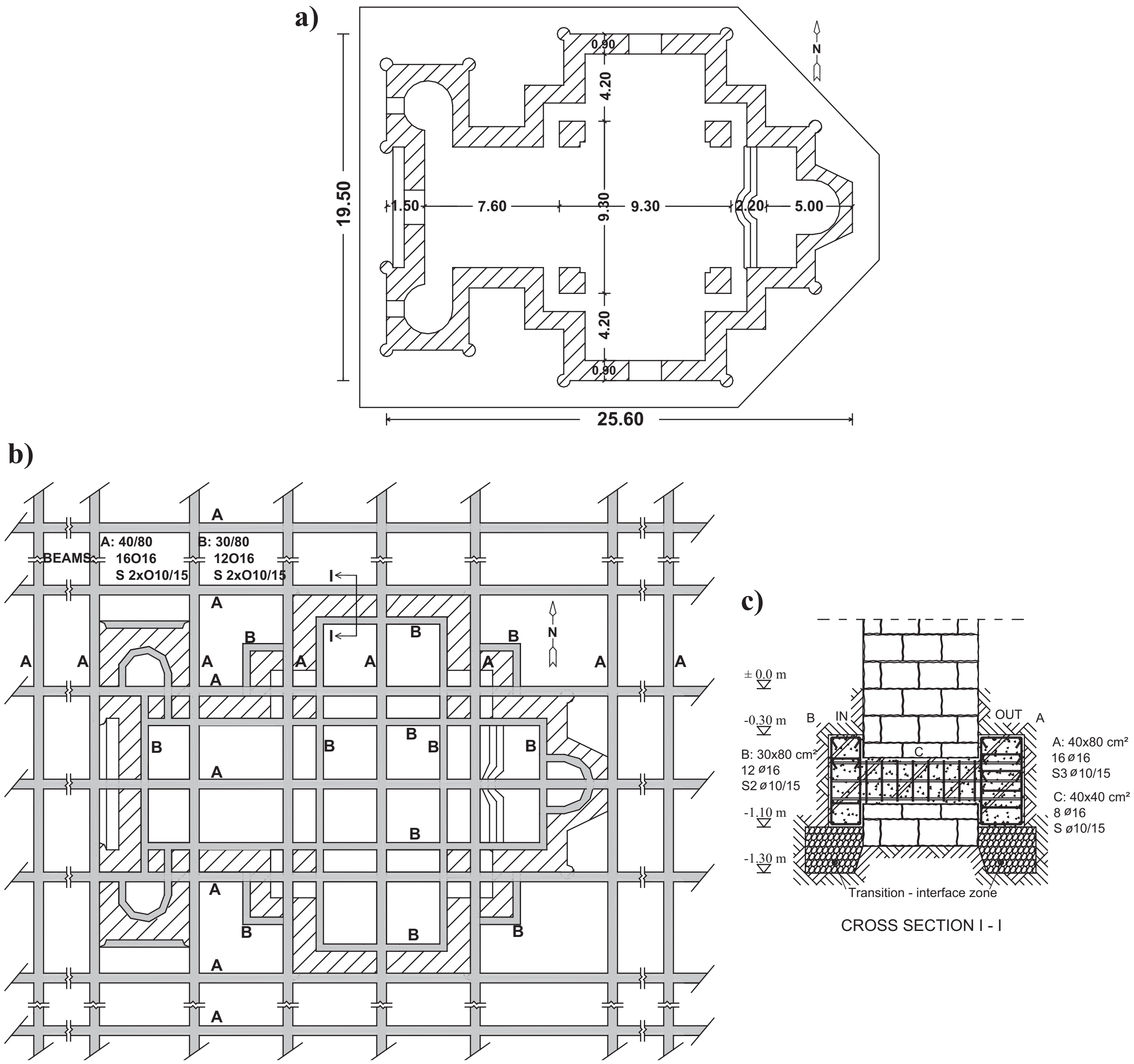

Fig. 20. *Creation of a stiff, strong and as long as possible grillage in the foundation body of the church in Egion, /26/; a) plan of the existing church; b) plan at the foundation level showing the grillage of reinforced concrete beams, A main and B secondary; c) a typical construction detail showing also the transition – interface zone under the grillage.*

ing foundation of the church and by forming openings into the foundation masonry alternately according to /5/. In this way a kind of mould would be shaped, into which the reinforced concrete beams would be poured. The level of the foundation of the walls of the church is about – 1.30 m below the ground surface. The transition – interface zone is constructed under the beams from - 1.50 m up to – 1.10 m (50 cm thickness) composed out of well compacted angular rubbles of about equal dimensions 5–7 cm. The compaction is performed with a special light weight vibrator at 20 cm thickness zones. From - 1.10 m up to - 0.30 m the grillage beams along the walls of the church are connected together by reinforced concrete studs at distances of no more than 1.50 m, for detailing see Fig. 20c, according to /5/. There are numerous details that follow the design whose presentation is beyond the scope of the present communication. Nevertheless it must be stated that the grillage is closed along its boundaries. The end beam is a little bit stronger than the internal ones. The repair and strengthening of the damaged church would be done in any case, by bringing the church at the same or at a better condition than it was before the earthquake. The so achieved earthquake resistance of the church is about 30% less than that required by the code. The above mentioned construction of the grillage proved to absorb even more than the 30% deficit.

5.2. *Application No 2*

In the following, the application of the above mentioned method for the foundation of a new industrial building is presented. The building is a power station

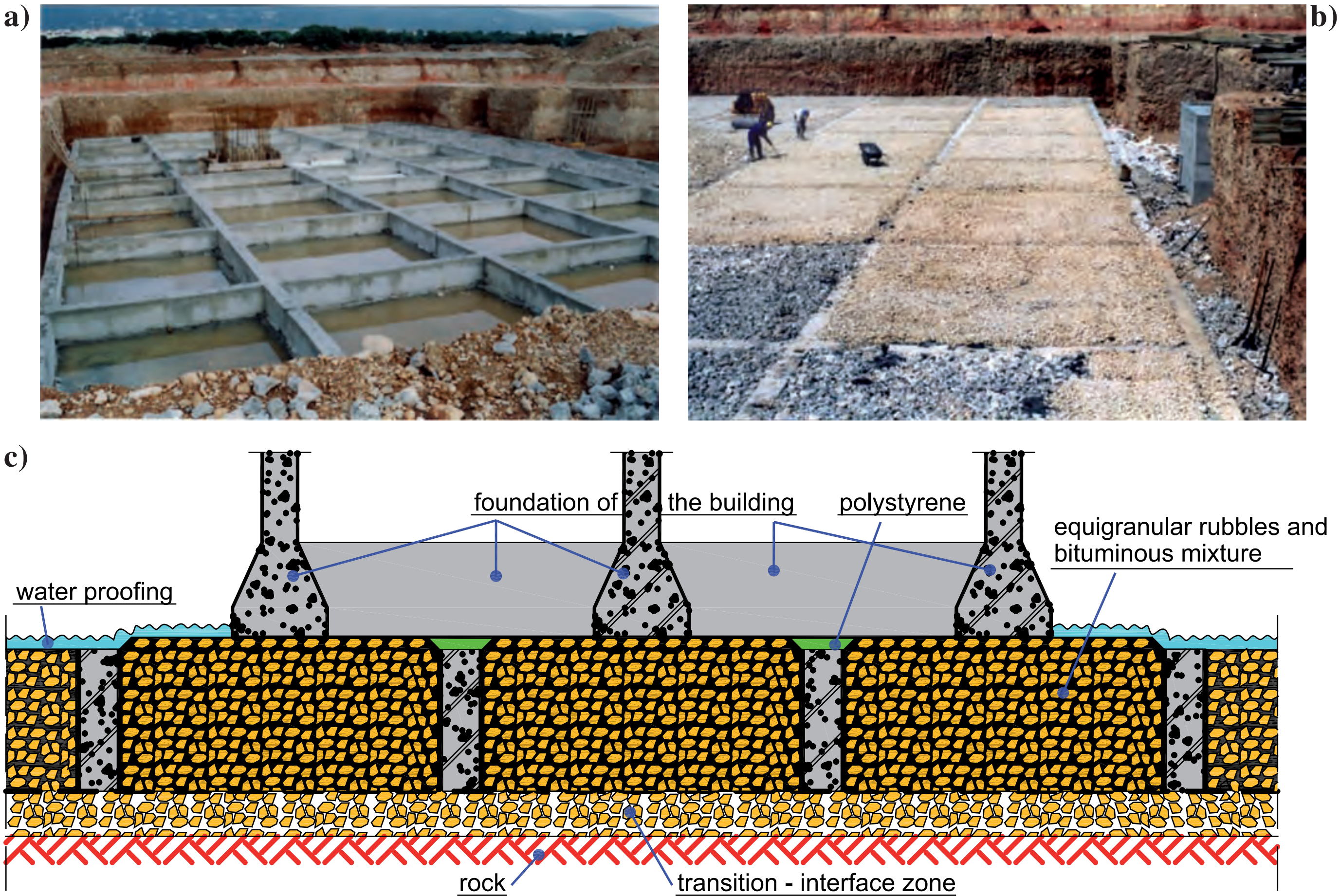

Fig. 21. *a) The excavation and the grillage of R/C beams, where the empty panels – boxes are shown; b) the panels – boxes filled up with the rubbles and bituminous mixture; c) a cross section of the constructed system indicating the transition zone, the R/C grillage beams, the special filling material, the polystyrene at the top of the beams and the foundation of the building.*

needing an extra earthquake protection, not only for the structure itself but also for the sensitive electro–mechanical machinery inside it, since it should be located within the epicentral region of the Athens – Parnitha earthquake /25/. The foundation ground is rock. The building consists of two parts separated by expansion joints. The first part of the building is a single tall storey one (8 m height) and the second one is a two storied (total height 7 m). The two parts of the building have been elastically connected together before finishing them (when almost all deformations have been developed). It has been proved by /12/ that a considerable reduction of seismic forces takes place after the referred connection of the adjacent parts above the foundation. This reduction is besides the one achieved by the application of the system of the foundation described here in.

In the present application No 2 we follow the already mentioned methodology but with a difference: the building with its foundation is – in a sense – separated from the long, strong and rigid foundation – grillage body.

In Fig. 21 the various parts of the construction of the whole foundation are shown. The excavation is at a depth of about 3.0 m from the physical ground surface. The transition – interface zone has a thickness of about 30 cm. On this layer rests the grillage out of R/C beams 35 × 120 cm^2 at spaces about 5 – 6 m. The grillage has a close rectangular plan with not any expansion joint. The plan dimensions are 60 m × 120 m about. The concreting of the grillage was done in parts in order to avoid shrinkage problems. Between the beams, panels – boxes are formed. Inside these boxes at layers with a thickness of about 30 cm, well compacted, rubbles, by vibration, are laid. The rubbles are of almost equal size of 7 to 10 cm. At each layer and within the so formed considerable voids among the equigranular rubbles a special grout, described below, is poured in hot condition. The top surface of the so formed layers is about 10 cm higher than the top level of the grillage beams. Polystyrene strips of equal height are set over the beams in order to ensure the separation between the two structures. In this point it must be mentioned that the plan of the main building has a quite complicated shape following the architectural needs. Its plan covers about the 20% of the total plan of the grillage. For the design of the above the grillage building foundation, the value of the subgrade reaction coefficient was taken adequately low, in order to correspond to the mechanical characteristics of the underlaying material.

The special grout consisted of a mixture of about: 35% ground car tires, 35% bitumen and 30% clear silicon sand. The grout is poured in hot condition in order to achieve the best liquidity in order to fill all the voids of each layer of 30 cm thickness each time. To further increase the diffusion of the grout, a small quantity of petroleum was added into its mass a little bit before pouring. After a few hours the whole compound (rubbles and grout) obtains the following characteristics as proved by shake table tests in the laboratory: (a) high viscosity which results to high damping at this part of the building structure. Besides the pure seismic motions this type of construction absorbs, also, the high frequencies coming from the ground, such as the mechanical noise due to traffic and other sources; (b) more uniform response among the body of the rubbles and the body of the grillage by gluing them, with the grout, and (c) increase of the stiffness of the grillage as a uniform body together with the rubbles.

It might be added to all these, that, as it is shown in Fig. 21c, it was tried to position the columns of the main building at the centers if the created paneled boxes of the foundation body. This was achieved by relocating the columns and the spacing among the grillage beams.

In another application for the construction of a new town out of about 1200 two and three storied houses and buildings, of a variety of use, on a seismically adverse ground conditions, the above mentioned transition – interface zone was replaced by elastomers of adequate dimensions that were fixed on concrete bases freely following the ground motion. The said elastomers had also the capacity to absorb considerable amounts of seismic energy. The elastomers were protected against humidity and against any adverse atmospheric attack, although they are rather protected from the latter one due to their position. Among and around the elastomers clean sand was laid. The sand was confined along the boundaries.

In these applications the grillage was incorporated into the foundation of the above standing structure functioning as a uniform structure.

6. **Conclusions**

With the presented methodology a sustainable seismic input reduction system may be constructed for existing structures and monuments, single, or in conglomerates. The basic key element of the system is a strong, large and stiff grillage of beams constructed in close contact with the foundation of the structure. Under the beams of the grillage a transition – interface zone may be constructed for the better effectiveness of the method. This zone is functioning also, as an energy absorbing zone, if adequately designed and constructed. In the case of new structures the above mentioned foundation grillage may be put under its foundation to function either almost separately, or to be incorporated into the structure. The effectiveness of the method over its cost proved to be superior than other existing methods (base isolation).

The increase of the horizontal dimensions of the foundation body – grillage of beams of a structure increases its aseismic stability, reducing the input motion, but it creates internal forces in the foundation body. These forces must be faced by an adequate design and detailing. Once these internal forces are successfully resisted by the above mentioned grillage, the structures based on the so constructed grillage will develop less seismic loads compared to structures without it.

The parameters for the design the proposed foundation body are: (a) the anticipated earthquake intensity, the respective predominant period of the ground motion, the apparent seismic wave velocity and the magnitude of the incoherency of the incoming into the foundation body seismic waves; (b) the degree of damping provided by the interface zone; (c) the dimensions of the foundation body along the horizontal and vertical planes; (d) the mass, the flexibility and the geometry of the above the foundation structure, and (e) the degree of interaction of the grillage with the main building.

The proposed methodology could be applied under various combinations of the particular site and structure requirements. The designer may select, for this goal, a great variety of parameters some of which are presented in the respective cases studies. With the proposed method may be also protected the contents of the building, the machinery and delicate instruments or exhibits.

According to the parametric analyses that were carried out, the obtained motions at the centre of gravity of the foundation gave spectra which are smaller compared to that of the respective free field motions. This reduction was up to 50%. In epicentral regions the reported reduction and the resulting protection of structures could be higher, compared to the far field case. Also, larger foundations extended beyond the plan of the building greatly contribute toward the mitigation of the incompatibility between building and ground motion that is usually observed in the form of gaps around the perimeter of the building.

The construction of the large in plan, stiff and strong grillage attempts also to avoid the creation of points of singularities especially against the vertical earthquake component, which lead to heavy damage or even collapses of structures. This heavy damage may occur even in well constructed structures according to a conventional design following the respective code. This was observed in numerous damage after destructive earthquakes in which the response of closely standing similar buildings was strikingly different.

In the conventional design of the building and its foundation, additional calculations must be included even in the case of not providing a larger foundation body. These might be: (a) the checking of the strength against deformations of the ground due to the convolution of the various waves (creating bending moments and shear forces); (b) the checking of the axial strength of the foundation body due to phase difference of the various incident seismic waves (creating axial forced). If the strength of the foundation body is not enough in order to confront the above loading case (a), the whole body of the building together with its foundation could be included in the design as a «Vierendeel beam».

Instead of using the term "effective ground acceleration", the term "maximum mean ground acceleration along the foundation", or a similar expression should be adopted, in order to discriminate between the free field motion and the actual input motion for the design, which is greately influenced by many parameters presented here in, the most important of which is the size of the foundation.

The present communication is dedicated to my best student and eminent Professor of Geomechanics in Thrace University the late Panos Papakyriakopoulos who encouraged me to publish it.

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RIASSUNTO ESTESO

Un sistema per la riduzione dell’input sismico mediante l’impiego di fondazioni estese, rigide e resistenti. Applicazioni pratiche su edifici nuovi ed esistenti

Panayotis Carydis

Il ruolo delle fondazioni di grandi dimensioni nel determinare la sicurezza sismica delle costruzioni è, da tempo, oggetto di dibattito tra gli ingegneri. Questo lavoro si propone di dare un contributo al tema, basato anche su un significativo numero di applicazioni reali. Tra queste se ne presentano due, riferite a:

1) Un intervento su un importante bene architettonico, una grande chiesa, gravemente danneggiate dal terremoto del 1995, per la quale l’impiego delle tradizionali tecniche di riparazione e rafforzamento era limitato dall’esigenza di conservare la struttura originaria, gli affreschi ed i dettagli architettonici. Inoltre queste tecniche non erano in grado di assicurare il livello di resistenza richiesto dalla norma, risultando un deficit di resistenza di circa il 30% rispetto a quello prefissato. Si è quindi scelto di affidare il compito di raggiungere la resistenza prescritta agendo sul sistema di fondazione, rendendolo il più esteso e rigido possibile, come illustrato nel par. 5.1.

2) L’intervento in una zona dell’area di Atene fortemente danneggiata ed in parte distrutta dall’evento del 1999. Tale zona non è stata abbandonata, per diverse ragioni, e si è deciso di intervenire con la costruzione di una fondazione di elevata resistenza e rigidezza ed assai estesa, che ha incorporato e sostituito le singole fondazioni preesistenti. Con lo stesso principio (V. par. 5.2) sono stati realizzati impianti industriali e di produzione dell’energia elettrica, perseguendo l’obiettivo di proteggere non solo gli edifici ma anche i delicati impianti meccanici ed elettrici da questi contenuti.

Questo lavoro trae origine dall’importante osservazione formulata da Housner, dopo il terremoto di Avril Tehachapi (1952), a proposito dei danni subiti da un edificio a pianta rettangolare, assai estesa in una delle due direzioni, con una rigidezza ben più elevata nella direzione maggiore rispetto a quella nella direzione minore di pianta. L’edificio era dotato di due accelerometri a tre componenti: uno situato in corrispondenza del seminterrato e l’altro alla distanza di circa 37metri dalla costruzione in modo da registrare il moto *free field*. Gli spettri calcolati dai segnali registrati mostrano rilevanti diversità. Quello relativo all’eccitazione lungo la direzione maggiore (e più rigida) dell’edificio dà luogo a valori pari a circa la metà dello spettro *free field*, mentre quello relativo alla direzione ortogonale, per la quale la rigidezza è di molto più bassa, presenta valori assai prossimi a quelli del moto *free field*. Da queste osservazioni di Housner ha preso avvio l’ipotesi che tanto maggiore è l’estensione delle fondazioni più basso è il livello di eccitazione agente sull’edificio. Va osservato che tale ipotesi è in contrasto con quanto prescritto dalla normativa sismica Russa che, ora come in passato, pone dei limiti allo sviluppo orizzontale degli edifici e delle loro fondazioni. Tale limite diviene più severo al decrescere della qualità costruttiva ed al crescere del livello di sismicità del progetto. L’origine di tali limitazioni risiede nella considerazione che maggiore è la lunghezza degli edifici maggiore risulta il loro impegno sismico, anche in termini di spostamenti. Tuttavia, se l’edificio è ben calcolato e dotato di buoni dettagli costruttivi, specie a livello delle fondazioni, l’azione sismica sulla sovrastruttura diminuisce di circa il 50% rispetto al caso in cui siano presenti fondazioni di ridotta estensione.

Nel lavoro si conduce uno studio sul modo in cui le onde sismiche colpiscono le costruzioni, specie nella zona epicentrale di terremoti superficiali. L’eccitazione di base che interessa l’edificio si presenta in modo differenziato lungo la fondazione, sia nella forma che nella fase. Dapprima sono eccitate le parti della fondazione più vicine all'epicentro e poi quelle più lontane. Si intuisce che quanto maggiore è la diversità delle eccitazioni agenti all'inizio ed alla fine della fondazione tanto minore risulta l'azione sull'edificio. La diversità tra le eccitazioni è descritta dal coefficiente di correlazione, ricavato dalla funzione di coerenza tra i due segnali. Quanto minore è tale funzione tanto più elevata risulta la riduzione dell'effettiva eccitazione sull'edificio.

Negli ultimi anni la disponibilità di segnali accelerometrici digitali, registrati in occasione di forti terremoti da strumenti disposti su edifici di geometria e configurazione note, ha consentito uno studio approfondito dell'evoluzione dell'eccitazione con la distanza dall'epicentro e della risposta in funzione delle caratteristiche dinamiche delle strutture e di numerosi altri parametri, compresi quelli relativi alle fondazioni. In questo lavoro si citano brevemente alcuni riferimenti bibliografici su questi temi, in particolare quelli riferiti alla variazione dell'eccitazione con la distanza. Successivamente, si considerano i cosiddetti *disastri ritmici* osservati in occasione di terremoti violenti, relativi cioè all'estrema variabilità del danno riscontrato in edifici simili, spesso assai vicini e con uguali condizioni del terreno di fondazione. L'effetto su tali costruzioni varia dal crollo totale fino alla quasi assenza di danno nell'edificio contiguo. In costruzioni di grandi dimensioni, d'altro canto, non è raro osservare crolli in alcune porzioni dell'edificio mentre il resto risulta praticamente intatto. Questa circostanza non appare dovuta al fatto che tali porzioni della struttura fossero di minore qualità e resistenza di quelle rimaste integre, ma all'incremento dell'eccitazione dovuto alla convoluzione delle *body waves* e delle *surfaces waves*. È lecito pertanto ritenere che se le fondazioni di tali edifici fossero state ben collegate tra loro ed avessero posseduto elevate caratteristiche di rigidezza e di resistenza, i gravi danni riscontrati sarebbero stati evitati.

Per verificare, seppure in via semplificata, questa ipotesi è stato considerato, per via numerica, il caso di travi di fondazione di diversa lunghezza (25, 50, 100,150 metri) eccitate da un'eccitazione sismica in punti prefissati e considerando diversi valori della velocità di propagazione. Le eccitazioni considerate sono diverse tra di loro: una è generata artificialmente (di breve durata e con un contenuto prevalente di armoniche ad alta frequenza), l'altra è naturale e di lunga durata, con contenuto prevalente di basse frequenze. Assumendo come parametri di confronto quelli descrittivi della risposta in corrispondenza della mezzeria delle travi, si è riscontrato che l'ampiezza spettrale della risposta è pari fino al 50% di quella relativa al *free field*.

A conclusione del lavoro si presentano alcuni schemi operativi sulle fondazioni, impiegati nei casi reali prima citati, in particolare per gli interventi sulla grande chiesa e sull'edificio industriale. In ambedue i casi le condizioni del terreno sono classificate come assai difficili. Il metodo descritto dà luogo ad interventi relativamente poco costosi e di facile applicabilità, senza interferire sulla funzionalità dell'opera e senza richiedere azioni di restauro non strutturale. Esso può essere applicato a sistemi massicci in zone ad alta sismicità. Va sottolineato che nel caso di beni monumentali si può pervenire a riduzioni dell'input sismico sull'opera dell'ordine del 30-50% unicamente intervenendo sulla fondazione, senza gire sulla sovrastruttura per accrescerne la resistenza e senza ricorrere a costose opere di isolamento di base.