UNIVERSITA' DEGLI STUDI DI MESSINA

Dipartimento di Ingegneria

DOTTORATO DI RICERCA IN INGEGNERIA E CHIMICA DEI MATERIALI E DELLE COSTRUZIONI – XXXII CICLO

NEW FRONTIERS FOR SEISMIC ISOLATION SYSTEMS

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It is always an emotion arriving at the end of a journey, thinking all efforts that you have done to build up your opportunities, to develop your ideas and to achieve your objectives.

This doctorate represented for me a very intense moment of my life, probably the closing of a circle that made me grow up and that is providing me with new energies for the opening of the further phase of my carrier.

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Introduction

Construction engineering counts different approaches to the design of structures that ensure reliability and stability towards any types of external actions. Surely, in this regard, seismic issue is one of the most relevant due to the dangerous effects that this phenomenon can originate. Ductility is the basic concept that leads to the design technique of conventional (not isolated) buildings through the capacity design method that aims to concentrate the majority of the dissipative features in well-defined parts of the structural elements, following a hierarchical logic that provides adequate protection towards brittle failures and abrupt collapses.

Furthermore, in the last decades, new design approaches based on the seismic isolation techniques have emerged vehemently. Instead of tackling the seismic forces with the structural strength, these techniques aim to contrast the earthquake effects breaking down the energy transferred by the ground to the structures.

Thus, the core idea of these new techniques is completely different because they target to tackle the problem at their roots, preventing buildings to undergo high level of stress and reducing dramatically the striking seismic energy.

With base isolation system, both super-structure and foundation of buildings can be designed not to overcome the elastic conditions ensuring the absence of significant damages in the occurrence of the design earthquake. Consequently, structures are designed in order to cope violent earthquakes without incurring in any serious damage.

Seismic isolation is one of the most relevant topic in research area and it counts different branches and approaches. During the time, different devices have been designed and plenty of experiments have been conducted as well as several attempts have already been put in place in real constructions.

Based on the huge amount of experience on this subject, the advantages universally accepted for this new technique are recognized to be:

- high decrease of the inertia forces activated by the earthquakes that lead to a drastic reduction of the stress in the major structural elements. Thanks to this, the structural frame of the constructions are protected against damages even after intense events;

- noticeable decrease of interstory drifts, that secures as well non-structural components against damage and allows building to work properly during earthquakes;
- reduction of vibrations in the super-structure that lower feeling of fear during earthquakes preventing panic and incoherent behaviour of the occupants;
- major protection towards the content of buildings as well as towards their operability post-event. This point is particularly important especially in regard to emergency management authority buildings, hospitals, firehouses, etc.
- less repairing costs after-shock or maybe none depending on the severity of the earthquake and the base isolation system response.

Seismic isolation is nowadays a mature technology that can become the most common and reliable earthquake protection system, not only for strategic buildings, but also for residential housing.

In the last decades, the points that hindered the diffusion of these techniques were basically three:

- a not in-depth knowledge of the behaviour of the isolated constructions;
- a lack of official technical regulations on the matter;
- the cost impact for isolators.

Whether in the first two points, to the present day, the problem is already sorted out thanks to the evidences of the experiments carried out and the update of the regulatory frame, about the third factor there are still hesitations, mostly by the costumer side, because cost/benefit rate for base isolation technique can show, depending on the case, a discouraging increase of costs.

However, recently the culture of prevention is more and more developing and a greater social awareness of the earthquake issue ensures the inexorable large-scale dissemination of such technique.

On the other hand, science keeps pushing in the direction of a continuous progress of the seismic isolation techniques both refining the current solutions and experimenting new devices and new design approaches.

Among new experimental proposals, isolation systems based on the concept of Seismic Metamaterial reveal to be very promising, getting increasingly attention from the scientific community.

Metamaterials are a sort of composite material that can interact with acoustic waves acting as a filter inside a frequency region, called bandgap. Many researchers have already tested their ability to attenuate seismic waves through theoretical demonstrations, numerical models as well as with full-scale experiments.

Additionally, several approaches have been already proposed as for instance seismic cloaks, vibrating barriers, sacrificial structures or antiseismic devices, all of them with the intent to isolate buildings or constructions in general, even up to entire portions of territory.

The current challenge in seismic Metamaterial environment is to find the most effective way to have this technology viable, reliable and serviceable in order to constitute at least a competitive alternative to the already existing anti-seismic technologies.

In this thesis the leading concepts of earthquake engineering designing are introduced, focusing on the national and international regulatory frame in force and following the logical path that guides the design of constructions in a seismic environment.

Then, two different approaches of seismic isolation systems based on the concept of Metamaterials are discussed: systems embedded with the ground and systems tuned with structures. For both systems, the working principles are presented and remarks are expressed in the light of the results coming from field investigations and experimental tests carried out about the two methods.

Specifically, as regard to systems connected with the ground, an extensive investigation on ground sub-surfaces was planned in order to collect field data of potential application sites and to relate such data with the assumptions of the isolation systems under study. Such research phase was conducted in two steps: at first in the context of an infrastructural engineering project performed in Tbilisi (Georgia) by a Joint Venture of Italian engineering companies, and then, during a period of study in the Institute of Earth Science and National Seismic Monitoring Center of Ilia State University, in Tbilisi.

As regard to systems connected directly with structures, an experimental session of tests on rubber specimens has been carried out in the Laboratory of Messina.

All results and considerations turned out from these phases has been merged with the foregoing experiences to advance in such a challenging intent of opening new frontiers of development for seismic isolation systems.

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Chapter 1

MODERN ANTISEISMIC CONCEPTS FOR BUILDING DESIGN

1.1 EVOLUTION OF THE ISOLATION TECHNIQUES FROM THE FIRST ATTEMPTS TO NOWADAYS

The concept of seismic isolation has very old roots since earthquakes have always been a shocking phenomenon for humankind and one of the most important issue to face since ancient times.

The first attempts to deal with this subject dates back to around 2500 years ago, when many Greek temples and wall fortresses were built with a protection system consisting of a designed stratifications of ground material or alternate layers of coal and sheet of wool [1]. Contemporarily in South America, ancient populations realized constructions on top of foundations made of soft strata and stones [2], but many other examples [3] [4] of rudimental isolation are present in Italy and China as well.

The more relatively modern attempts [4] [5] [6] of antiseismic constructions come from the end of the XIX sec. In 1891, K. Kawaj realized foundations of building where layers of concrete sandwiched wood logs. A similar idea was experimented in 1906 by Jacob Bechtold who proposed to build constructions on top of rigid plates provided beneath with a layer of spherical bodies to carry the base-plate freely. In 1897, Westwood proposed the first attempt conceptually near to the modern Friction Pendulum System. This consisted of a sliding system constrained to move on curved surfaces. Subsequently, in 1909, J. A. Calantarientes proposed another idea of base isolation system where structures and foundations where separated by a layer of talc.

In 1915, in Japan, F. L. Wright realized the Imperial Hotel of Tokyo using a revolutionary system of foundation made by floating piles in a soft layer of silt, whose deformability was supposed to isolate the construction that actually got through the strong earthquake of 1923.

In 1929, R. R. Martel proposed a construction technique called "*Flexible first story concept*" that aimed to increase the flexibility of the building by inserting some flexible columns in the first floor. Similarly, Green (1935) and Jacobsen (1938) elaborated "*The soft first story method*" as a technique to build earthquake-proof buildings on the basic idea to absorb the energy through plasticization of pillars in the first floor. However, these proposals failed in the practical applications they had, because such flexibility all targeted in the first floor led to a weakness of the building that entailed the collapse of the structures. Such occurrence is well known nowadays and it takes the name of soft-story collapse mechanism.

Focusing now on the more modern attempts of antiseismic applications, apart from some simple and still basic trial in the Ex- Soviet Union around sixties, an interesting trial for the scientific community was made in 1960 in Skopje (Macedonia) where a school was provided with a prototype of the present-day elastomeric bearings. The isolation system consisted of five bearings in not-armed rubber. This created many inconveniences because of the low vertical stiffness that was comparable with the horizontal one. The structure suffered strong rocking problems in the occurrence of non-seismic horizontal forces like the wind.

In seventies, the *Malaysian Rubber Producers' Research Association* (*MRPRA*) manufactured the first modern example of elastomeric devices in armed rubber using the vulcanization method with rubber and iron sheets. Just some hears before, in 1967, Penkuhn proposed the first patent of modern friction pendulum system. Many applications arrived back-to-back trying to improve constantly the features of such type of isolators. The main applications took place mostly in France, New Zeeland, Japan, Italy, USA.

From the eighties on, the subject of the antiseismic isolation received a strong boost tanks to lots of development projects and experimental campaigns performed all over the world that brought us to the modern era of earthquake engineering.

Nowadays, antiseismic techniques can distinguish in three different categories [7]:

- Active control
- Passive control
- Hybrid control

Active control systems consist of sensors or electric devices that monitor structural response of buildings. Such devices activate in the occurrence of a quake event and enable forces in the structure to the contrary.

Very often, this kind of control is executed through hydraulic actuators that govern the motion of inertial masses. When the system detects a change in acceleration, velocity or displacement of the structural response, such devices activate these masses in a counter-phase motion that opposes to the vibration provoked by the wind or the earthquakes.

It goes without saying that such a mechanism need a high-level maintenance over a long time. In fact, in the event of a failure of the system, the building will be totally free of protection and therefore extremely vulnerable. Such technique can be tuned to a very large range of frequencies.

Passive control systems consist of putting in place strategy for reducing the incoming seismic energy and/or in dissipating the energy that has already entered by means of appropriate elements. This category includes the isolation system as such and it is the most largely used strategy in earthquake engineering.

Dissipation elements can be metallic, friction, viscoelastic solid, viscous fluid, tuned mass (TMD) or tuned liquid (TLD). Such technique exploits only the mechanical features of the antiseismic elements. *Table 1* summarizes the different types of devices or elements used for dissipation.

Hybrid control is a combination of active and passive control. This system requires anyway a strong control for the system arrangement, then it is also known as semi-active.

In the hybrid control, the active control forces are supposed to enhance the effectiveness of the passive system. Despite of the active systems, the hybrid ones do not need a high level of power to run and, in any case, the presence of passive devices guarantees a level of protection against quake events.

Category of device	Basic principle	Materials and technology	Performance provided
Hysteretic devices	Yield of metals Friction	Iron or lead Connection metal-metal or non-metal	Energy dissipation Increase in resistance
Viscoelastic devices	Deformation of viscoelastic solids Deformation of viscoelastic fluids Fluid extrusion	Viscoelastic polymers Highly viscous fluids High technologic designed fluids (orifices, seals)	Energy dissipation Increase in resistance
Re-centring devices	Pressurize fluids and orifices Friction-spring Phase transformation materials	Compressible fluids, high pressure seals Connection metal-metal or non-metal Memory form alloy, super elastic behaviour	Energy dissipation Increase in resistance Re-centring capability
Dynamical vibration absorber	Oscillators with tuned masses Oscillators with tuned liquid	Mass-spring-dissipater fluids Water reservoir	Increase in damping Increase in damping

Table 1 - Summary of dissipation devices for passive systems

1.2 CURRENT TECHNICAL STANDARDS: FROM THE CONCEPT OF RESISTANCE TO THE BASE ISOLATION TECHNIQUE

The most up-to-dated antiseismic standards, referring to conventional buildings, aim to prevent the structural collapse in the occurrence of a strong earthquake and to restrict damages in the non-structural elements in the occurrence of a medium intense earthquake. They achieve this objective by defining appropriate structural parameters like stiffness, resistance and ductility, and by using a design procedure that fit case by case with the levels of stress/strain caused by the external forces and with the construction techniques of the structures.

Engineer’s general approach is to try to maintain the structures in elastic field, where the structural behaviour is always easier to be predicted. However, in earthquake engineering, because of the level of stress/strain provoked, it is usual to exceed elastic conditions and reach plastic behaviour.

In such a case, the main issue is to keep under control the plastic behaviour of the constructions and the potential damages that might lead to the collapse of the structures.

Conventional constructions that meet the modern antiseismic standards base their conception on the criterion of facing the seismic worse-case events by their own structural resistance capacities. This strategy is the simplest and the most straightforward approach, identical for all the forces acting on the structures, like gravity, wind or snow: as much solicitations rise as resistance must increase accordingly.

However, in such a case, many aspects make this strategy not convenient and sometimes even dangerous. That is because increasing the structural resistance entails having often a massive construction whereby details of the connection elements are extremely tricky to be manufactured and failing a proper realization of these parts could jeopardize the capability to get through a strong event.

Furthermore, these types of structures turn out to be generally economically not convinient for many reasons. First of all, strong earthquakes probability of occurrence is pretty low so that maybe the structure might not to be struck over its design life. Besides, these structures are conceived to face quakes through their own damage and then the aftershock result lead likely to have a structure to be repaired.

In the last decades, a new strategy for the earthquake engineering was proposed in most of the international codes. This approach origins from the consideration that structures can have more resilience whether the process of failure is handled conveniently during strong events.

In fact, when a conventional structure collapses because of an earthquake, often it is because only one or a few localized parts of the structure reach the yield whereas all the remaining elements are still in elastic conditions then they are not exploited to the bottom.

The new strategy shifts the focus from resistance to ductility. A ductility design, as a matter of fact, enhances structural resistance against earthquakes, but the prize to pay for this is again in terms of damages because seismic energy starts to be dissipated when the structural elements turn in non-linear field, bending and cracking themselves to take advantage of all their own ductility capabilities [8] [9]. As in the forgoing case, construction details play an essential role and therefore they need special care both in design and in construction phase.

The most relevant international codes adopted this criterion as the basic for the antiseismic constructions, calling it "*capacity design*". The principle consists of providing structures with a high level of global ductility through a design that aims to develop plastic mechanisms following a prefixed order or concentrating the dissipation within specific parts of the structures.

In all circumstances, the criterion decreases solicitations in the most of the structural elements, excludes brittle failures, like shear failure or fatigue failure, preferring bending failures in beams rather than in the columns and finally provides an advantageous consistency in the structural response.

Some techniques for earthquakes engineering use specific elements called braces, placed in the meshes of the loom structures, to concentrate on them the horizontal forces and dissipation [6]. *Figure 1* displays the more usual layout of braces in a building frame.

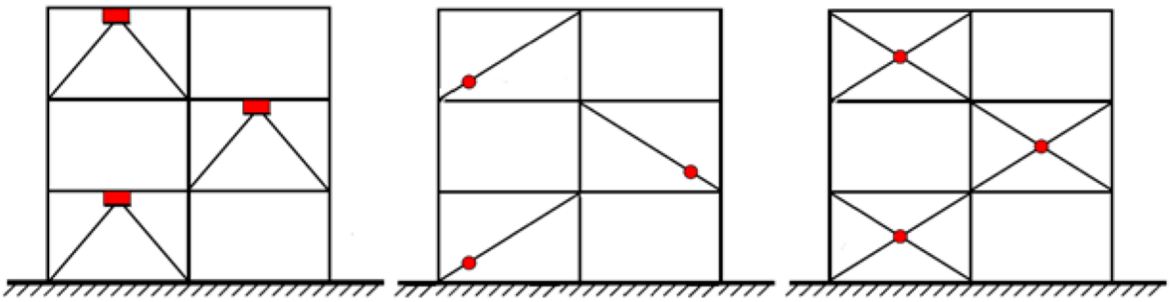


Figure 1 – Location of dissipative braces in building frames ([6])

Their effect during dynamic motion is to contrast seismic forces by means of their stiffness and, additionally, isolating some specific parts of the structural elements (red parts shown in *figure 1*), in the braces themselves or in the beams, where all the dissipation can develop. Obviously, such parts of the structures are the ones with highest potential of dissipation and the less critical for stability. Since dissipation means damage, these parts are designed to be replaced aftershock in the event that they get damaged.

That is the reason why the most recent versions of braces present dissipation tools properly conceived to operate during dynamic motion without incurring in any damage then without benefit of the ductility reserves of the structure.

Dissipation tools can be viscous or hysteretic. In the first case, they accommodate the natural motion of the structures that remain conveniently in elastic condition but the dissipation of the whole system is improved. In the second case, dissipation tools have an elasto-plastic behaviour and the maximum force for these elements is supposed to be the parameter to design the other structural elements connected with them, under the logic of the capacity design process.

In addition of the cited above techniques, technical standard have introduced the concept of base isolation system as an alternative for the last decades. Of course, worldwide the introduction of these new criteria for earthquake engineering have a different development and timing from country to country and there are still some of them that do not have yet a regulatory framework for the matter. However, technical standard are giving more and more prominence to the base isolation technique, since it is reckon to be the most convenient solution to face the problem of earthquake.

This strategy can reduce dramatically the incoming energy by decoupling the structural response of the foundation ground to the one of the structure by means of the introduction of specific elements, called isolators, appropriately conceived and inserted as portrayed in *figure 2*:

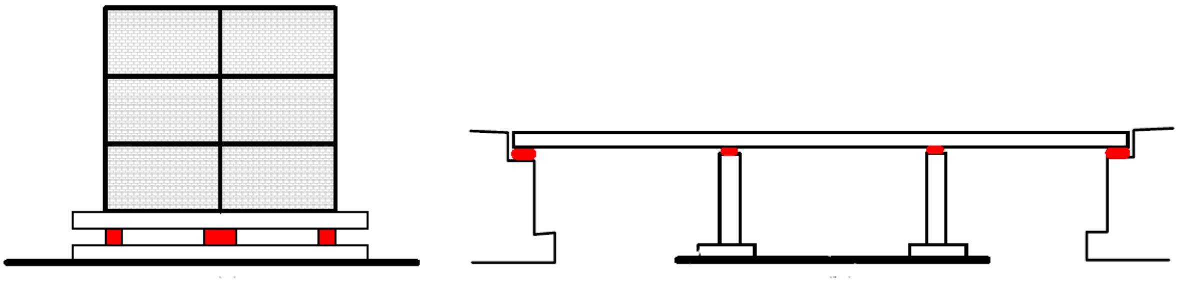


Figure 2 – Example of isolator introduction in buildings and bridges ([6])

Such a technique can be applied to new constructions as well as to the existent ones. Standard rules require two ways how to exploit base isolation system: the most common way is to provide deformability to the structure through isolators in order to disconnect the building with the ground. The second way is limiting the maximum horizontal force that the ground foundation transfers to the structure, again, through isolators. Both strategies can be used together and both can be optimized exploiting dissipation in the isolating devices.

Surely, since the structural safety relies on a small number of isolators, standard impose that such elements must be designed on a higher resistance level and all the devices to be put in place must have very strict certifications about their quality.

Basically, new anti-seismic techniques aim to decouple the dynamical motion between structures and their ground foundations, as a result of the introduction of specific devices called isolators that create an isolation floor.

Accordingly, the entire construction can be considered as divided into two parts: the superstructure (everything is above the isolators), and the substructure (the foundation beneath the isolators, directly connected with the ground), as described in *figure 3*.

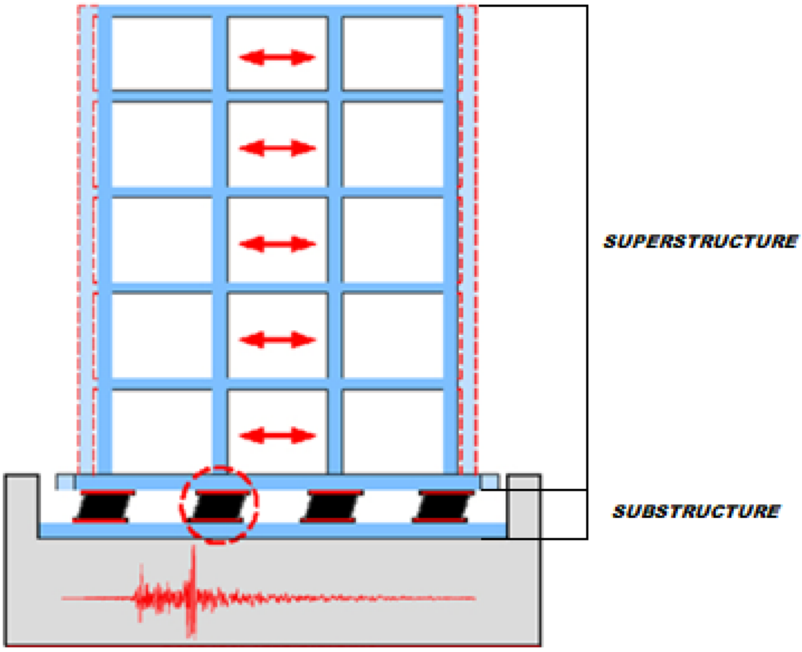


Figure 3 – A concept of base isolation system

As a rule, substructure remains very robust and it is subject to the same *PGA* (Peak Ground Acceleration) of the ground, conversely superstructure benefits of the isolation frame that provides deformability to the entire system.

This is the main effect of the base isolation frame: a higher deformability gets a shift of the principal resonance frequencies of the structures towards the high period region of the response spectra, where usually PGA values are significantly lower. Thus, the problem of resonance between the natural frequency of the main vibration mode and the main harmonic components of the seismic signal will be avoided [5] [10].

Indeed, the standard shape of the acceleration response spectra shows a peak of amplification (plafond) at low period values of approx. *0,2/0,8 sec*, exactly where the natural periods of the majority of the constructions are, and then, subsequently, a constant decrease with the period growing. Assuming an elastic behaviour of the isolators, the natural period of the entire system (composed by superstructure, isolation system and foundations) can be easily pushed over *2 sec* period values, depending on the isolation frame tuning. Forcing isolation tuning, natural periods can reach as well *4 ÷5 sec* values.

Although, this considerable advantage in terms of acceleration acting on the superstructure counterbalances an emerging disadvantage because the whole system will undergo to a rise in terms of displacements. Nevertheless, such a large displacements will be concentrated in the isolation system, where the most of the input energy will be trapped and dissipated.

Figure 4 describes the effects of the base isolation system on the design parameters for a building [11].

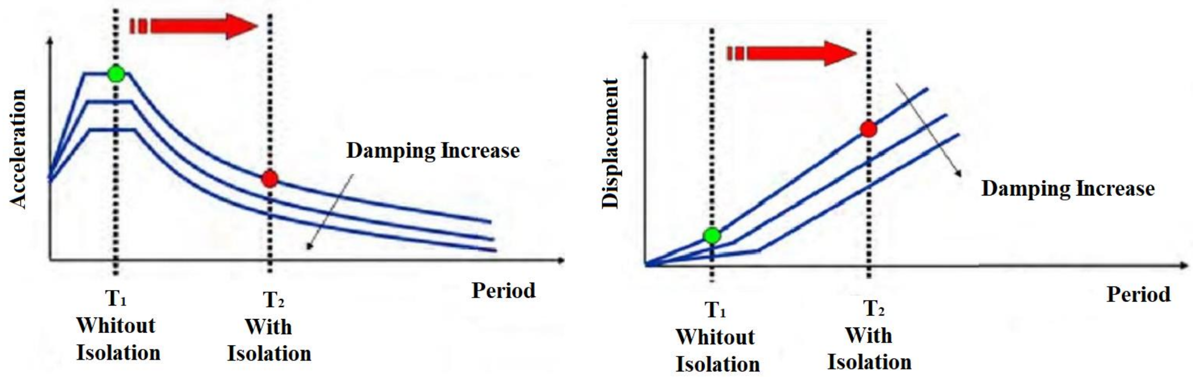


Figure 4 – Effects of the base isolation system with respect to design parameters ([11])

As a result, the base isolation technique leads to select acting accelerations substantially lower comparing with the traditional techniques. In traditional techniques, the elastic response spectra are reduced by the capacity factor (taking into account the ductility reserves of the structures) to become design spectra. In the base isolation technique, the elastic response spectra essentially do not change to become design spectra. The latter is a great advantage for the engineers because they can analyse structures dynamic motion in the elastic domain, simplifying dramatically their calculation and, above all, keeping the phenomenon under control easily.

All the countries affected by huge problem of earthquakes have supplied themselves with codes and standard that regulate the isolation technique. *Eurocode* (*EC8*) [12] as well as the *Italian code* (*NTC 2008* and new version *2018*) [13] [14] cover this topic in specific chapters and envisage the application of the isolators in bridges and buildings construction.

As per codes, superstructure and substructure can be regarded as having elastic behaviour in the modelling phases. It is worth to stress the fact that this is a remarkable advantage for the engineers. Indeed, maintaining the structural behaviour in elastic domain rather than in plastic/inelastic conditions is undoubtedly convenient and manageable because the more calculation modelling is closer to the real behaviour of the structures during earthquake, the more predictions will be reliable.

On the other hand, as per codes, isolators can be considered either viscoelastic or with nonlinear elastic properties, and even linear equivalent elastic accordingly with its own mechanical features. In the event that modelling of the structure in elastic condition is not allowed, codes prescribe to use models where non-linearities are concentrated only within the isolation system. Again, as per codes, details of the structural elements in an isolated building do not need to be burdensome and difficult to implement than in a conventional ductile structure. This aspect is definitely very convenient because simplification of details means both reduction of construction costs and increase of the reliability of the construction phases that are often crucial to achieve good structural performances during earthquakes.

As mentioned above, the benefits of the isolation technique can be obtained in two ways basically: the first one (mostly used in building design) by shifting the natural period of the structures to the regions of the spectra where accelerations are lower, as shown in the foregoing *figure 4*. The second one is by setting a maximum permitted value for the force transferred from the ground to the structures. The logic is the same than for the first case but in the limited force approach the point of view changes, therefore the main requirement is the maximum force permitted at the expense of the natural period of the structure that will be adjusted accordingly. In both cases, damping plays a beneficial role in limiting both seismic accelerations and displacements. Actually, dissipation energy of isolators prescribed by the standards varies between *2 ÷30 %*, much more comparing with values of *2 ÷10 %* for traditional not isolated structures.

Damping plays a very relevant role in this context because it is the key factor to handle displacements-related issues as the presence of other structures in the surrounding of the isolated building or regarding to the correct functionality of the facility systems.

Italian codes for anti-seismic design require to fulfil two performance levels: *SLD* in terms of drifts (displacements among decks) and *SLU* in terms of strength on the structural elements (with *capacity factor* usually equal to *1,5*, featuring elastic behaviour).

Higher quality and reliability is required to the isolator devices because of their critical role. For their verification, codes prescribe to fulfil high security requirements comparing with all the other structural elements by introducing a further performance level with higher return period of earthquakes. Thus, isolators are designed to resist at a bigger design earthquake, more intense comparing with the one acting on the structure.

A base isolation system can consist of a combination of seismic devices working together in order to give an overall result that secures buildings from earthquakes as well as common external actions and guarantees its functionality without interfering with other surrounding constructions.

Many requirements are demanded to a base isolation system. From the operational point of view, a base isolation system must ensure appropriate vertical bearing capacity as regards to both seismic and static conditions, high horizontal deformability in the occurrence of an earthquake but adequate stiffness for non-seismic external actions like wind, impacts or vibrations. Additionally, it must ensure good damping behaviour, durability and a consistent behaviour over the time and in every working condition.

Finally, isolation system must guarantee re-centering capabilities after-shock in order to get back to the original working configuration. Apart from these strict requirements concerning their functioning, there are many other important features that engineers need to examine during the designing phases, like installation easiness, floor space and affordability.

1.3 APPROACH TO THE DESIGN OF BASE ISOLATED STRUCTURES

The concept for the design of isolated buildings need adjustments to the usual practise for non-isolated constructions. The first step is the assessment of the seismologic-geotechnical conditions of the site in order to determine the expected design events. As a function of the earthquake' features, geotechnical conditions and building features, engineers can evaluate the opportunity to use the isolation base technique and which type of device putting in place.

However, practicability and efficiency of the base system isolation is actually a delicate subject because sometimes earthquakes can have peculiarities that make them different to the ones prescribed by the codes.

As a general rule, building and ground foundation have a favourable decoupling when the natural period of the isolated structure is at least 3 times the one of the non-isolated structure. The more the increase of period is, the more high is the reduction of the spectral forces acting on the superstructure. Usually obtaining period values beyond *2 sec* is considered satisfactory. The price to pay for this, on the other hand, is in term of displacements. Although, as already explain, such large displacements are supposed to be concentrated on the isolation system, but they must be taken into account carefully during the design phases for practical and constructive reasons.

About practicability and efficiency of the base system isolation, situations requiring special attention are when the expected earthquakes have got energy contents at high values of period. In such cases, resonance phenomenon between the isolated structures and the external acting force can occur making isolation effect completely unfavourable.

These cases arise when earthquakes are expected to have got high magnitude and in a propagating situation of near fault and when the earthquakes origin from deep fault (more than *100 km*), especially whether the superficial deposit of the site foundation is deformable (and as much as this layer is thick).

In the first case, seismic signal is characterised by single pulses in low frequencies that move very quickly across the ground. As a consequence, the response spectra show picks at the high periods even beyond *2 sec* with considerable displacements in case of isolated structure. Additionally, vertical accelerations might occur and favour dangerous tensile stress in the isolation devices. In the second case, signal are characterised by long duration and highly intense pulses at the low frequencies with spectral accelerations around *2 sec* values.

It is apparent that such kind of earthquakes have a dangerous nature that can threaten a wide range of structures, both non-isolated and isolated. In such a conditions, efficiency of the isolation system must be assessed carefully because this technique might not be the most suitable solution against earthquakes.

In all circumstances, the features of the isolation system must be well-calibrated eventually pushing the natural frequency of the structure over the high values period (*4/5 sec*) or, alternatively, adopting a strategy that aims to limit transmitted forces in the superstructure. In all these cases, the selected devices must be able to bear large displacements, even larger than the ones prescribed by the codes for isolators.

Deformability is a parameter that is always relevant for the choice of the isolation system and it must be considered at any level. Above, it has been mentioned the influence of deformability in particular kind of situation where expected earthquakes have specific characteristics. Although, also in case of usual design earthquakes happening on sites where ground foundation consist of a deposit with high deformability (layer from about *10 meters* to *100 meters*, or even more), the choice to use the isolation technique can be not effective. Indeed, soft soil layer foundations always amplify spectra, reducing considerably the beneficial effects of the isolation system, as shown in *figure 5*. First attempts of isolation system, in the past, were based exactly on this principle.

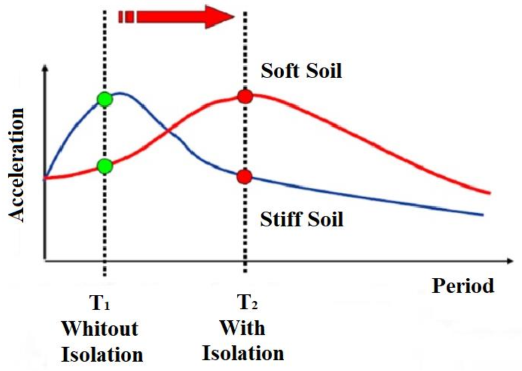


Figure 5 - Effect of soil deposit deformability in the response spectrum ([11])

For the same reason, tall buildings and in general slender constructions do not need usually a base isolation system against earthquakes because such type of structures present already high deformability and, therefore, high natural frequency that would make the isolation system practically useless to this end.

The other crucial parameter to account for it to assess the convenience in adopting a seismic base isolation system is displacements. Displacements, as already said, can be very large but they are expected to be concentrated within the isolation system. Damping is a proper way to keep them under control, avoiding that they will reach very large measures under dynamic motion. Just to give an idea of the size of the problem, comparing displacements in isolated structures and in non-isolated structures, the gap can reach easily one order of magnitude.

In this context, buffer areas among adjacent buildings must be properly designed in order to avoid that constructions can slam from one another under dynamic motion, destroying themselves (pounding phenomenon). For the same reason, another important design aspect is to keep under control relative displacements between superstructure and substructure, namely attributing an appropriate extent of the free space between the superstructure and the boundary outlining the foundation area.

Obviously, isolator devices constituting the isolation system must be able to absorb the expected displacements in safety conditions. Additionally, they must have re-centering capability after-shock in order to allow the building not to lose is functionality.

Functionality is an essential factor to take into account for a base isolation design. Facilities serving the building (electric, hydraulic, gas, etc.) as well as the connecting works (mostly vertical rather than horizontal) like stairs or elevators need special attention. All these elements, structural and not, must be designed in the perspective of not to obstacle the relative displacements and, in their turn, not to be damaged by the same. Connection joints play a very important role to this end.

Also the selection of the typology of isolation system to adopt in the project is not a trivial issue, since it depends on which kind of application has to be put in place (if new construction or reinforcement of an old building, characteristics of the seismic inputs and site conditions, intended use of the construction, etc.).

Moreover, the decisional process in the designing phases implies always compromise decisions among the most important structural factors. As an example, the reduction of the maximum value of shear stress in the superstructure entails a rise of the displacements in the isolation system and sometimes, according with the type of isolation device selected, the emergence of significant accelerations at high frequency. The latter can play a role in modifying substantially inertia forces throughout the entire height of the building and then consequently the inter-story drifts as well as the shear forces acting at each floor, providing an unusual behavior under motion.

Selection of the type of isolator devices depends on which one, among the design parameters, is considered as the most critic. With reference to isolated multi-story buildings, it is possible to distinguish three different performance objectives to fulfill during the design phase:

- Minimizing the shear forces acting on the structure;
- Minimizing displacements on the structure;
- Minimizing accelerations forces at each floors at high frequency (more than *2 Hz*).

The objective to minimize the acceleration forces at high frequency are relevant when the protection towards the content of the buildings is the crucial point. In such a situation, the best choice could be the use of linear or quasi-linear devices. Unfortunately, as is normally the case, the optimum design choice is not always the most convenient from the economic or technical-executive point of view.

Other guide parameters are consistency of the device behaviour over the time and in any situation as well as their service life. About this last point, it is worth to mention that all the isolation devices have a service life (by codes) shorter than the structure below which they are placed.

This last point brings to the attention the matter of isolation system costs which is an issue that need a thorough analysis because it is considered the main reason why such technique was not extensively implemented so far.

The circumstance that an isolated building costs more than a non-isolated one depends on many aspects: dimensions, building configuration, number and types of isolators consisting the system, etc., and the number of floors, above all.

Definitely, purchase, installation, acceptance test, as well as all other operations necessary for the isolation system arrangement have a cost that non-isolated buildings do not present. Generally, the more the number of floors increases, the more such extra costs can be spread on many floors lessening their incidence in the total amount of the construction costs.

Nevertheless, buildings that are not expected to undergo important level of stress, as isolated buildings are instead of non-isolated buildings, must comply only the minimum requirements prescribed by the standards about size of the structural elements and bar reinforcements. If the number of floors is small, the incidence of the isolation system can still be significant, reaching significant portion of the total amount of the building. Whereas, if the number of floors is not small, the extra costs of the isolation system can be absorbed by the saving in quantities of material (principally steel), so much to turn out convenient.

However, the real strong convenience of the isolation system is in cutting dramatically the reparation costs post-event, in case of earthquake occurrence, likely wiped them out. In this regard, it is relevant to highlight that reparation costs of a conventional building can be hugely expensive, considering that in the worst-case scenario the only way to make again usable a damaged building is by demolishing and rebuilding it. On the other hand, isolators need maintenance, and such a maintenance, that usually consist of periodical inspections every *5 years* approximately and after the occurrence of an earthquake, add extra costs over the time.

At the end, isolation system reveal to be very competitive and cost-effective, especially in a long-term perspective even for constructions where apparently its installation entails some extra costs

1.4 DETERMINATION OF THE SEISMIC ACTION

Seismic design actions to be assumed for calculation and verification of the structures are imposed by the technical codes on the basis of the existing seismic microzonation maps and the condition of the ground foundation, determining the effect of the site-specific response analysis. Careful collection, recording and interpretation of geotechnical information are supposed to be provided as one of the initial step of the project.

Generally, geotechnical investigations shall deliver sufficient data concerning ground and ground-water conditions at and around the construction site for a proper description of the essential ground properties and a reliable assessment of the characteristic values of ground parameters to be used in design calculations. Additionally, in the event of seismic risk, the assessment of the foundation soil shall be carried out to determine the nature of the supporting ground to ensure that hazards of rupture, slope instability, liquefaction, and high densification susceptibility in the event of an earthquake can be managed.

In such a context, a proper assessment of the subject should include many aspects like geology, geomorphology, seismicity, hydrology and history of the site. To that end, big projects like infrastructural projects or suchlike usually rely on a useful instrument for the design, called *Geological Reference Model (GRM).*

Geological Reference Model describes the evolutive history of the area of interest, providing a reconstruction of the lithological, stratigraphic, structural, hydrogeological, geomorphological and, more generally, geological hazards of the site of interest. The geological model, thus, is a useful reference point for the designer to frame geotechnical problems and to define the geotechnical survey program. Geological Reference Model is an essential stage of projects that allow to have a comprehensive overview of the project and to predict the potential issues to face for the realization of the construction.

The elaboration of the Geological Reference Model need the cooperation of many different professionals of the project in order to take into account different aspects of the construction in design, like its functionality, limit state and structural performances required, etc. Moreover, such model is usually up-to-date and refined over the production of the project, in order to fit better and better to the design choices. The direct result of this process is the planning of the geotechnical investigation program. The scope of geotechnical investigation is to allow, from one side, the characterization of the soil design parameters for the extent of the ground that governs the problem and, on the other side, the determination of dynamic characteristic of the soil for the evaluation of the seismic design actions.

Indeed, dynamic motion provoked by earthquakes hinges on the site-specific peculiarities that modify the characteristics of the seismic signals along their propagation from the bedrock to the surface. The step of the estimation of such effects takes the name of seismic response analysis and need a detailed knowledge of topographic conditions, stratigraphy of the subsurface, physical and mechanical parameters of the different layers composing the ground foundation.

Technical codes recommend accepted procedures to perform a seismic response analysis by establishing both seismic input and reference site-specific conditions with the purpose of defining the external forces acting at the level of the foundation.

In general, seismic action depends on the typology of analysis utilized in the project for the calculation of the structure and it can be characterized as:

- Maximum acceleration;
- Maximum acceleration along with the pertinent response spectrum;
- Time-history of the motion.

When the subsurface shows an arrangement that can be simplified as a mono-dimensional problem, international standard identify some types of foundation soil accordingly to their stratigraphy and their average parameters, as described in *table 2*, extract from the *Eurocode 8* [12]. Ground types *A*, *B*, *C*, *D* and *E* can be used to account for the influence of local ground conditions on the seismic action. This classification is based on the average shear wave velocity $v_{s,30}$, which can be computed in accordance with *equation 1.1*:

$$v_{s,30} = \frac{30}{\sum_{i=1,N} \frac{h_i}{v_i}} \qquad \text{Eq. 1.1}$$

Where h_i and v_i stay for the thickness (in metres) and the shear wave velocity (at a shear strain level of 10^{-5} or less) of the *i*-th formation or layer, in a total of *N*, existing in the top *30 metres*. When such parameter is not available, in alternative, classification can be performed on the basis of other parameters that define the resistance of ground, N_{SPT} for cohesionless ground and

c_u for cohesive ground. However, technical codes recommend the usage of the shear wave velocity profile for determining the soil category.

In the event that ground conditions match one of the two special ground types *S1* or *S2*, standard require specific studies to be carried out for the definition of the seismic action. In the present case, advanced analytical procedures are required and the investigation of the ground parameters shall be adapted to this need, preferring, when possible, high strain laboratory tests and strain dependent measures for stiffness and damping.

Table 3.1: Ground types

Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	–	–
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s > 800 m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	–	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1			

Table 2 – Table 3.1: Ground types (Extract form Eurocode 8)

1.5 GROUND RESPONSE ANALYSIS

Local soil conditions have a strong influence in the ground response during earthquakes. When seismic quakes occur, ground motion parameters such as amplitude, frequency and duration of the signal change as the seismic waves propagate from the bedrock towards the overlying strata. The phenomenon wherein the upper strata of the soil act as a filter, modifying the ground motion characteristics is very relevant in earthquake engineering and all its branches.

From an engineering point of view, the objective is to predict the characteristics of the seismic signals at the surface or at any depth of the site of interest in order to be able to calculate properly the acting dynamic forces. Once accurately evaluated, they will be taken into account in all the aspects that an engineering project requires such as extra loads in the structures, ground stability, slopes etc. [15] [16]

The parameters involved in this study range in a wide variety. The most important are certainly nature and geometry of the depositional soil overlying the bedrock, slopes of the bedding planes of the soil or, as well as, of the bedrock, variations of the lithotypes in the site and also possibly variations of the features within each lithotype, presence of faults, fractures or voids, etc.

Calculating the site-specific dynamic response of a ground foundation is commonly referred to as site-specific response analysis or soil amplification study. Physically, the schematic models the phenomenon as a wave propagation problem across a continuous medium by virtue of the stiffness and the attenuation capacity of the medium itself. The analysis can be 1-D, 2-D or 3-D depending on the geometry of the problem and loading conditions. Surely, accuracy and completeness of the analysis change but not all the time they can justify the endeavour of performing a higher dimensional problem, therefore this choice has to be made case by case.

The analysis for the characterization of the seismic motion can be conducted both in terms of time and in terms of frequency. These two domains are physically dual to each other in dynamics context. In time domain, the main parameters are the peak values (acceleration, velocity, displacement) and the time histories duration while, in the frequency domain, the main factors are Fourier spectra and response spectra, therefore harmonics amplitudes for each specific frequency [17].

The schematic of wave propagation phenomenon from the bedrock to the surface is depicted in *figure 6*:

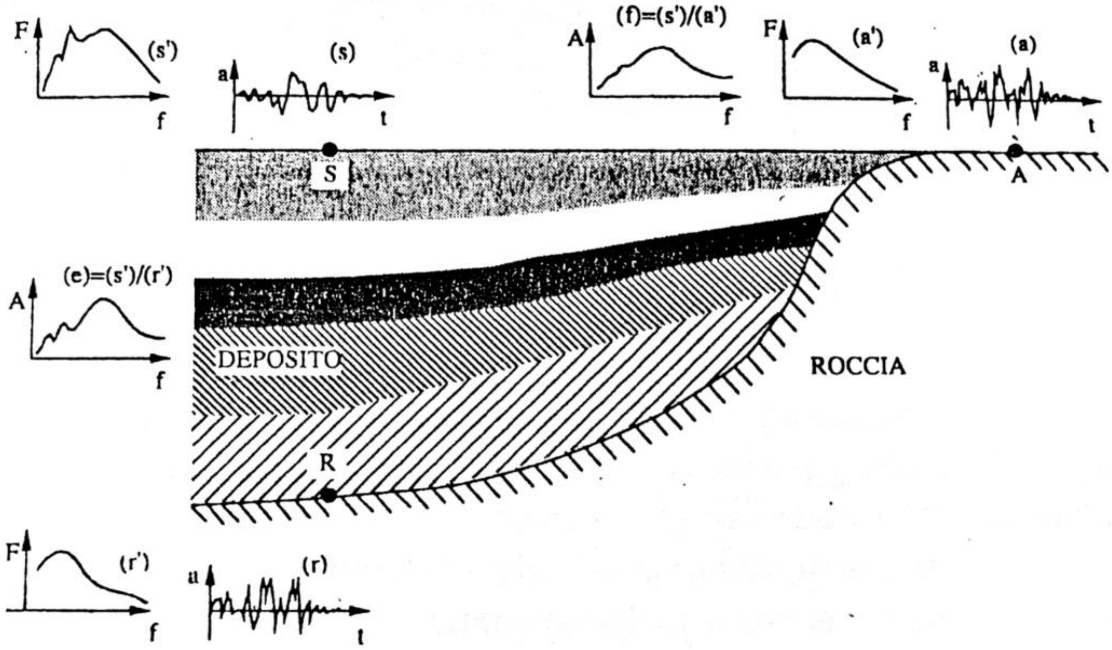


Figure 6 – Conceptual scheme for the soil amplification analysis ([17])

Site filter effect is strictly defined as the modifications that the signal encounters due to its propagation from the bedrock (point *R*) to the surface (point *S*). However, it is often physically meaningful taking into account also superficial measurements in outcrops (point *A*) since all earthquakes records come in fact from monitoring stations located onto superficial rock outcrops whereas records in deep seismic bedrocks are not available and they can be generated only through mathematical calculations.

The quantitative evaluation of the site filter effects is carried out comparing the significant parameters for both domains. In the case of time, the ratio between the peak ground acceleration in the surface ($a_{max,s}$) and the one at the top of the bedrock ($a_{max,r}$) or in alternative of the outcrop ($a_{max,a}$) is the relevant value and it is called amplification factor. Despite its own name, the amplification factor can be higher than *1* and lower as well, implying in such a case a de-amplification of the signal.

In the case of frequency, the ratio between the Fourier spectrum of the signal at the surface (s') and the one at the top of the bedrock (r'), or in alternative of the outcrop (a') gives rise to the transfer function, usually indicated by the symbol $H(f)$. From the latter it is possible to evaluate modifications of the amplitudes at each frequency of the signals.

Transfer function is generally a complex function and it is a very powerful tool because it allows the user to understand all the seismic characteristics of the system. Its module, called

Amplification function, permits to identify signal changes effortlessly. *Figure 6* shows the amplification functions in two above-mentioned cases, *e* and *f* respectively. The only restriction that encompasses the transfer function is that Fourier spectra presume for its calculation a linear constitutive behaviour for the soil. Under such circumstance, transfer function depends only upon deposit geometry and mechanical characteristics of the ground.

There are few simplified cases where the transfer function can be calculated by mathematical closed-form solutions. Such cases are elementary and they cannot describe properly the conditions and the behaviour of a real-case ground but they are precious because they can outline the part that parameters play in such context.

The ideal pattern consists of the mono-dimensional problem of a harmonic oscillation representing a shear wave, propagating upright from the top of the bedrock across a homogeneous horizontal deposit of thickness *H*. The sketch is shown in *figure 7*.

In the simplest case, bedrock is considered as infinitely rigid while the upper stratum is linear elastic.

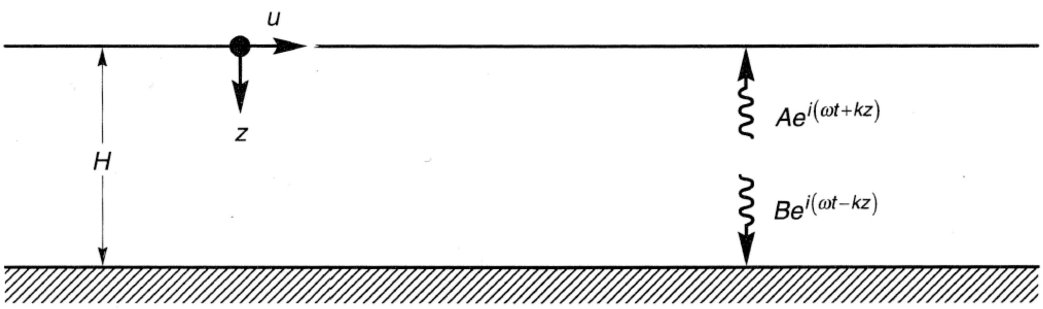


Figure 7 – Schematic of homogeneous elastic stratum above an infinitely rigid bedrock ([15])

The motion law is described analytically by the differential equation of dynamic equilibrium ruling the 1-D propagation phenomenon for an elastic medium, (*Equation 1.2*):

$$\rho \frac{\partial^2 u}{\partial t^2} = G \frac{\partial^2 u}{\partial z^2} \qquad \text{Eq. 1.2}$$

Where $u(z,t)$ is the horizontal component of displacement. When the acting force is a harmonic with angular frequency $\omega = 2\pi f$, the solution for displacements can be found by separation of variables using $p(z)$, so-called shape function, (*Equation 1.3*):

$$u(z,t) = p(z)\, e^{jkz} \qquad \text{Eq. 1.3}$$

Shape function describes the amplitude of motion of every point along the vertical while the Eulerian notation takes into account the flow of time, thus how this shape moves over the time. Replacing the latter in the general formulation (*Equation 1.4*):

$$G\frac{\partial^2 p}{\partial z^2} + \rho\omega^2 p = 0 \qquad \text{Eq. 1.4}$$

Wherein solution is *equation 1.5*:

$$p(z) = A\, e^{jkz} + B\, e^{-jkz} \qquad \text{Eq. 1.5}$$

Where $k = \omega/V_s = 2\pi/\lambda$ is the wavenumber, V_s is the velocity of the shear waves in the ground and λ is the wavelength. Thus, horizontal displacements have the solution as per *equation 1.6*:

$$u(z,t) = A\, e^{j(kz+\omega t)} + B\, e^{-j(kz-\omega t)} \qquad \text{Eq. 1.6}$$

Where A and B are the amplitudes of two waves propagating across the upper stratum, upwards and downwards respectively. Imposing as boundary condition the absence of shear stress in the surface $\tau(0,t) = 0$, the only solution able to meet the identity occurs when $A = B$. Thus, *equation 1.7* can define the horizontal displacements:

$$u(z,t) = 2\, A\, \frac{e^{jkz} + e^{-jkz}}{2} e^{j\omega t} = 2\, A\, \cos(kz)\, e^{j\omega t} \qquad \text{Eq. 1.7}$$

This is a steady-state wave of amplitude $2\,A \cos(kz)$. Such formula can be used to calculate the transfer function $H_r(\omega)$ in terms of displacements for the present case between the surface of the upper stratum $(z = 0)$ and the basement $(z = H)$, as per *equation 1.8*:

$$H_r(\omega) = \frac{u_{max}(0,t)}{u_{max}(H,t)} = \frac{2\ A\ e^{j\omega t}}{2\ A\ \cos(kH)\ \ e^{j\omega t}} = \frac{1}{\cos(kH)} = \frac{1}{\cos F} \qquad \text{Eq. 1.8}$$

Where the parameter $F = k\ H = \omega\ H\ /\ V_s$ is defined as the frequency factor. The amplification function will have the expression of *Equation 1.9*:

$$A_r(\omega) = |H_r(\omega)| = \sqrt{\{R[H_r(\omega)]\}^2 + \{I[H_r(\omega)]\}^2} = \frac{1}{|\cos F|} \qquad \text{Eq. 1.9}$$

Where $R\ [H_r\ (\omega)]$ and $I\ [H_r\ (\omega)]$ are the real part and the imaginary part (equal to *0* in this case) of the equation, respectively. *Figure 8* shows the amplification function for the present case:

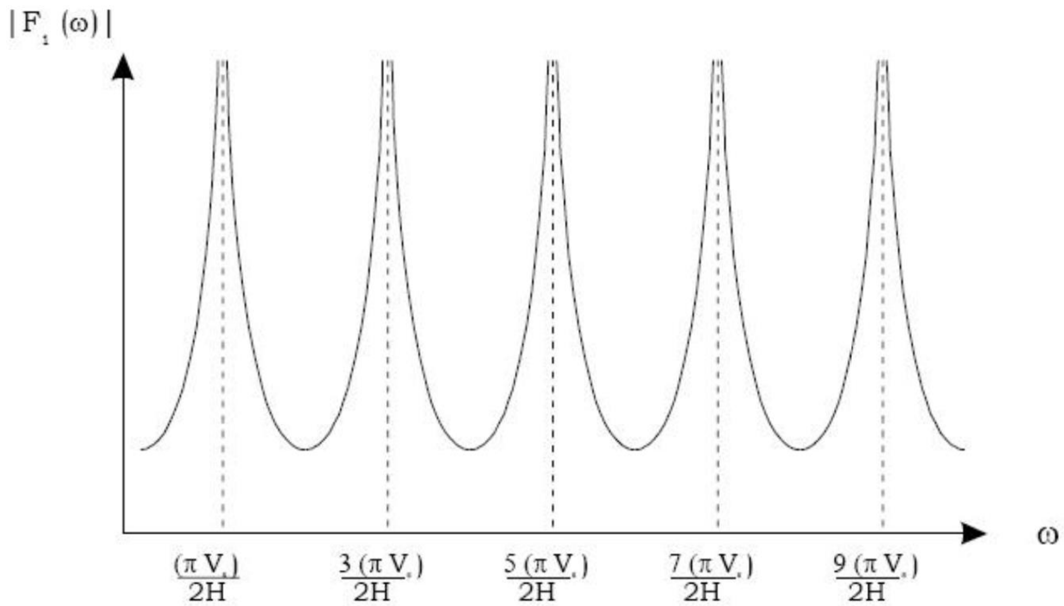


Figure 8 – Amplification function for a homogeneous elastic stratum above an infinitely rigid bedrock ([15])

From *equation 1.9*, it is easy to notice that the function is always greater or equal to *1*, thus essentially amplification function in this case amplifies nearly all harmonic components. Moreover, it attains infinite values periodically at some frequencies called, because of that, natural frequencies of the deposit. Very often, the most influent component for the determination of the deposit motion is the first natural frequency that gets the name of fundamental frequency. The formulas of the natural angular frequencies, frequencies and periods are expressed below as a function of *n* (*Equation 1.10*):

$$\omega_n = \frac{V_s}{H} F_n = \frac{V_s}{H}(2n-1)\frac{\pi}{2}$$
$$f_n = \frac{\omega_n}{2\pi} = \frac{V_s}{4H}(2n-1) \qquad n = 1, 2, \ldots, \infty \qquad \text{Eq. 1.10}$$
$$T_n = \frac{1}{f_n} = \frac{4H}{V_s(2n-1)}$$

To each of the natural angular frequencies, frequencies or periods is associated a modal shape that can be expressed as per *equation 1.11*:

$$U_n(z) = \cos(kz) = \cos\left[(2n-1)\frac{\pi}{2}\frac{z}{H}\right] \qquad n = 1, 2, \ldots, \infty \qquad \text{Eq. 1.11}$$

Analysing the amplification function for the current case, it is possible to appreciate the dangerous potential effects could arise whether the acting solicitation frequencies matches one or more deposit natural frequencies.

When resonance occurs, in such ideal case, signal amplitudes can attain infinite values at the surface. Here amplification function depends only upon the thickness (*H*) of the deposit and the mechanic features (V_s) of the soil. The reason why the amplitude function has several infinite values stays in the fact that the infinitely rigid bedrock bounces back in the upper layer all the waves travelling downwards then seismic energy remains solely confined in there.

Moving on, in the case where the upper stratum is linear elastic but the bedrock is deformable, the waves travelling downwards from the upper layer transmits a part of their energy to the bedrock. *Figure 9* displays the schematic for such case:

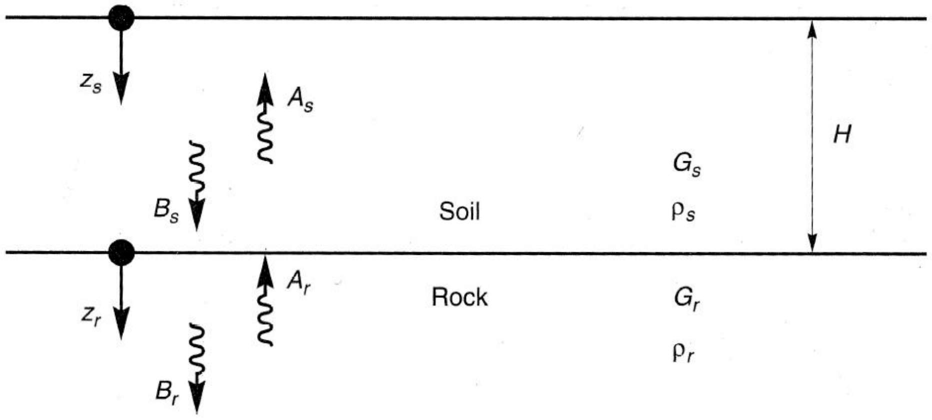


Figure 9 – Schematic of homogeneous elastic stratum above a deformable bedrock ([15])

The propagation phenomenon hinges on the parameter I, called impedance contrast, defined as per *equation 1.12*:

$$I = \frac{1}{\mu} = \frac{\rho_r V_r}{\rho_s V_s} \qquad \text{Eq. 1.12}$$

Where ρ_r is the density and V_r is the velocity of shear waves in the bedrock. In such a case the function transfer $H_{db}(\omega)$, calculated as the ratio between displacement amplitude in the surface and displacement amplitude at the top of the bedrock, is equal to the one calculated in the previous case (*Equation 1.8*).

Instead, if the transfer function refers to displacements in the surface and in the outcrop, $H_{da}(\omega)$, this assumes complex-form as shown in *equation 1.13*:

$$H_{da}(\omega) = \frac{1}{\cos\left(\frac{\omega H}{V_s}\right) + j\mu \sin\left(\frac{\omega H}{V_s}\right)} = \frac{1}{\cos F + j\mu\ \sin F} \qquad \text{Eq. 1.13}$$

Amplification function, plotted in *figure 10*, depends directly upon the Impedance as per *equation 1.14*:

$$A_d(\omega) = \frac{1}{|\cos F + j\mu \sin F|} = \frac{1}{\sqrt{\cos^2 F + \mu^2\ \sin^2 F}} = \frac{1}{\sqrt{\cos^2 F + \frac{1}{I^2}\ \sin^2 F}} \qquad \text{Eq. 1.14}$$

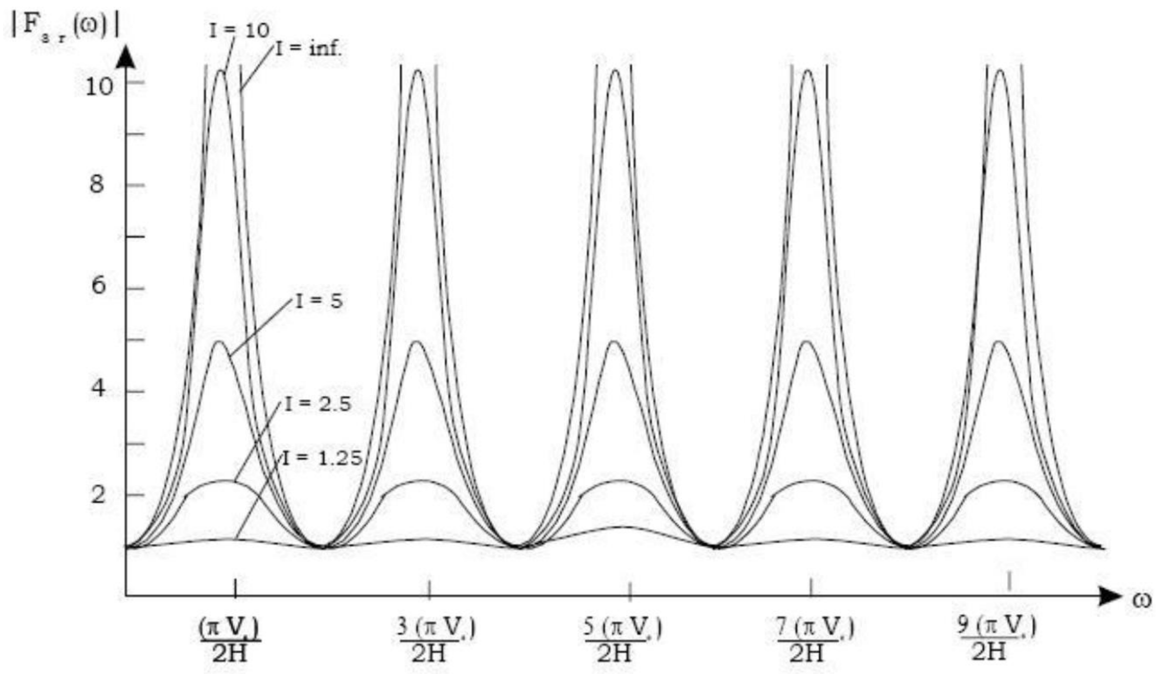


Figure 10 – Amplification function for a homogeneous elastic stratum above a deformable bedrock ([15])

Here it is possible to realize that the function remains periodic with the same natural frequencies but, unlike the previous case, peaks show in all circumstances finite values that decrease as the impedance factor reduces, following *equation 1.15*:

$$(A_d)_{max} = I = \frac{1}{\mu} \qquad \text{Eq. 1.15}$$

This represents the effect of dissipation of energy that occurs when seismic waves reach the surface and then propagate back to deformable bedrock. The more bedrock is deformable, the more seismic energy is dissipated to the earth's crust and the effects in the upper layer are mitigated.

A more realistic model to describe the behaviour of a soil is considering damping capability through a visco-elastic model (Kelvin-Voigt medium). In this case, the differential equation of dynamic equilibrium is *equation 1.16*:

$$\rho \frac{\partial^2 u}{\partial t^2} = G \frac{\partial^2 u}{\partial z^2} + \eta \frac{\partial^3 u}{\partial t \partial z^2} \qquad \text{Eq. 1.16}$$

Where the coefficient η indicates viscosity. Dumping factor for a visco-elastic medium, following a full cycle of harmonic solicitation, is expressed as per *equation 1.17*:

$$D = \frac{\eta\omega}{2G} \qquad \text{Eq. 1.17}$$

As before, looking for a solution by separation of variable through the shape function, the dynamic equilibrium equation can be right as per *equation 1.18*:

$$(G + j\omega\eta)\frac{\partial^2 p}{\partial z^2} + \rho\omega^2 p = 0 \qquad \text{Eq. 1.18}$$

At this stage, it could be convenient to introduce the complex factors, G^* shear complex modulus, k^* complex wavenumber and V^*_s complex shear velocity where (*Equation 1.19*):

$$G^* = G + j\omega\eta = G\,(1 + 2jD)$$
$$k^* = \frac{\omega}{V_s^*} = \frac{\omega}{\sqrt{\frac{G^*}{\rho}}} \qquad \text{Eq. 1.19}$$

Using such new parameters, formulas of the shape function and the horizontal displacements become respectively as per *equation 1.20* and *equation 1.21*:

$$p(z) = Ae^{jk^*z} + Be^{-jk^*z} \qquad \text{Eq. 1.20}$$
$$u(z,t) = Ae^{j(k^*z + \omega t)} + Be^{-j(k^*z - \omega t)} \qquad \text{Eq. 1.21}$$

Following the same mathematic steps for the linear case, it is possible to calculate the complex transfer function $H^*_r(\omega)$ in terms of displacements between the surface of the upper stratum $(z = 0)$ and the basement $(z = H)$. The final *equation 1.22* is shown below:

$$H_r^*(\omega) = \frac{1}{\cos(k^*H)} = \frac{1}{\cos F^*}$$ Eq. 1.22

Where $F^* = k^* H$. The form of the complex velocity of the shear waves can be expressed, with the hypothesis of low values of damping *D*, as per *equation 1.23*:

$$V_s^* = \sqrt{\frac{G^*}{\rho}} = \sqrt{\frac{G\ (1 + 2jD)}{\rho}} \approx \sqrt{\frac{G}{\rho}}(1 + jD) = V_s(1 + jD)$$ Eq. 1.23

Equation 1.24 expresses the complex wavenumber, as a result:

$$k^* = \frac{\omega}{V_s^*} \approx \frac{\omega}{V_s(1 + jD)} \approx \frac{\omega}{V_s}(1 - jD) = k(1 - jD)$$ Eq. 1.24

Hence, *equation 1.25* and *equation 1.26* indicate respectively the transfer function and the amplification function for the present case:

$$H_r^*(\omega) \approx \frac{1}{\cos[k\ (1 - jD)\ H]} = \frac{1}{\cos[kH - jDkH]}$$ Eq. 1.25

$$A_r^*(\omega) = \frac{1}{\sqrt{[\cos(kH)]^2 + [senh(DkH)]^2}} \approx \frac{1}{\sqrt{[\cos(kH)]^2 + [DkH]^2}} = \frac{1}{\sqrt{[\cos F]^2 + [DF]^2}}$$ Eq. 1.26

The amplification function is plotted in the *figure 11* as a function of different values for dumping factor.

In the event where *D = 0*, the function gets the same values than in the elastic case. As soon as *D* appears, the function loses the periodic shape, natural frequencies do not change but only some of the first natural frequencies emphasize because their peaks reduce with increasing the frequency. *Equation 1.27* describe the formula to calculate the peak values attained by the amplification function, depending on *n*:

$$(A_r^*)_{max,n} \approx \frac{2}{(2n - 1)\ \pi\ D} \qquad n = 1, 2, \ldots, \infty$$ Eq. 1.27

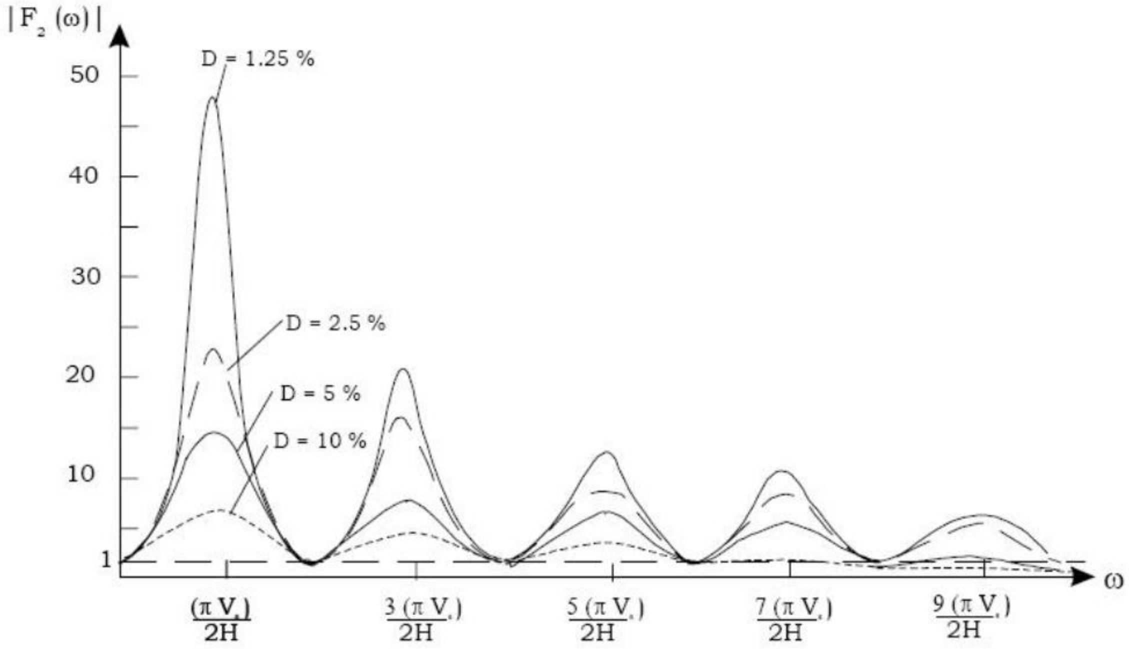


Figure 11 – Amplification function for a homogeneous visco-elastic stratum above an infinitely rigid bedrock ([15])

Ultimately, when a soil is supposed to have dissipating behaviour, the larger values attained by the amplification function depend only upon damping factor, as per an inversely proportional relationship.

The final case, the most realistic and generalized one among the closed-form solutions, consists of a visco-elastic deposit laying above a deformable bedrock. The complex transfer function $H^*_d\ (\omega)$ in terms of horizontal displacements between the surface of the upper stratum and the outcrop is showed below as per *equation 1.28*:

$$H_d^*(\omega) = \frac{1}{\cos\left(\frac{\omega H}{V_s^*}\right) + j\mu^* \sin\left(\frac{\omega H}{V_s^*}\right)} = \frac{1}{\cos F^* + \frac{j}{I^*}\sin F^*} \qquad \text{Eq. 1.28}$$

Where the complex Impedance ratio in *equation 1.29* is independent from the ω:

$$I^* = \frac{1}{\mu^*} = \frac{\rho_r V_r^*}{\rho_s V_s^*} \qquad \text{Eq. 1.29}$$

The peak values attained by the amplification function can be calculated by mean of *equation 1.30*. In this case, peak values depend upon damping factor and impedance ratio and the effect of both parameters is similar, namely, peaks decrease as increasing one of such variables. Again, for this case as well, natural angular frequencies, frequencies and periods turn out from *equation 1.10*.

$$(A_d^*)_{max,n} \approx \frac{1}{\mu + (2n-1)\,\frac{\pi}{2}\,D} \qquad n = 1, 2, \dots, \infty \qquad \text{Eq. 1.30}$$

Having simple pre-boiled solutions as the ones described above to study a so complicated problem is very relevant because the latter allow the designer to have a first picture of the matter, maybe not so accurate but usefully explanatory in a preliminary phase.

However, it is necessary to make some step forward to get a deep knowledge of the characteristic of the site in order to figure out how the phenomenon develops and which kind of implications it might have on the project in progress.

Indeed, having the transfer function is not enough to determine the characteristics of the motion at the surface. Two basic components must be evaluated to achieve an exhaustive site-soil amplification analysis: the transfer function for a realistic soil deposit and the design earthquake for the site of interest. By combining the latter, it is possible to obtain a quake time history on the surface to be used for calculations.

The purpose of the study is to manage carefully the resonance issue between the design earthquake and the soil deposit or about the whole system bedrock-deposit-structure. Once again, such calculation can be carried out in both dual domains, time and frequency. In the time domain, the necessary mathematical operation is the convolution integral, whereas in the frequency domain a simpler multiplication delivers the outcome.

From the engineeristic perspective, having an overview of the Fourier spectra of the signals and of the deposit transfer function could reveal extremely convenient to better understand the matter. A schematic of the process for a site-specific response analysis is provided in *figure 12*.

To assess quantitatively the motion on the surface, the evaluation of the expected design earthquake is fundamental. This affects as well the modelling of the transfer function of a real subsurface.

The design earthquake comes from regional seismic hazard studies that could be deterministic, probabilistic or more frequently semi-probabilistic. About the transfer function, instead, it is relevant to take into account the inherent dishomogeneities of the soil such as the variation of the mechanics parameters (stiffness and damping) among each strata, keeping in mind that even in a homogeneous lithologic stratum these parameters actually change in accordance with the stress state, then with the depth of the soil element.

Moreover, the hypothesis of linear elastic or visco-elastic behaviour for the ground is not very realistic since soil shows non-linear and dissipative behaviour as early as it undergoes to small strains. Furthermore, geometry in a real-case scenario could be extremely irregular so that a horizontal schematic could turn out to be inadequate.

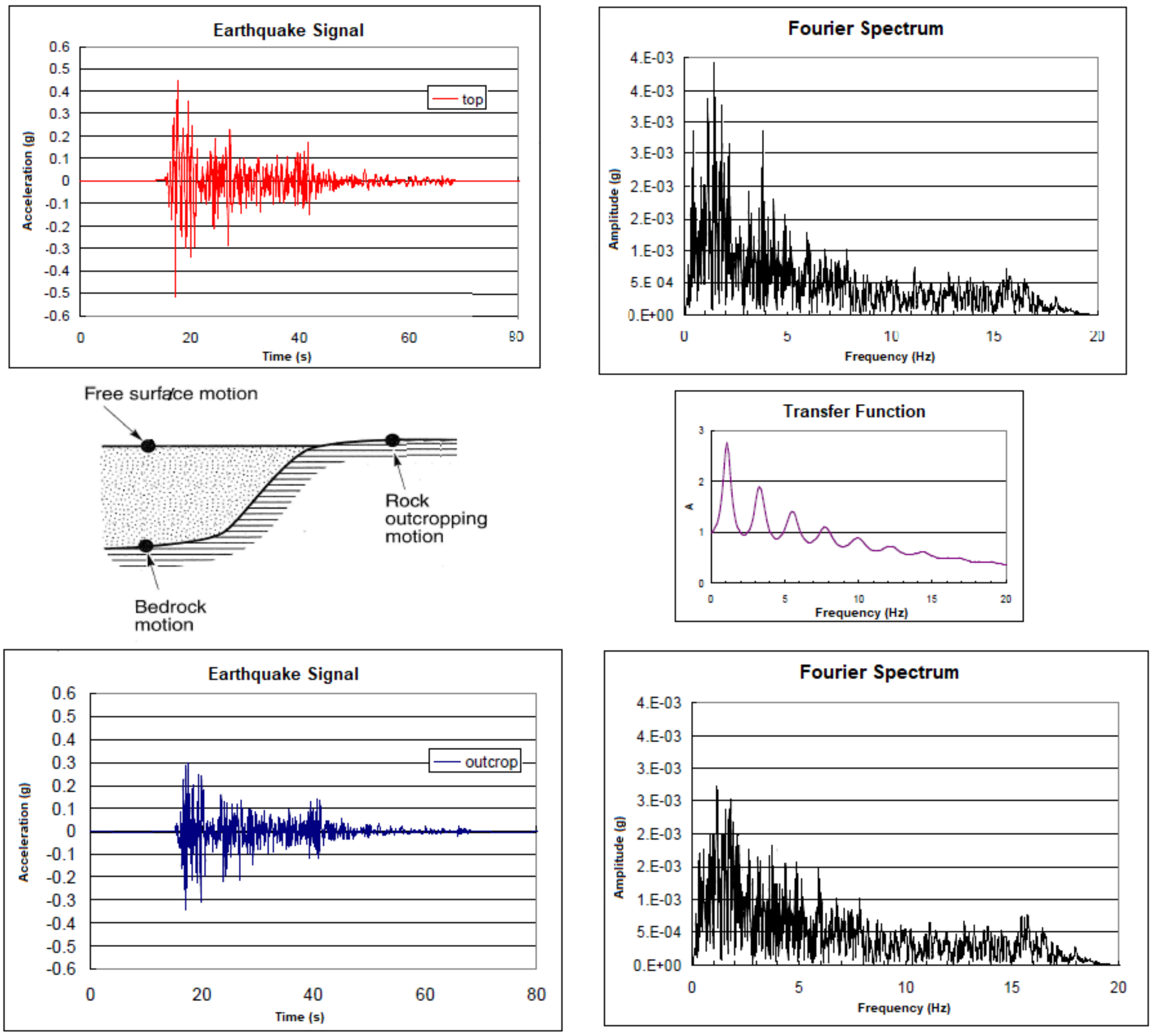


Figure 12 – Schematic for the soil amplification analysis of a homogeneous deposit

Considering again the subsurface like a continuum, it is possible to express changes with depth through a variation law. Different formula has been proposed over the time, the most adherent one to a real soil behaviour is displayed below *equation 1.31* both for shear module *G(z)* and shear waves velocity $V_s(z)$, for a soil with constant density ρ:

$$G(z) = G_0\left(1 + \alpha\frac{z}{H}\right)^{2m}$$
$$V(z) = V_0\left(1 + \alpha\frac{z}{H}\right)^{m} \qquad \text{Eq. 1.31}$$

Where G_0 and V_0 are the elastic parameters on the surface and α and m introduce the heterogeneity degree [18]. This solution is as suitable as the variation of the soil characteristics are regular. In the case of linear variation with depth for the velocity ($m = 1$), as an example, α expresses the heterogeneity ratio *equation 1.32*:

$$\frac{V_H}{V_0} = 1 + \alpha \qquad \text{Eq. 1.32}$$

The effect of such kind of heterogeneity gives rise to some change in the shape functions of the deposit: increasing α, the amplitude of horizontal displacements reduce with depth whereas the higher values concentrate in the superficial part.
The natural frequencies also change: the fundamental one slightly grows up and conversely all the others shift towards lower values and they approach to one another. This trend is more marked the more α increases. As a result, not considering heterogeneity of the deposit implies a mismatch of the frequencies and an underestimation of the amplitudes of the seismic motion at the surface.
In the event of higher irregularities of the soil behaviour, the continuum modelling is not anymore suitable and the deposit need to be considered as a sequence of layers where each layer has its own physic and mechanical parameters.
Hence, the study cannot be conducted using analytical functions as described so far, but the solution can only result by means of numerical methods [15] [17]. Usually, in such a case, there are two ways to represent the soil: continuum and discrete. *Figure 13* portrays the schematic of the two methods in the context of a 1-D propagation problem through a horizontally layered deposit. In both models, the site analysis response is performed solving the equations of motion

starting from a known acceleration time history *a(t)* beneath the deposit, coming from the bedrock.

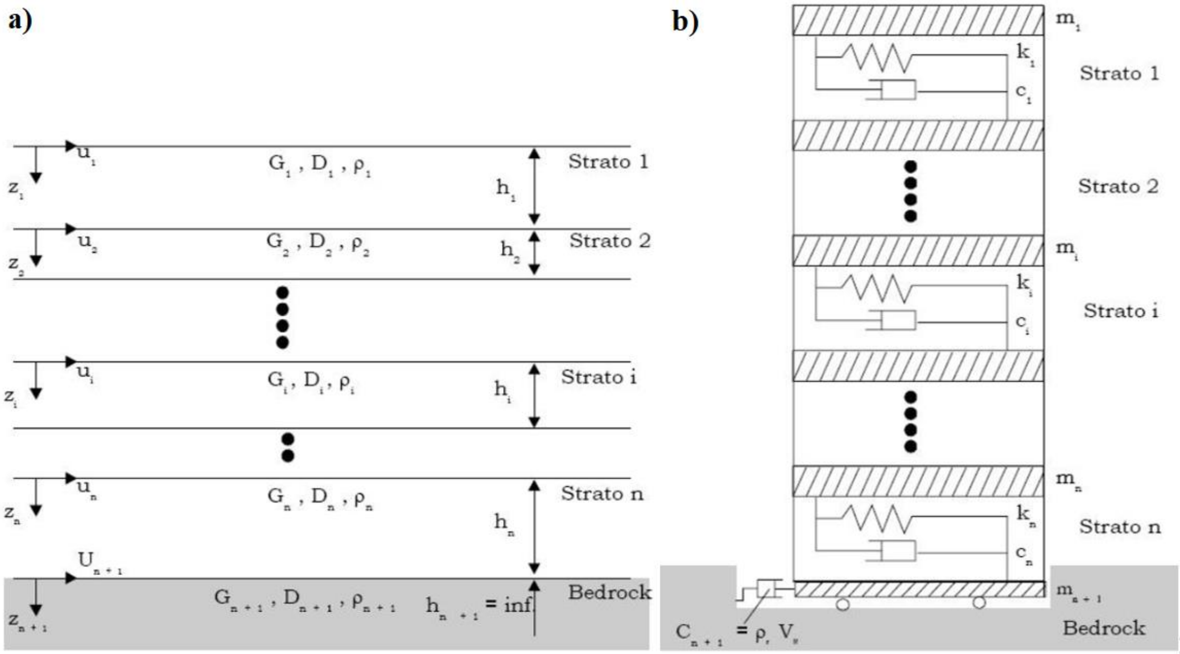


Figure 13 – Horizontally layered deposit for 1-D propagation problem
a) continuum model, b) discrete model ([17])

Within continuum model, the deposit is assumed as a continuum-layered soil where each layer is considered to be homogeneous and to have a linear visco-elastic behaviour. The specific parameters featuring each stratum are thickness h_i, density ρ_i, shear modulus G_i and damping factor D_i which is linked to the viscosity coefficient η_i by the following *equation 1.33*:

$$D_i = \frac{\eta_i \omega}{2G_i} \qquad \text{Eq. 1.33}$$

The differential equation of dynamic equilibrium ruling the phenomenon of propagation in each stratum is *equation 1.34*:

$$\rho_i \frac{\partial^2 u_i}{\partial t^2} - G_i \frac{\partial^2 u_i}{\partial z^2} - \eta_i \frac{\partial^3 u_i}{\partial t \partial z^2} = 0 \qquad \text{Eq. 1.34}$$

Where z is the coordinate for each layer. As for the previous cases, considering harmonic external force with frequency ω, displacements can be expressed through the following *equation 1.35* by means of the shape functions and the complex parameters already introduced in *equation 1.18* and *equation 1.19*:

$$u_i(z,t) = p_i(z)e^{j\omega t} = \left(A_i e^{jk_i{}^*z} + B_i e^{-jk_i{}^*z}\right)e^{j\omega t} = A_i e^{j(k_i{}^*z+\omega t)} + B_i e^{-j(k_i{}^*z-\omega t)} \qquad \text{Eq. 1.35}$$

Where A_i and B_i are respectively the amplitudes of the upwards and downwards propagating waves. From the latter, it is possible to find displacements [$u_i(0,t)$, $u_i(h_i,t)$] and tangential forces [$\tau_i(0,t)$, $\tau_i(h_i,t)$] of the interfaces, then imposing continuity conditions among strata over the time, the result is *equation 1.36*:

$$\begin{aligned} &A_i e^{jk_i{}^*h_i} + B_i e^{-jk_i{}^*h_i} = A_{i+1} + B_{i+1} \\ &k_i^* G_i^*\left(A_i e^{jk_i{}^*h_i} - B_i e^{-jk_i{}^*h_i}\right) = k_{i+1}^* G_{i+1}^*\left(A_{i+1} - B_{i+1}\right) \end{aligned} \qquad \text{Eq. 1.36}$$

Introducing the complex impedance ratio μ_i (independent from ω) in *equation 1.37*:

$$\mu_i = \frac{k_i^* G_i^*}{k_{i+1}^* G_{i+1}^*} \equiv \sqrt{\frac{\rho_i G_i^*}{\rho_{i+1} G_{i+1}^*}} \qquad \text{Eq. 1.37}$$

It is possible to find the recurrence equations (*Equation1.38*) of the amplitudes A_{i+1} and B_{i+1} relative to A_i and B_i, for the generic layer:

$$\begin{aligned} A_{i+1} &= \frac{1}{2}\left[A_i(1+\mu_i)e^{jk_i^*h_i} + B_i(1-\mu_i)e^{-jk_i^*h_i}\right] \\ B_{i+1} &= \frac{1}{2}\left[A_i(1-\mu_i)e^{jk_i^*h_i} + B_i(1+\mu_i)e^{-jk_i^*h_i}\right] \end{aligned} \qquad \text{Eq. 1.38}$$

Imposing the condition of free surface $\tau_1(0)=0$, incident wave is the same of reflected one ($A_1 = B_1$). By means of the recurrence formula in *equation 1.38*, iterating for each layer ($i = 1 \ldots n$), the transfer function $a_i(\omega)$ and $b_i(\omega)$ of the upwards and downwards component in relation to the surface can be defined as per *equation 1.39*:

$$A_i = a_i(\omega)A_1 \qquad \text{Eq. 1.39}$$
$$B_i = b_i(\omega)B_1 \equiv b_i(\omega)A_1$$

Ultimately, the transfer function $H_{ik}\ (\omega)$ between any two layers i and k is the following *equation 1.40*:

$$H_{ik}(\omega) = \frac{A_k + B_k}{A_i + B_i} = \frac{a_k(\omega) + b_k(\omega)}{a_i(\omega) + b_i(\omega)} \qquad \text{Eq. 1.40}$$

Where $a_k\ (\omega)$ and $b_k\ (\omega)$ are harmonic functions, therefore, the formula can be easily adapted to produce as well transfer functions for velocity and acceleration between two strata. When $k = 1$ and $i = n$, this expression represents the amplification function of the whole layered deposit. All in all, to perform a site-specific response analysis with the model of the continuum layered soil, the process to follow is described in the foregoing *figure 12*:

- Determining the Fourier Spectrum of the acting acceleration time history $a(t)$, for instance, by using the Fast Fourier Transform (FFT) tool, obtaining $a(\omega)$;
- Calculating the deposit transfer function $H_{ik}\ (\omega)$;
- Multiplying $H_{ik}\ (\omega)$ and $a(\omega)$ to obtain the spectrum of the seismic motion at the surface $a_s(\omega)$;
- Getting back to the time domain, for instance, by using the Inverse Fast Fourier Transform (IFFT) for $a_s(\omega)$ to have finally the acceleration time history at the top of the surface $a_s(t)$.

In the discrete model, instead, the deposit is again layered but stratigraphy is considered as a set of concentrated masses placed in the boundaries between two different layers and connected one to another by springs and viscous dampers in order to form an n degrees of freedom system. In such a case, parameters are thickness h_i, masses m_i, spring stiffness k_i, and viscous damping coefficients c_i.

Masses, stiffness, and dumpers for the discrete system come from density ρ_i, shear modulus G_i, viscosity coefficient η_i of the corresponding volume element, as per *equation 1.41*:

$$m_1 = \frac{\rho_1 h_1}{2}; \quad m_i = \frac{\rho_i h_i + \rho_{i-1} h_{i-1}}{2} \ (1 < i \leq n); \quad m_{n+1} = \frac{\rho_n h_n}{2}$$
$$k_i = \frac{G_i}{h_i} (i \leq n) \qquad \text{Eq. 1.41}$$
$$c_i = \frac{\eta_i}{h_i} (i \leq n); \quad c_{n+1} = \rho_r V_r$$

Equation 1.42 expresses the dynamic equilibrium of the system:

$$\begin{aligned} & m_1 \ddot{u}_1 - c_1(\dot{u}_2 - \dot{u}_1) - k_1(u_2 - u_1) = 0 \\ & m_i \ddot{u}_i + c_{i-1}(\dot{u}_i - \dot{u}_{i-1}) - c_i(\dot{u}_{i+1} - \dot{u}_i) + \dots \\ & \dots + k_{i-1}(u_i - u_{i-1}) - k_i(u_{i+1} - u_i) = 0 \ (1 < i \leq n) \end{aligned} \qquad \text{Eq. 1.42}$$

Where u_i is the horizontal absolute displacement of the mass *i*. Assuming the bedrock as deformable, it is necessary to add one more equation to the *n*-equation system to impose the equilibrium between the mass *n+1* and the consistency with the motion on the top of the bedrock (*Equation 1.43*):

$$m_{n+1}\ddot{u}_{n+1} + c_n(\dot{u}_{n+1} - \dot{u}_n) + c_{n+1}\dot{u}_{n+1} + k_n(u_{n+1} - u_n) = c_{n+1}\dot{u}_r \qquad \text{Eq. 1.43}$$

Ultimately, *equation 1.44* expresses the system of linear differential equations in u_i:

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = \{J\}f(t) \qquad \text{Eq. 1.44}$$

Where masses matrix is diagonal whereas damping and stiffness matrixes are diagonal band as shown in *equation 1.45*:

$$[M] = \begin{bmatrix} m_1 & & & \\ & m_2 & & \\ & & \dots & \\ & & & m_{n+1} \end{bmatrix}$$
$$[C] = \begin{bmatrix} c_1 & -c_1 & & \\ -c_1 & c_1 + c_2 & -c_2 & \\ & & \dots & \\ & & -c_n & c_n + c_{n+1} \end{bmatrix} \qquad \text{Eq. 1.45}$$
$$[K] = \begin{bmatrix} k_1 & -k_1 & & \\ -k_1 & k_1 + k_2 & -k_2 & \\ & & \dots & \\ & & -k_n & k_n + k_n \end{bmatrix}$$

Term *{J}* is a vector indicating the external acting forces upon each mass, then it has all zero values except for the last one corresponding to the bedrock *equation 1.46*:

$$f(t) = \rho_r V_r \dot{u}_r(t) \quad \text{Eq. 1.46}$$

In the case where the bedrock is deformable so that the loss of energy is taken into account by the coefficient c_{n+1}, representing the bedrock impedance. On the other hand, in the case where the bedrock is infinitely rigid, u_{n+1} and the equation *(n+1)* are erased from the system and the last equation *n* becomes *equation 1.47*:

$$m_n\ddot{u}_n + c_{n-1}(\dot{u}_n - \dot{u}_{n-1}) + c_n\dot{u}_n + k_{n-1}(u_n - u_{n-1}) + k_n u_n = c_n\dot{u}_r + k_n u_r \quad \text{Eq. 1.47}$$

That is equal to use the following relation *equation 1.48* in the general system:

$$f(t) = c_n\dot{u}_r + k_n u_r \quad \text{Eq. 1.48}$$

To resolve this equations system, typical methods of structural engineering can be used, such as direct integration of the equation or modal analysis as well as resolution methods in the frequency domain.

Considering now the constitutive law, the best options to represent the real behaviour of the soil within one of the foregoing analysis are essentially two. The rigorous solution is to implement the real stress-deformation curve for the soil of interest, but this needs strictly a step-by-step procedure for the integration of the equations of motion. As an alternative, it is possible to implement an equivalent linear analysis where the final parameters outcome as a result of an iterative resolution process that fits very well with the methods described above.

In such a case, the analysis are performed using trial parameters for shear modulus and damping ratio, starting from their values referring to very small strains, G_0 and D_0. Once the analysis turn out the results, such parameters are verified on the basis of the shear strain attained. Following the stress-deformation relation of the soil, the trial parameters are updated until the shear strain attained matches reasonably with them. *Figure 14* portrays the sequence of this procedure.

It goes without saying that the transfer function, in the event of non-liner behaviour, depends directly upon the level of strain induced by the striking earthquake. Particularly, strong events cause a marked decay of the shear modulus and inversely a raise of the damping ratio, then the corresponding transfer function will have lower values than in case of medium or minor events. As displayed in *figure 15*, the more strain level increases, the more non-linear behaviour gives rise to a reduction of the amplification peaks, inversely proportional to damping. Moreover, all the peaks move towards lower frequencies (higher periods) because of the shear modulus decrease (then an increase of the impedance factor).

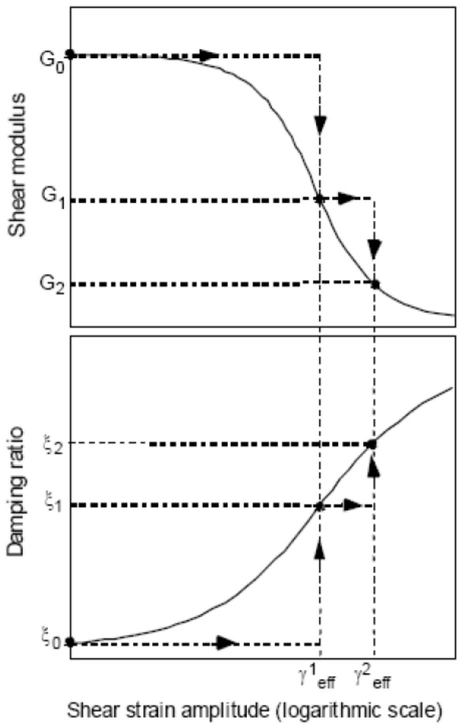


Figure 14 – Iterative procedure in the case of linear equivalent constitutive law

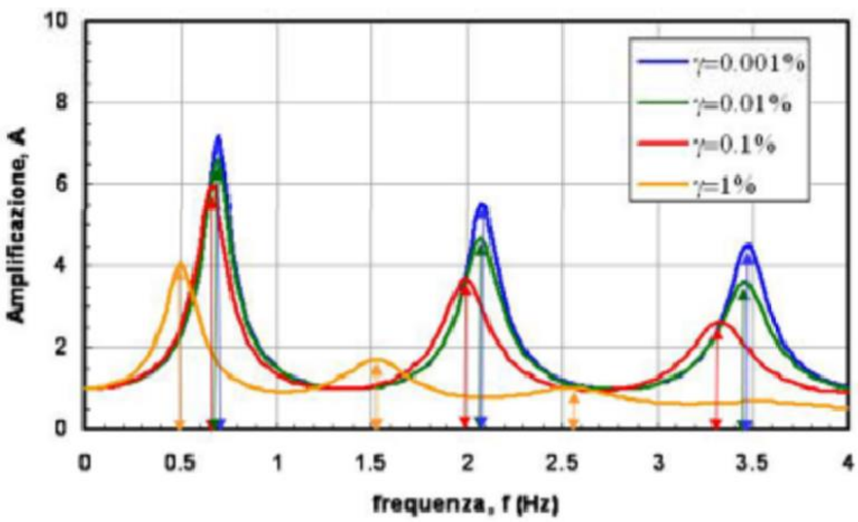


Figure 15 – Influence of soil non-linearity in the Amplification function

Topography of the area also plays an important role in defining the final seismic motion. Due to phenomena of focalization of seismic waves caused by reflection on the free surface and interactions among different types of waves, crests and scarps need special attention. International standards and technical codes provide for constants that take into account properly such effects.

Furthermore, numerous cases of amplification effects depending on the geometry have been recorded in alluvial deposit struck by earthquakes near the boundary of valleys. Here a combination of effects consisting of the amplification due to the impedance contrast and the 2-D constructive interference among waves due to the topographic arrangement might amplify intensely the signal as depicted in *figure 16* [19].

Such kind of analysis in complex geometric conditions (2-D or 3-D) can only be performed using advanced FEM, FDM or BEM methods.

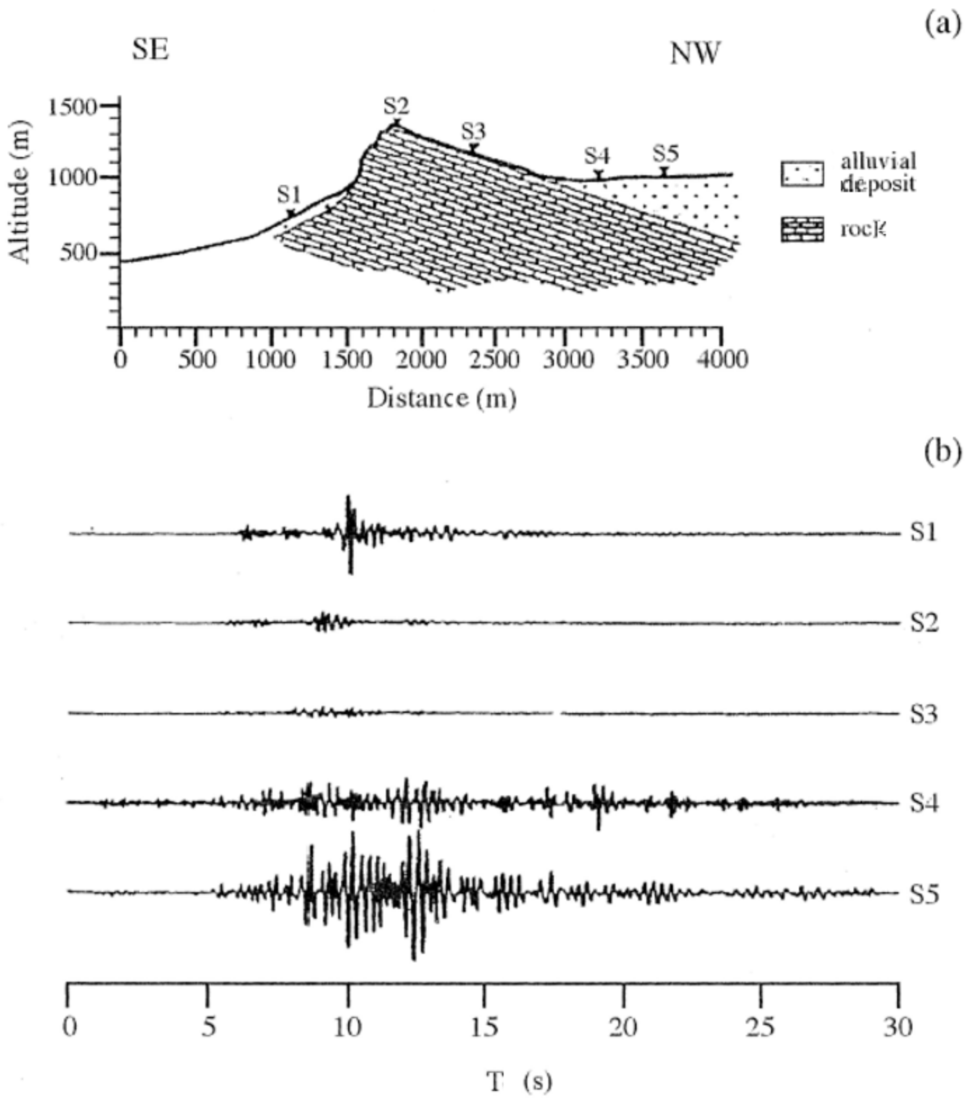


Figure 16 – Topographic amplification: a) schematic of the measure points, b) corresponding records ([19])

Chapter 2

NEW FRONTIERS FOR SEISMIC ISOLATION SYSTEMS BASED ON THE CONCEPT OF METAMATERIAL

2.1 METAMATERIALS: CONCEPTS AND PROPOSALS IN ANTI-SEISMIC ENVIRONMENT

Metamaterials are a cutting-edge technology having huge potential for future developments. Metamaterial is a general meaning term, used to indicate a particular assembly of composite materials that, regarding to their micro or macro arrangement, acquire unique properties that could not be found in nature. [20] [21]

The first extraordinary peculiarity that placed metamaterials at the forefront of technical innovation is the capability to get atypical index of refraction, even negative, allowing them to play a role in the wave propagation phenomena. The index of refraction N is defined as in *equation 2.1*:

$$N = \pm\sqrt{\varepsilon \cdot \mu} \qquad \text{Eq. 2.1}$$

where ε is the permittivity and μ is the magnetic permeability.

This feature does not exist in any natural material indeed the largest number of material has got both permittivity and magnetic permeability positive, hence they are transparent to the electromagnetic radiations. Metals can have negative permittivity but just in the range of visible light wavelength and this is the reason why almost all metals are opaque. Conversely, artificial Metamaterials might even have got both permittivity and magnetic permeability negative. This allows Metamaterials to manipulate electromagnetic waves by stopping, absorbing, amplifying or bending them, achieving benefits that could not be possible with conventional materials.

The remarkable advantage of this technology comes from the point that such a specific feature does not derive inherently from the properties of the composing materials, rather than from their arrangement as a whole-Metamaterial. In fact, to interact with some specific phenomenon, metamaterials practically need only to be sized at the same scale or smaller than the wavelengths of the pertinent waves [22] [23] [24]. Metamaterial setup can comprise geometry, shape and orientation of components, interplay among elements, often embedded in repeating and periodic patterns, that is a very typical characteristic of Metamaterials.

Metamaterials can be specifically designed and engineered in accordance with the goals that they are intended to fulfil for. This major purpose opened the way to their dissemination in a number of areas. Indeed, Metamaterials have become a multidisciplinary argument that involves different fields like electrical engineering, electromagnetics and optics, acoustics, physics, microwave and antenna engineering, material sciences, nanosciences, semiconductor engineering and earthquake engineering.

Metamaterials has very huge application fields: creation of optical filters, development of superlenses and cloak of invisibility in the optics field, medical devices as well as a wide variety of military equipment, like high-frequency battlefield communication or remote aerospace applications, any kind of sensor for monitoring and detection, just to give an example. [25] [26]

As regards to seismic engineering, Metamaterials can be exploited to filter the earthquake' waves in order to protect buildings or any other kind of construction, even up to entire portions of territory. Metamaterials have the capability to attenuate the effect of the earthquakes inside a frequency band region, hence called bandgap, where seismic propagating waves decrease drastically preventing constructions to incur in structural hardships.

Applying the concepts employed in classic optics, acoustic and electromagnetics to other fields where phenomena propagations are the central issue, the researchers reckoned convenient to apply to new scientific areas such a solid knowledge enhanced over the time. In fact, wave vector, wave impedance and direction of power flow are universal and, under some conditions, seismic waves show to be closely analogous to the acoustic ones.

Metamaterials can show negative bulk modulus, negative mass density, or both. When an earthquake occurs, seismic waves radiate away from the hypocentre to the surface generating body waves during the propagation through the inner layers of the earth crust that generate S waves and surface waves through the outer layers that generate Rayleigh waves. Constructions in general suffer both of these waves, especially the one that provide horizontal accelerations to the structures.

The use of Metamaterials to filter seismic waves has been demonstrated both for waves P and S and for surface waves Rayleigh and Love. Over the time, researchers tested metamaterials applications with theoretical demonstrations, numerical models and as well with full-scale experiments.

Several approaches have been attempted proposing seismic invisibility cloaks, vibrating barriers, sacrificial structures or antiseismic devices. Mainly, seismic metamaterials with sub-wavelength local resonators can be set up in order to exhibit negative mass density and/or negative Young's modulus as a way to tackle elastic wave's propagation within the pertinent seismic frequency range.

Nowadays, the challenge in the scientific community is to find the most effective configuration to give to this technique viability, reliability and serviceability necessary to make Metamaterial devices fully available within the framework of earthquake engineering.

As already anticipated, the concept of Metamaterials has been developed originally in the context of electromagnetics [20] [21], but over the time many branches have been evolving into many other fields thanks to their versatility. Above all, elastic Metamaterials have high potentialities of development. Wu et al [22], in this regard, have given an important contribution proposing an effective medium theory for elastic Metamaterials showing that the relevant parameters, like bulk modulus, shear modulus and mass density, could became negative near the resonance frequency if appropriate resonant scatters are chosen. Subsequently, they made [23] an elastic Metamaterial with fluid-solid composite inclusions that shows both negative shear modulus and negative mass density over a large frequency region.

Liu et al [24] give a milestone in demonstrating the capabilities of Metamaterials to filter the energy of propagating waves within a frequency region. The experiments focused on sonic crystals with a simple microstructure assembly made by lead spheres acting as the internal resonator with a coating of an elastic material like rubber or silicon.

Another important category of Metamaterials is the phononic crystal system that has been investigating in numerous applications, of which Page et al. [27] and Sukhovich et al. [28] papers represent outstanding examples. Nevertheless, the latter models of Metamaterials are not suitable for interacting with elastic waves whose wavelengths can vary from a few to hundreds of meters, as in the range of seismic engineering. Only a small portion of phononic crystal with a sub-wavelength arrangement of resonators, called locally resonant elastic metamaterials, well described by the works of Khelif et al. [29] and Achaoui et al. [30], can be applied in seismic environment.

Hussein and Frazier [31] proposed a new model for the design of Metamaterials and systems that show high damping rate maintaining high level of stiffness. This high damping property, that the authors called *Meta-damping*, comes from two features: the presence of locally resonant elements and the presence of at least one component that exhibits damping characteristics.
For this purpose, the authors carried out a very helpful comparison between two different configurations of Metamaterial, as shown in *figure 17*, namely acoustic Metamaterials (mass-in-mass system) and phononic crystal system.
One of the advantages of the acoustic Metamaterials in comparison with the phononic crystals is that they do not rely on structural periodicity to filter acoustic waves, so their size can be substantially smaller than in the case of phononic Metamaterials.

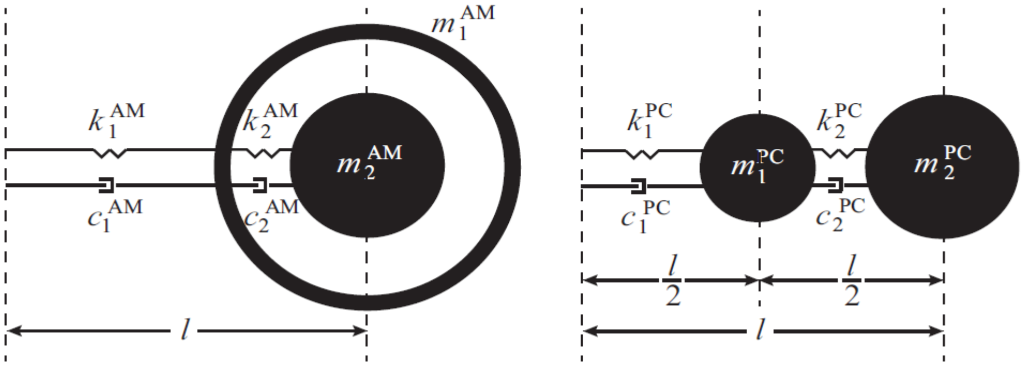


Figure 17 – Unit cells of statically-equivalent periodic chains of acoustic Metamaterial (on the left) and phononic crystal (on the right)([31])

More recently, many different applications as well as full-scale experiments have been realized to prove Metamaterials efficiency.
Brûlé et al [32], for instance, conducted a seismic experimental test in a Metamaterial consisting of an engineered natural soil. Ménard Engineering Company [33], in a deep basin of homogeneous silty clay, made the experiment and the Metamaterials was formed by a regular mesh of vertical voids, circularly shaped. The lattice was composed of *3* rows of voids, as shown in *figure 18*, while the source, a vibro-compaction probe, was located at *1,5 m* distance to the Metamaterial and provided a monochromatic force of *50 Hz*, with an initial displacement of *0,014 m*.

Sensors have been placed inside the Metamaterial, behind and in the surrounding, to detect the motion. The experiment demonstrated the attenuating effect of this special type of Metamaterial.

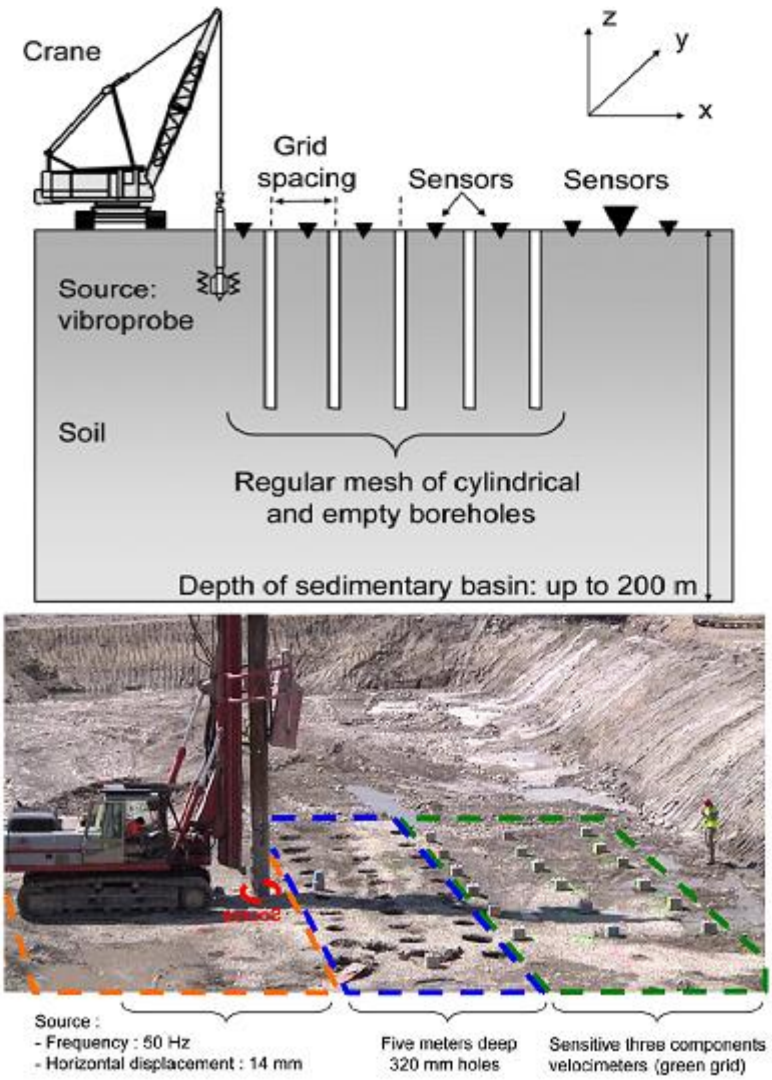


Figure 18 – Schematic of the Seismic Metamaterial experiment: cross section in the x-z plane (at the top) and photograph of the execution (at the bottom) ([32])

In Colombi et al [34] contribution, the concept of Metamaterial was utilised along with the ones of plasmonics, elasticity and geophysics, in order to design a seismic meta-wedge. Such type of graded vertical array of resonators, in the case in point trees placed atop an elastic substrate, can interact with Rayleigh waves in two ways: classical wedge, trapping destructive Rayleigh waves and providing an effect like seismic rainbow, or inverse wedge, generating conversion into harmless bulk shear waves. Therefore, the authors demonstrated how their Metamaterial could be utilised for vibration protection and elastic wave filtering.

In the Krödel et al [35] work, the authors advanced the proposal to use an array of resonating structures, called as *metastructure*, buried around sensitive building to protect them from earthquakes, aiming at the design of new civil infrastructures. The metastructure consist of arrays of cylindrical tubes containing a resonator suspended by soft bearings. The authors studied the response of these systems through numerical analysis and scaled experiments in a relevant range of frequency, from *1* to *10 Hz*.

Low frequency seismic waves ranging from *1* to *10 Hz* have been investigated also by Achaoui et al [36], using a periodic array of very small columns clamped in the bedrock and surrounding the building to be protected (*Figure 19*) in order to determine a zero-frequency stopband zone.

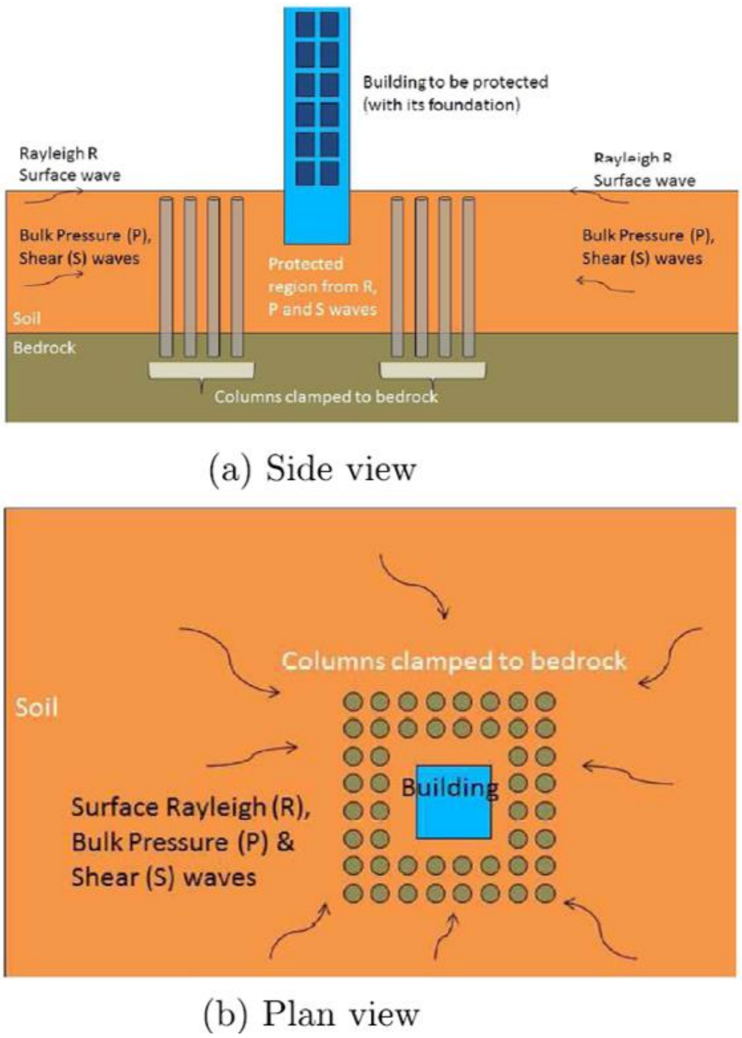


Figure 19 – Seismic Metamaterial consisting of columns clamped to the bedrock and surrounding the building to protect ([36])

In Maniaci et al [37] work, the authors performed an extensive and realistic examination about Metamaterials behavior in both phononic crystals and locally resonant configurations, investigating the influence of geometric and mechanical parameters and taking into account the characteristics of the ground where they are embedded, like soil layered structure and viscoelastic effects. The paper demonstrates the feasibility and the effectiveness of Metamaterials. The authors proposed different arrangement of Metamaterial (*Figure 20*) consisting of cavities (boreholes) in the ground and periodic inclusions in the soil, in a square array surrounding the structure to be protected.

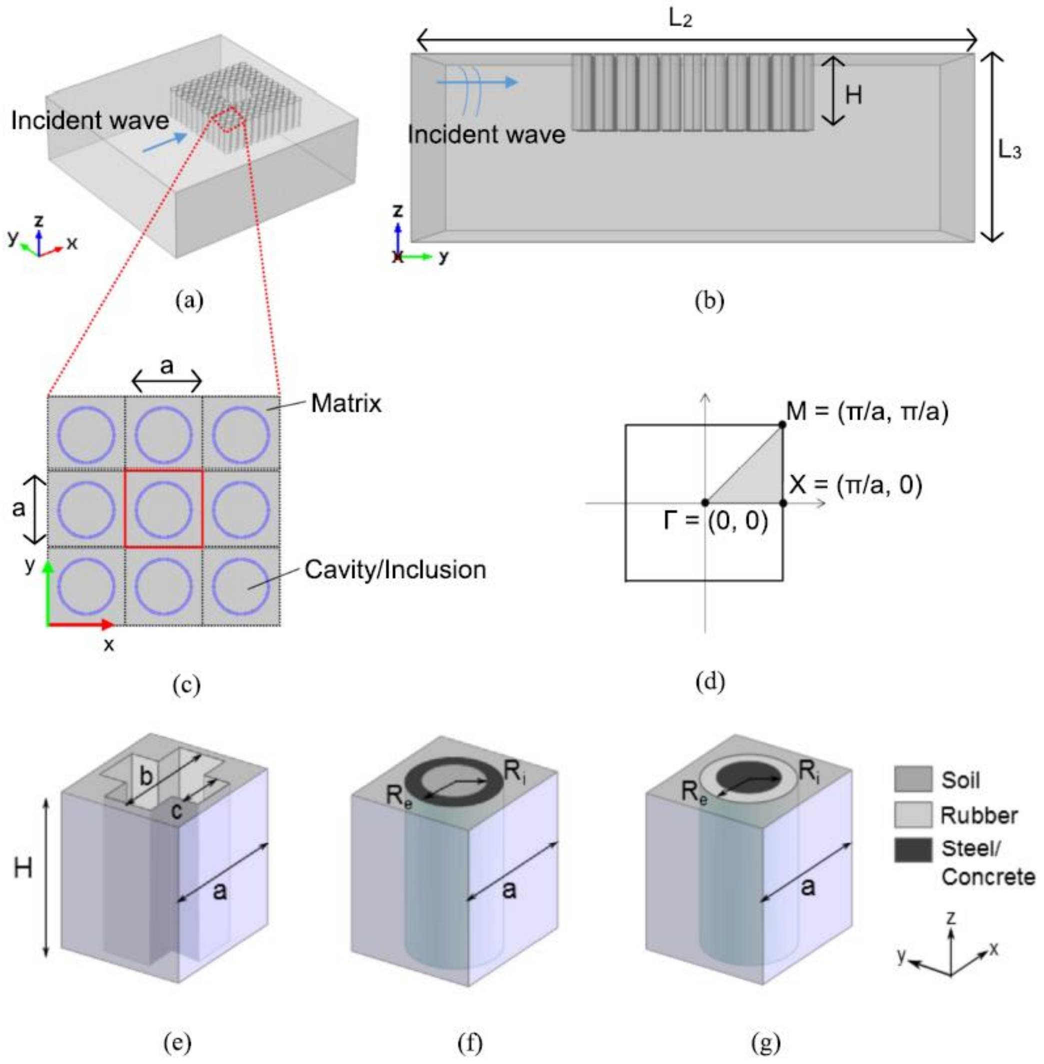


Figure 20 – Schematic representation of the proposed seismic shield based on the concept of Metamaterial: (a) 3-D section, (b) cross section in the y-z plan, (c) unit cell of periodic inclusions embedded in a matrix, (d) the First Brillouin zone, (e) cross-like cavity unit cell, (f) hollow cylinder unit cell, (g) coated cylinder unit cell ([37])

Finocchio et al [38] proposed a seismic Metamaterial consisting of a chain of mass-in-mass units able to filter the S waves of earthquakes. The paper covers the effect of the Metamaterial in the soil response analysis within the phenomenon of the vertical propagation of the seismic waves coming from the bedrock. Differently from the schemes proposed in the previous contributions, the authors implemented a continuous structure for the seismic Metamaterial with a pattern of 3-D matrix of cycloidal oscillators. Elementary units consisting of a sphere constrained by the gravity to roll over a cycloidal trajectory build up the isolator. The cycloidal pendulum is isochronous then the oscillation frequency is not dependent on the amplitude of the oscillation but only on the geometrical dimensions of the cycloid. *Figure 21* displays the attenuating effect of such type of isolator.

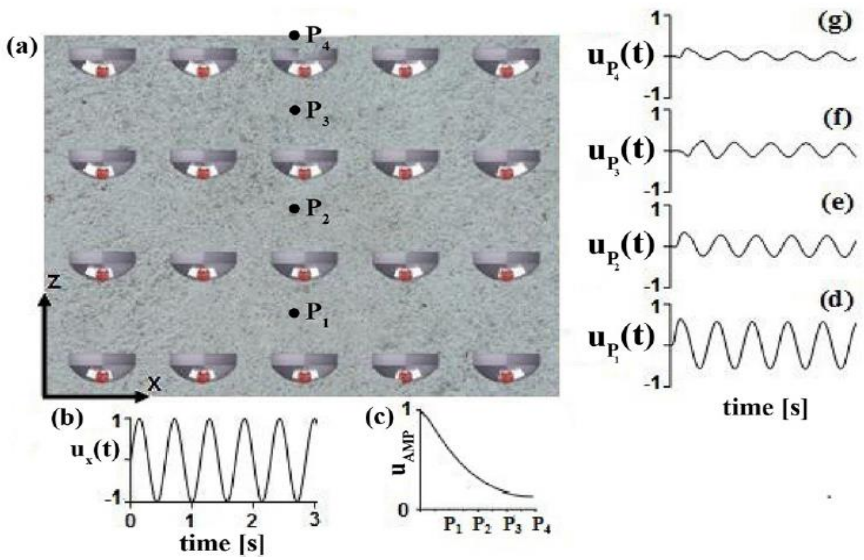


Figure 21 – Numerical implementation of the Seismic Metamaterial based on isochronous oscillators: (a) the model, (b) imposed displacements at the bottom surface, (c) amplitude of displacements as function of the thickness of the Seismic Metamaterial, (d)-(g) time domain traces of displacements computed at the control points P_1, P_2, P_3 and P_4 ([38])

Another proposal of periodic foundation comes from Shi and Huang [39] paper, where the authors experiment a new foundation consisting of a three-dimensional cell unit formed by a high-density core and a soft coating included periodically in a concrete matrix. Similarly, with the previous case, seismic waves having frequency matching with the foundation bandgap reduce significantly during their propagation to the top of the foundation.

Other proposals of Metamaterial come from Liu & Hu [40] paper. Exploiting the concept of chirality, the authors proved that coupled rotation and bulk deformation of 2-D chiral solids provide a new resonant mechanism that gives rise to bandgaps at relatively low frequency. They proposed such new technique as a potential new frontier for the design of elastic Metamaterials. Subsequently, using the same concept of chirality applied to acoustic Metamaterials, Torres-Silva & Cabezas [41] proposed a new design method for earthquake resistant structures. Their device is equivalent to an attenuator consisting of a barrier buried into the ground surrounding the building to protect. This method could be very convenient for the upgrading of existing buildings because the installation works of the barriers around them would not affect them.

A very interesting contribution comes from Xiang et al [42]. In their work, the authors introduced the concept of foundation with a layered periodic Metamaterial consisting of concrete and rubber, tuned in order to have the same resonance frequency of the overhead building.

Based on analytical results, the authors, created a scaled model to put in the test, as shown in *figure 22*, one including the Metamaterial foundation and one without any isolation device at the base. By applying different kinds of input signals, they proved that such typologies of foundation can be usefully used as vibration isolators to mitigate potential damages to the structures.

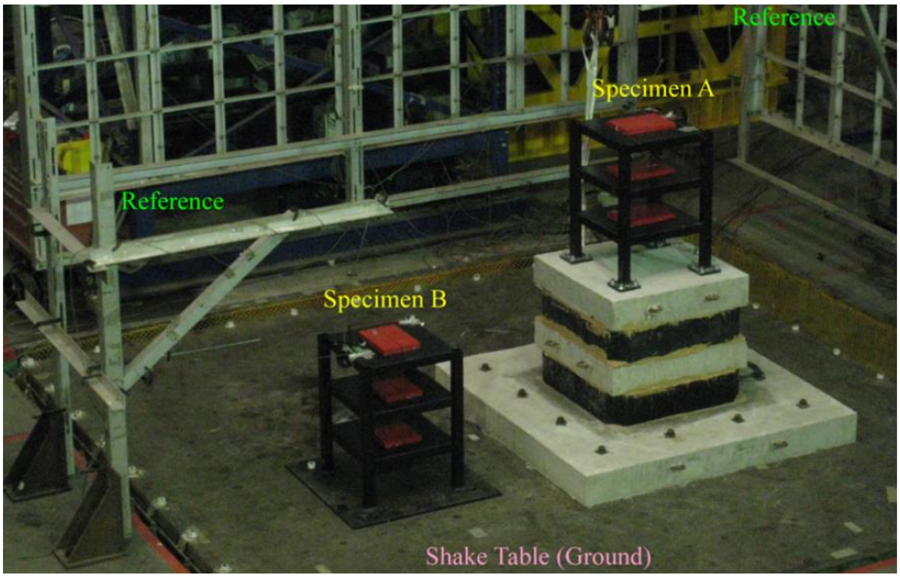


Figure 22 – Test setup: structure with periodic foundation (specimen A) and structure without periodic foundation (specimen B) ([42])

Layered periodic device made by commonly used materials is the same leading idea of the proposal of Casablanca et al. [43]. Their paper introduces a new concept of isolation that mix the characteristics of the standard foundations and periodic Metamaterial in a new paradigm called *composite foundation*. The new device was assembled in a prototype for testing its own capability to filter horizontal displacement. The excellent results achieved during the testing phase can open the way to the development of a new frontier of seismic isolators.

Another special typology of mechanical Metamaterials has been recently proposed with the name of *pentamode* materials as an evolution of the acoustic and elasto-mechanical cloak. Pentamode materials are composite materials that can assure small shear moduli and high bulk modulus. With the aim of realizing innovative seismic device, many prototyping techniques have been presented in literature at both macroscale and microscale.

In the work of Amendola et al. [44], the authors have experimentally investigated the response of their pentamode materials to lateral and vertical force-displacements tests, noticing many analogies with the response of the elastomeric bearings, made by alternating rigid layers and soft rubber layers.

Similarly, in the work of Fabbrocino et al. [45], the authors demonstrated experimentally that their proposed pentamode isolators exhibit a mechanical response comparable to the one of conventional rubber bearings, as shown by the stress-strain diagram in *figure 23*, referring to a cyclic stress test.

Finally, different approaches as the one of dummy structures absorbing significant portion of seismic energy have been proposed over the time as an alternative of the vibration control techniques. In the work of Cacciola et al. [46], the authors set up a novel passive control device called *ViBa*, namely Vibrating Barrier, based on the structure-soil-structure interaction between two vibrating structures and the soil.

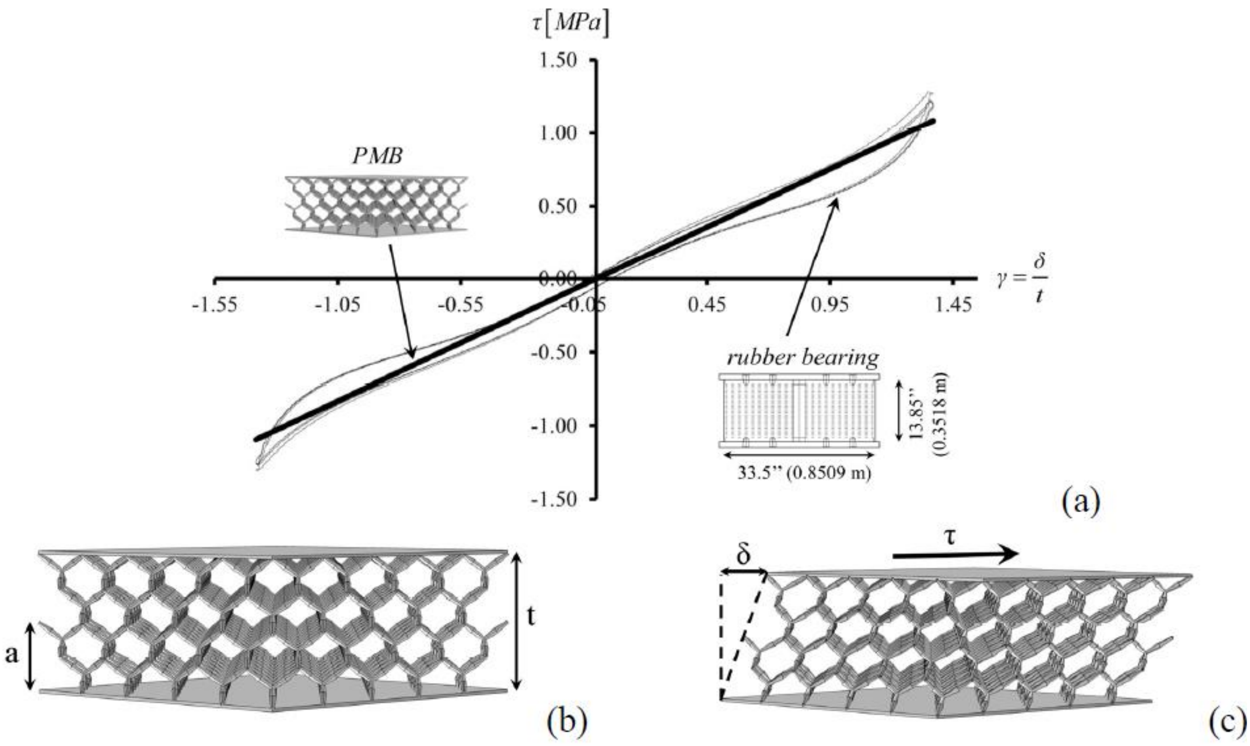


Figure 23 – Comparison (a) between the shear stress vs shear strain response of the pentamode bearings (PMB) and low damping rubber bearings, (b-c) undeformed and deformed configuration of a PMB ([45])

2.2 ISOLATION SYSTEMS CONNECTED WITH THE GROUND

The largest number of proposals for isolation techniques based on the concept of Metamaterials refers to systems embedded in the ground foundations, surrounding the building to protect or surrounding even entire areas to shield. Various approaches demonstrated the ability of Metamaterials to interact successfully with different components of seismic actions, body waves as well as superficial waves. This point, over the time, was an important element that encouraged researchers in pursuing such branch of development on the argument.

The reasons why many scientists followed this line of research is clear and blessed by robust logics. In fact, there are enormous advantages in limiting the impact of seismic waves operating only in a restricted area, where putting in place the isolation system, to get benefits for a number of constructions or big infrastructures.

This outcome would represent a serious step forward for the earthquake engineering. Proposals as the one of Colombi et al. [34] or Brulé et al. [32], just to provide examples, aim to this point but still have an approach based on the physics of the topic, rather than on the engineering aspects. Nevertheless, their valuable ideas deserve a fine-tuning, getting the most of their potentialities and exploring some possible flaw in order to improve them and making them ready to operationalize.

To that aim, after having experienced and validated these new isolation techniques, the key point is definitely, their immersing in the real, random conditions of the ground foundations. Therefore, the full knowledge of the ground conditions, geological characteristics, geometry, dynamic behaviour and all is included in this field is essential to figure out the best way in which such kind of Metamaterials can be exploited.

Besides, Colombi et al [34] proposal of using forests as natural Metamaterials where trees are considered as sub-wavelength resonators, for instance, is led by the parameter of the acoustic impedance between the ground and the trees.

Sensitivity of the result is obviously depending on the mechanical properties of the principal actors of the experimentation as well as on the geometry and homogeneity of the upper strata of the ground.

Similarly, Brulé et al [32] (*Figure 18*) highlights how the successful application of seismic Metamaterials hinges on the modification of the global properties of the medium, through a proper engineering action in the ground (as the application of a Metamaterial can be considered) or a more classic ground improvement activity.

Also, Achaoui [36] (*Figure 19*), Krödel [35] and Finocchio [38] point out how site effects can take an important role in strengthening the impact of an earthquake over a site. This was one of the essential issue to deal with for the development of their proposals of shielding buildings or infrastructures over a range of frequency as large as possible.

To go through the topic of the knowledge of ground foundation behaviour, many subjects can be necessary. Geotechnics, geology and geophysics are fields actually closed to one another. In addition, the perspective of the expert who acts as a bridge between the ground knowledge and the consciousness of the intended result to implement isolation systems can make the difference for the fine-tuning of the new devices.

2.2.1 EXPERIMENTAL INVESTIGATION OF THE SUB-SURFACE

Geophysics is the application of physics principles to the study of the Earth in general. More specifically, geophysics investigates Earth conformation, stratification and the physical properties of its own composing materials, especially of the more superficial strata directly connected with human activities.

Geotechnical geophysics is the application of geophysics to geotechnical engineering problems. Such kind of investigations extend usually to a maximum depth of around *100 m* in the vast majority of the circumstances, but they could be pushed further of three, four times and even more in the most relevant cases.

Geotechnical geophysical surveys are carried out on the ground surface along with the classic boreholes with the main purpose to expand the punctual knowledge of the soil to a larger area, with the advantages of seeking out spatial variations of the geometry, the physical properties of the different strata and, if any, singularities.

Geophysical surveys were originally used in large-scale investigations to identify the big buried structure of the Earth' crust, mostly for the oil and gas industry. Since seventies, such techniques spread out as well in the field of smaller scale project, due to the increasing demands of the dynamic characterization of the soil foundation, following the publication of more demanding anti-seismic standards and international codes.

Geotechnical geophysical methods are commonly used to investigate the ground in many types of engineering applications as well as many infrastructural projects, with the scope of [47]:

- Subsurface characterization for the local seismic response analysis under the design of works in areas at risk from earthquakes (as per international standard and codes). This includes the identification of bedrock depth, rock/soil types and properties like stiffness, density, porosity, etc., layer boundaries, water table and groundwater flow, position of the faults and fractures, weak zones like layers where liquefaction might occur, etc.;
- Ground detection of potential cavities in the field of subsidence analysis or location of buried manmade entities, like utilities, etc.

Such type of techniques are often the most cost-effective and rapid means to extract information directly from the surface and they become the more convenient the more the extent of the investigation increases.

Geotechnical geophysical surveys can be used to choose boreholes positions during the planning phases of the investigation and they can extend, as mentioned earlier, the results of the drilling along a larger area considering variations in the geometry and in the material properties.

Additionally, geotechnical geophysical methods present many other advantages like high site accessibility, high portability, a total non-invasiveness and an almost complete operator safety. Geophysical equipment can frequently be deployed beneath bridges and power lines, in heavily forest areas as well as in contaminated sites, in urban area, along steep slopes and in many other areas where the equipment of the other typical geotechnical surveys cannot access or cannot be used.

Furthermore, geophysical methods, unless boring or trenching, leave little if any imprint on the environment. Such aspect can become crucial when the investigation must be performed in environmental sensitive areas or in contaminated sites.

Geophysical methods are conceived to measure specific parameters that are functions of the mechanical and physical properties of the materials in the subsurface and they are generally used to detect the spatial variation of these specific parameters within the area of interest.

They can be categorized as shown in *figure 24*:

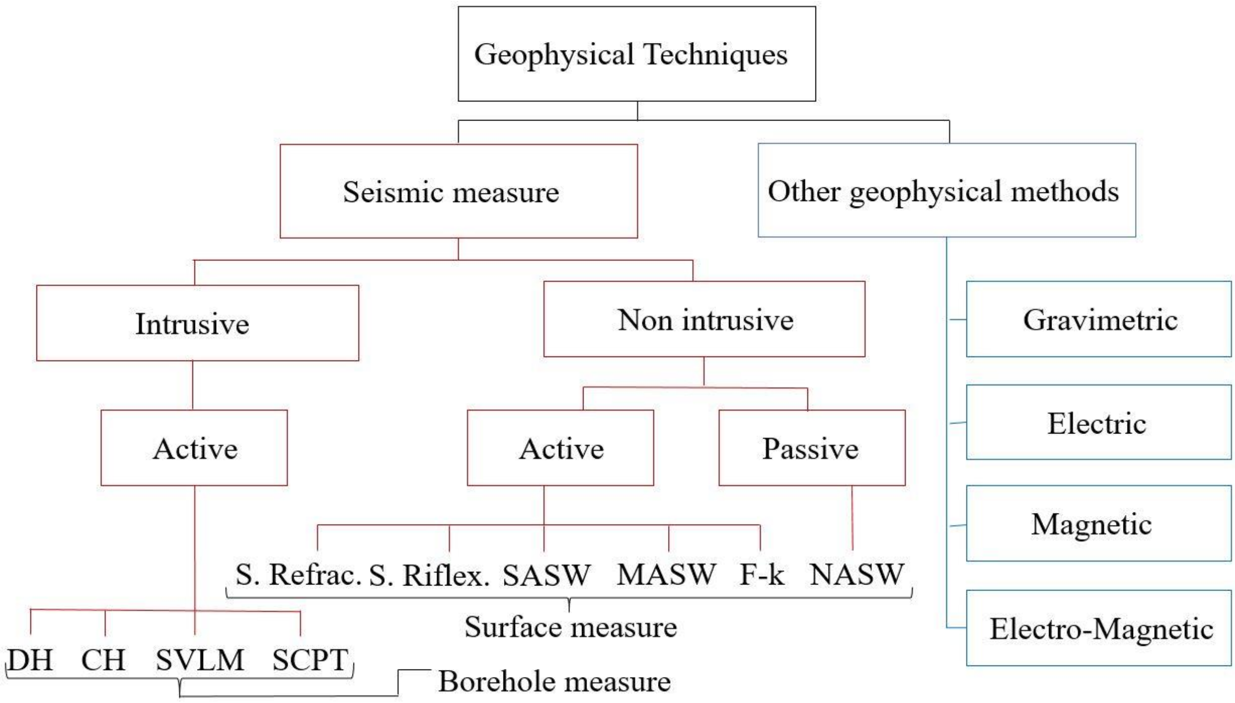


Figure 24 – Classification of the most common geophysical methods

As all typology of technics, also geophysical methods have some restriction.

The most significant limitation stays in its non-uniqueness. In particular, in the event that ground truth data or geological map are not available, one specific geophysical data-set can produce a final model where the solutions that are theoretically correct can be even large. Other relevant limitations are related to the intrinsic nature of the parameters for which geophysical tools are designed to measure, the spatial resolution that such instruments provide and the background noise level.

Furthermore, the accuracy of the acquired information depends on the depth (or in general on the distance), where the measurement are detected. That is because instruments provide increasingly low-resolution targets the more the distance raises. Basically, as a general principle, geophysical tools can detect small targets at shallow depths, but only large targets at greater depths.

In fact, geotechnical geophysical devices have a maximum operating depth of investigation that is irrespective of the size of the target and it is always relevant to point out that some targets can be too deep or too small to be reliably detected using any of these methods. Furthermore, there are other kinds of target that cannot be detected because the difference of their own properties are small in comparison with surrounding environment.

Reflection and refraction methods are by far the most important geophysical methods. The preponderance of these methods over the other geophysical methods is due to various factors such as higher accuracy, higher resolution and greater penetration. Further, their importance lies in the fact that data, if properly handled, furnish an almost unique and unambiguous interpretation.

Data sets derived from seismic methods consist of a series of travel-times versus distances charts. The survey is performed placing in specific locations several seismic sources that energize the ground. The detectors (geophones) are usually located in a line (array or spread) spaced as per the degree of resolution and target distance required.

Geometry is a fundamental aspect of the survey. Detectors catch waves and convert the ground vibrations into an electrical signal. This electrical signal is recorder and processed by the seismograph that turns out as outcome a seismogram as showed in *figure 25 a).*

The knowledge of the travel times to the various receivers and the velocity of waves in the various media enable geophysicists to reconstruct the paths of these seismic waves. The relevant information originate mostly two main categories of path, as shown in *figure 25 b),* reflected tracks and refracted tracks. These paths can be generated using seismic reflection and refraction methods.

a)

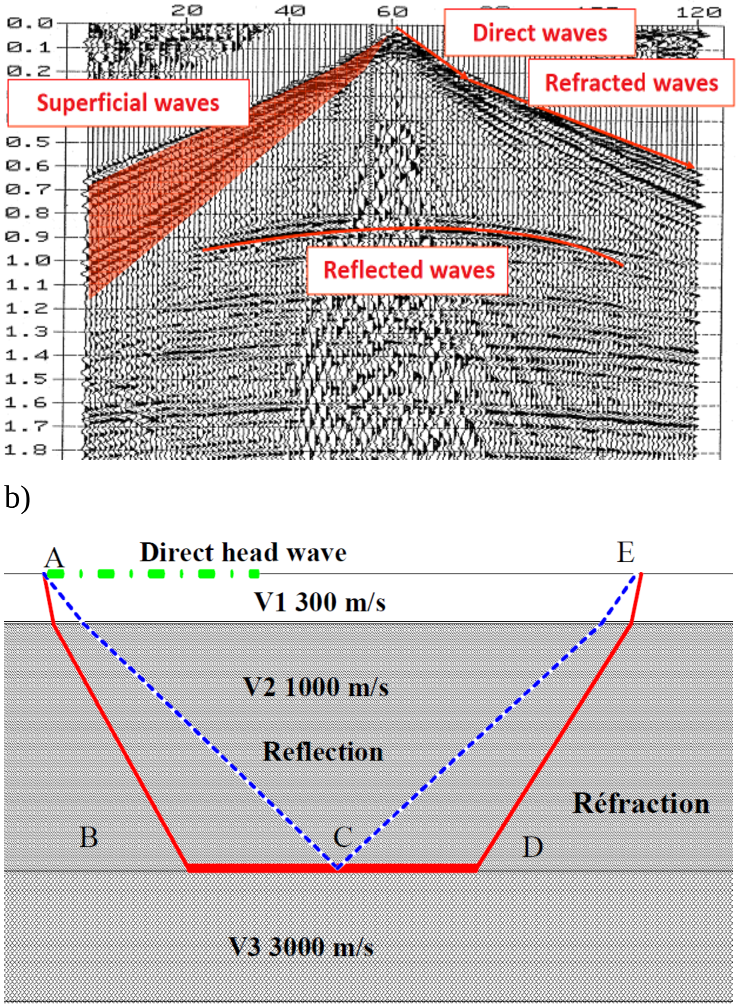


Figure 25 – a) Seismogram calculated by mean of seismic prospection, b) schematic of the wave tracks across the subsurface investigated

In seismic reflection method, waves travel initially downward and, at some point, they are reflected back to the surface, having an overall path essentially vertical. In this sense, reflection method is a very sophisticated version of the echo sounding used in submarines, ships, and radar systems.

Whereas, in seismic refraction method, principal portion of the wave-path is along the interface between two layers and hence approximately horizontal. The travel times of the refracted wave paths are consequently interpreted in terms of the depths of the refracting interfaces and the velocities of each layer where seismic waves propagate across. The objective of the seismic explorations is to deduce information about geometry of the subsurface and physical and mechanical properties of the different layers, based on the arrival times vs distance charts.

Both the seismic techniques have specific advantages and disadvantages when compared to each other and when compared to other geophysical techniques. For these reasons, different industries apply such techniques to differing degrees. For example, oil and gas industries use mostly seismic reflection technique rather than other geophysical techniques. The environmental and engineering communities use seismic techniques less frequently than other geophysical techniques and when seismic methods are selected, the trend is to emphasize refraction methods rather than reflection methods.

Non-intrusive methods count a large number of applications also in the category of geophysical methods that not include seismic measurements, such as electric or magnetic measures. Particularly, geo-electric surveys are widespread thanks to their capability to picture large portion of subsurface in a quite fast and cost-effective way.

The purpose of electrical surveys is to measure the subsurface resistivity distribution from the ground surface in order to produce the imaging of the subsurface. Such technique has been used traditionally in hydrogeological, mining and geotechnical investigations and recently for environmental surveys as well.

The fundamental parameter for electrical measurements is the resistivity of materials. This can be related to several geological parameters such as the fluid and mineral content, porosity and degree of water saturation in the ground, percentage of fracturing in the rock, etc.

Basically, the procedure consists of injecting current into the ground through two current electrodes (*C1* and *C2*) and measuring the resulting voltage difference at two potential electrodes (*P1* and *P2*). The driving formula for the measurement of ground characteristics is *equation 2.2*:

$$p_a = k \cdot \frac{V}{I} = k \cdot R \qquad \text{Eq. 2.2}$$

Where p_a is the apparent resistivity value calculated, I is the current, V is the voltage, R is the electrical resistance and k is a geometric factor depending to the arrangement of the four electrodes, as described in *figure 26*.

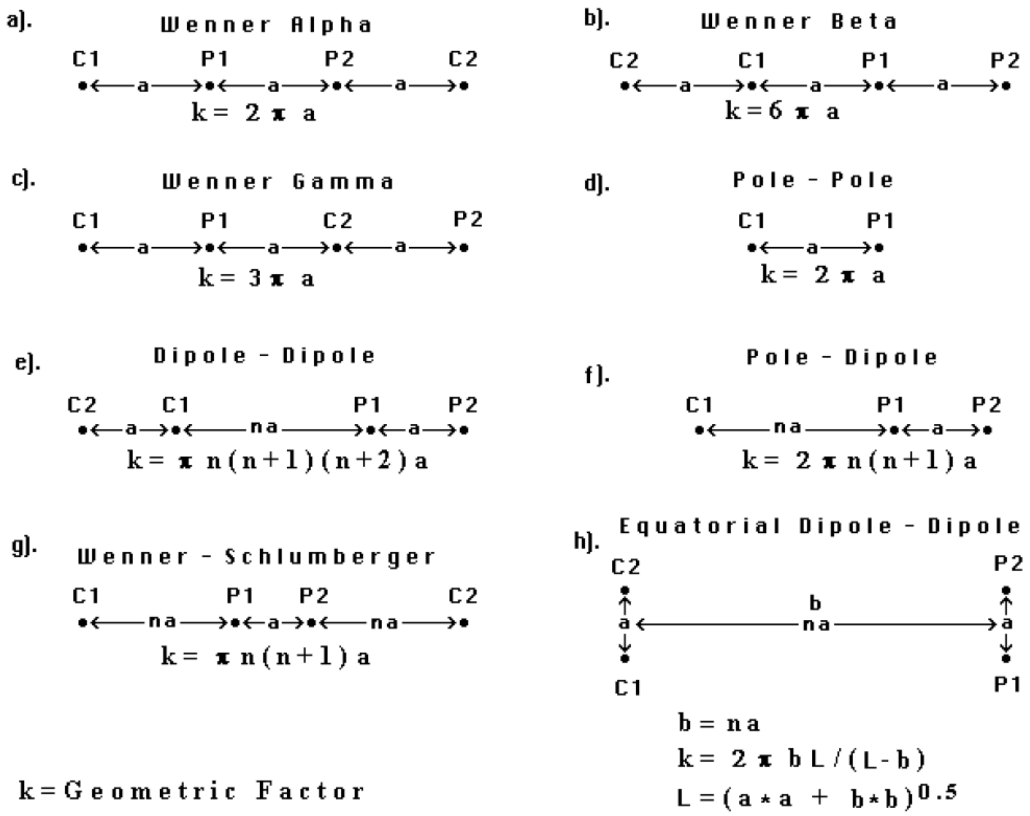


Figure 26 – Most common arrays used for geoelectrical survey and their geometric factors ([48])

Each geometric array has own characteristics with different capabilities and this factor is one of the most important in the geo-electric technique.

The calculated resistivity is not the true resistivity of the subsurface but an apparent value that is referred to a virtual homogeneous ground, which gives the same resistance for the same arrangement in place. The relationship between the apparent resistance and the true resistivity is complex and the extraction of true values needs an appropriate computer calculation.

Electrical resistivity tomography can be carried out with different arrangement: 1D, 2D and 3D. In many geological situations, 2D electrical imaging surveys can give useful results that are complementary to the information obtained by other geophysical method. Usually they are used in conjunction with seismic or ground penetration radar surveys as they provide complementary information about the subsurface.

2.2.1.1 FIELD INVESTIGATION ACTIVITY IN GEORGIA

The experimental investigation has been carried out in the context of the "*Preparation of feasibility study and detail design for E-60 highway section from Natakhtari to Rustavi (Tbilisi Bypass)*", a still in progress infrastructural engineering project, developed by a Joint Venture of three Italian engineering companies. The scope of the work consists of the upgrade of the stretch (about 80 km) of highway circling around Tbilisi (Georgia).

The design team performed a thorough and extensive investigation of the design area in order to determine a satisfactory geotechnical model based on which all calculations for the concerning works, such as bridges, tunnels, embankments and all correlated structures, will be carried out.

The extent of the project and the complexity of the geological scenario, including areas of serious geological instability, have led to planning a large analysis and investigation of the ground site conditions along the potential corridors for the passage of the infrastructure.

Investigations consisted of new boreholes, in situ tests and laboratory tests together with a preliminary collection of data coming from foregoing investigations and publications about the area of the project.

Designers have planned geotechnical surveys based on a well-defined geological frame that comprises all the pertinent requirements to evaluate potential cases of instability. In this undertaking, the Joint Venture asked for consultancy to the *Institute of Earth Science and National Seismic Monitoring Center* of *Ilia State University*, as a corporation depositary of a vast knowledge about the area of interest [49] [50] [51] [52] and valuable professional competences to develop these items of the project.

The planned surveys were required to clarify all the aspects related to the stratigraphy, topography, physical and mechanical parameters of all the layers of the subsurface, included the bedrock. To do that, designers in liaison with the advisor have taken into account, case by case, the appropriate extent of ground and the analytical method for the verification of the structure in the design.

For the purposes of this thesis, characterization of the soil related with the seismic response analysis have been brought to the attention. Specifically, the results of seismic refraction profiling and geo-electric imaging have been reckoned as powerful tools for the evaluation of the interrelations between the soil and new isolation systems based on the concept of Metamaterials, in a real random design context.

Seismic sections shown later have the same data acquisition geometry hereinafter referred:

- n° *32* geophonic stations (*32* vertical geophones and *32* horizontal geophones)
- *5 m* station spacing;
- *155 m* section length;
- n° *9* shots for each section;
- shot every *4-5* stations (*15-20 m*);
- shots with hammer blows in vertical and horizontal polarized mode.

Technical equipment consisted of 2 *24*-channel Geometrics Geode collecting signals coming from 32 vertical geophones (*40 Hz*) and 32 horizontal geophones (*14 Hz*). Shots were executed with a *10 kg* sledgehammer and data interpretation were performed through specialist software. The results of the identification of the geophysical units are expressed in elastic parameters:

- section in gradient Velocity V_p,
- section in Velocity V_p,
- section in Velocity V_s;
- log of v_{s30} (Eurocode *EC8*);
- section in Poisson Ratio;
- section in Young Modulus E_0;
- section in Shear Modulus G_0.

Table 3 displays formulae used to calculate elastic parameters as a function of V_p and V_s:

Elastic Parameter	Formula	Dimension
Poisson Ratio ν	$\dfrac{V_p{}^2 - 2V_s{}^2}{2(V_p{}^2 - V_s{}^2)}$	-
Volumetric weight $\gamma_0 = \gamma_{din}$	$0{,}51 \cdot V_{p\,m/sec}^{0,19}$	t/m^3
Density $\delta_0 = \delta_{din}$	$\dfrac{\gamma}{g}$	t/m^3 (g = 9,8 m/sec)
Elasticity Modulus $E_0 = E_{din}$	$V_p{}^2 \cdot \delta_{din} \cdot \dfrac{(1-\nu)(1-2\nu)}{(1-\nu)}$	Kg/cm^3
Shear Modulus $G_0 = G_{din}$	$\delta_{din} \cdot V_s{}^2$	Kg/cm^3

Table 3 – Formulae for calculation of the elastic parameters as a function of V_p and V_s

It is important to highlight that dynamic moduli are always higher than the static ones. In fact, during in situ tests, seismic pulses provide only instantaneous deformations in the soil because they are very short lasting, providing limited stress level.

However, shear modulus G_{din} has a very prominent role for the study of the seismic conditions of the soil foundations. *Table 4* exhibits seismic parameters directly depending on G_{din}, without forgetting the simplified solution for the seismic analysis response provided by the Eurocode *EC8* using $v_{s,30}$ factor.

Elastic Parameter	Formula	Dimension
Seismic Stiffness *R*	$\delta_{din}\sqrt{\frac{G_{din}}{\delta_{din}}}$	t/(m^2·sec)
Fundamental Period *T*	$\frac{4H}{\sqrt{\frac{G_{din}}{\delta_{din}}}}$	sec
Impadance Ratio λ	$\lambda = \frac{\delta_{din1}\cdot G_{din1}}{\delta_{din2}\cdot G_{din2}}$	-

Table 4 – Elastic parameters for the seismic analysis response directly depending on G_{din}

Geophysical surveys shown later have been carried out along an area dominated by sand and gravel alluvial deposits, conglomerates quaternary (Holocene) and fluvial terraces. The main goals of the surveys are:

- defining the geometry of the interfaces among different layers;
- assessing elastic parameters for each layer of ground;
- defining the borders of the alluvial deposit and the position of the bedrock.

Isolines and false colours indicate velocity values and the main stiffness interfaces, displayed with lines in bolt which have been identified through gradient analysis.

Table 5 shows the main interfaces identified on the basis V_p and V_s:

1st interface	V_p = 1,50 km/sec	V_s = 0,50 km/sec
2nd interface	V_p = 1,95 km/sec	V_s = 0,80 km/sec
3rd interface	V_p = 2,75 km/sec	V_s = 1,10 km/sec

Table 5 – Main interfaces identified with seismic refraction survey

Such interfaces act as a separation between four geophysical units which are assessed as alluvial deposits of incremental density or cementation with depth. Geophysical units are listed in *Table 6*:

1st geophysical unit	V_p < 1,50 km/sec	V_s < 0,50 km/sec
2nd geophysical unit	1,5 < V_p < 1,95 km/sec	0,50 < V_s < 0,80 km/sec
3rd geophysical unit	1,95 < V_p < 2,75 km/sec	0,80 < V_s < 1,10 km/sec
4rd geophysical unit	V_p > 2,75 km/sec	V_s > 1,10 km/sec

Table 6 – Geophysical units identified with seismic refraction survey

Ultimately, v_{s30} value, relative to Eurocode *EC8*, is calculated along single vertical of tomographic calculation cells and the median value for the whole section.

The result of two examples, called in this thesis, *Seismic Section 1* (*SeSe1*) and *Seismic Section 2 (SeSe2)*, particularly representative of the large investigation, are displayed in the following *figure 27 to 40*:

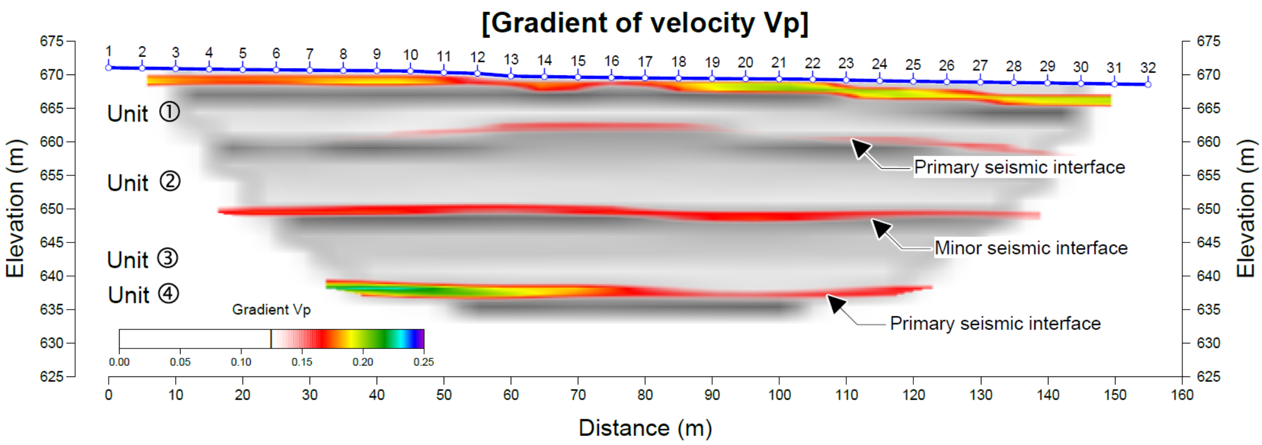


Figure 27 – SeSe1, gradient of velocity V_p

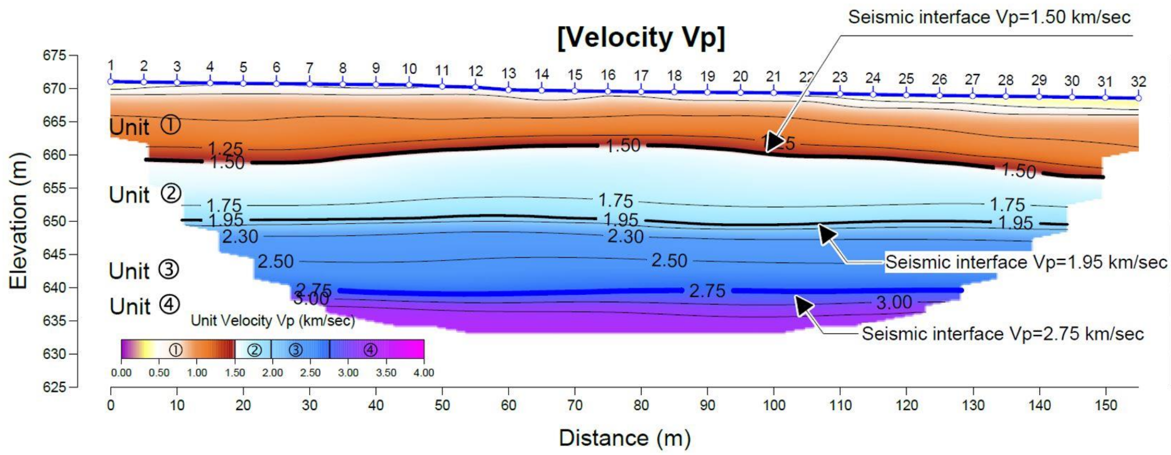


Figure 28 – SeSe1, velocity V_p

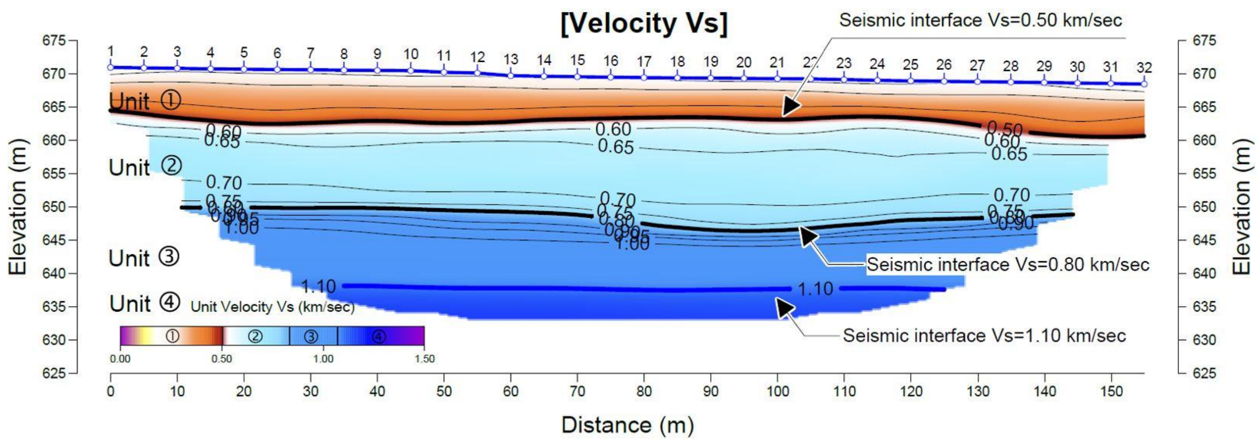


Figure 29 – SeSe1, velocity V_s

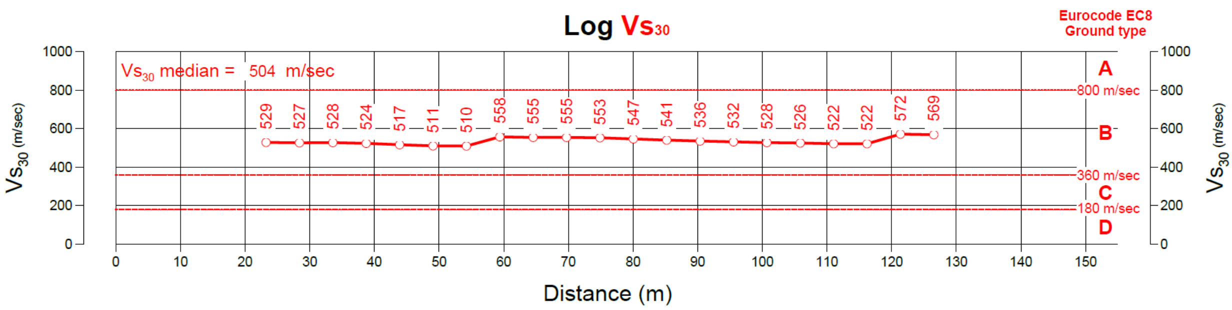


Figure 30 – SeSe1, log V_{s30}

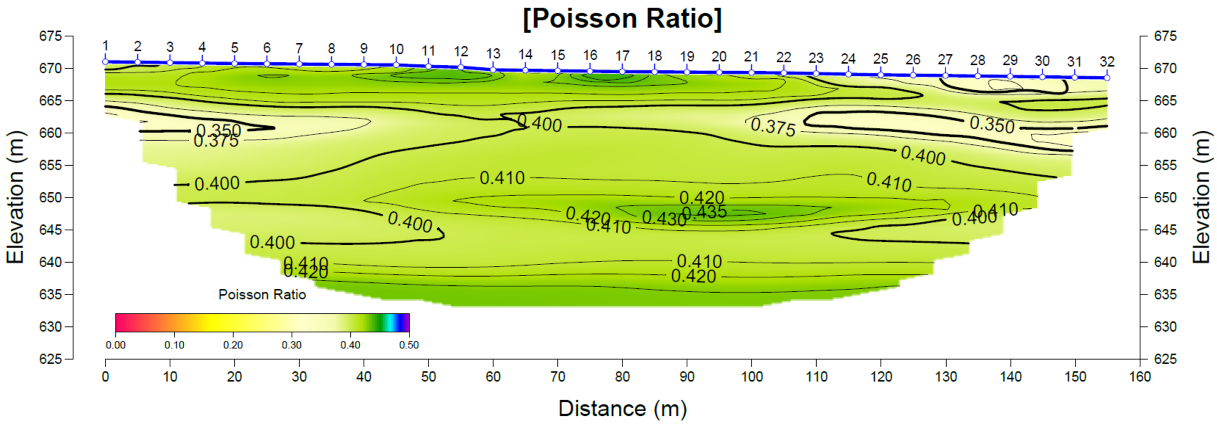


Figure 31 – SeSe1, poisson ratio

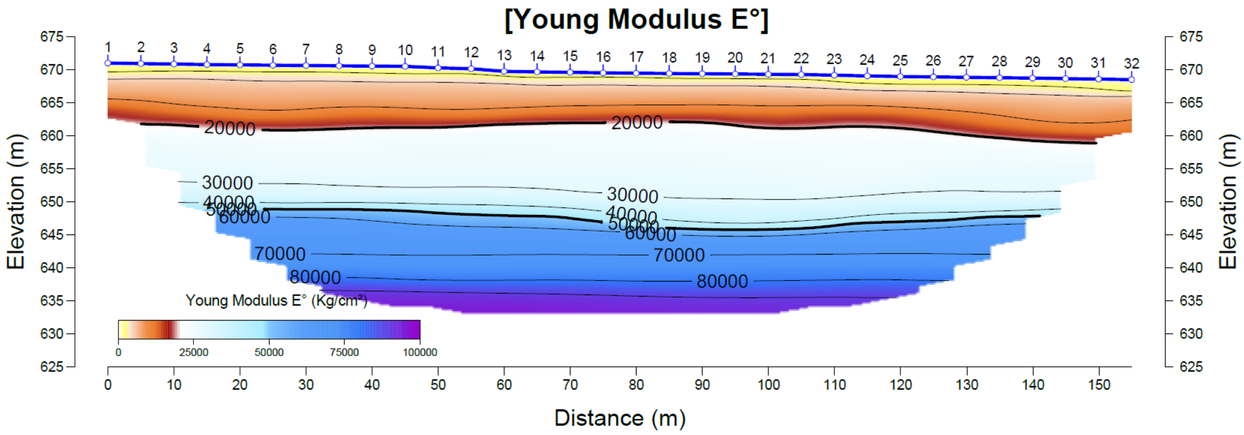


Figure 32 – SeSe1, Young's modulus E_0

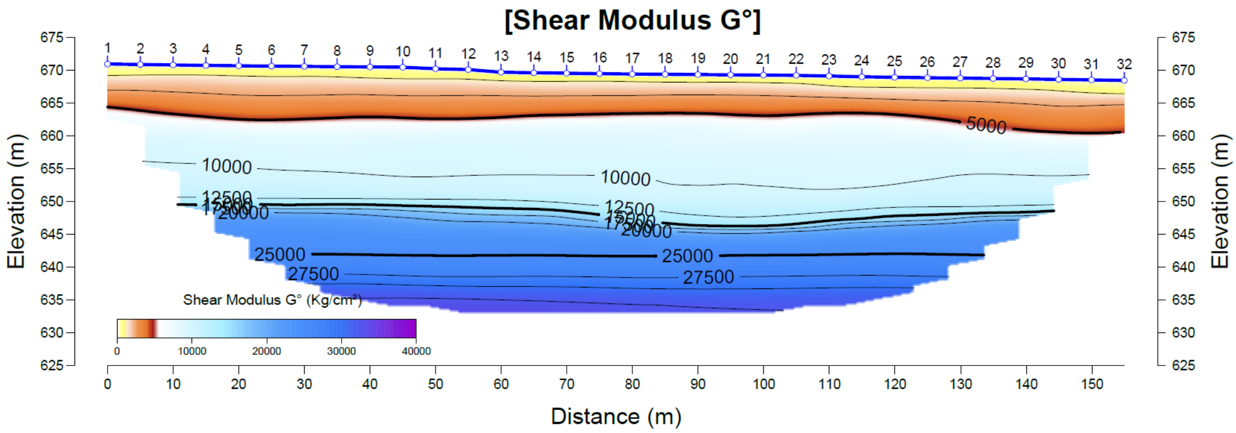


Figure 33 – SeSe1, shear modulus G_0

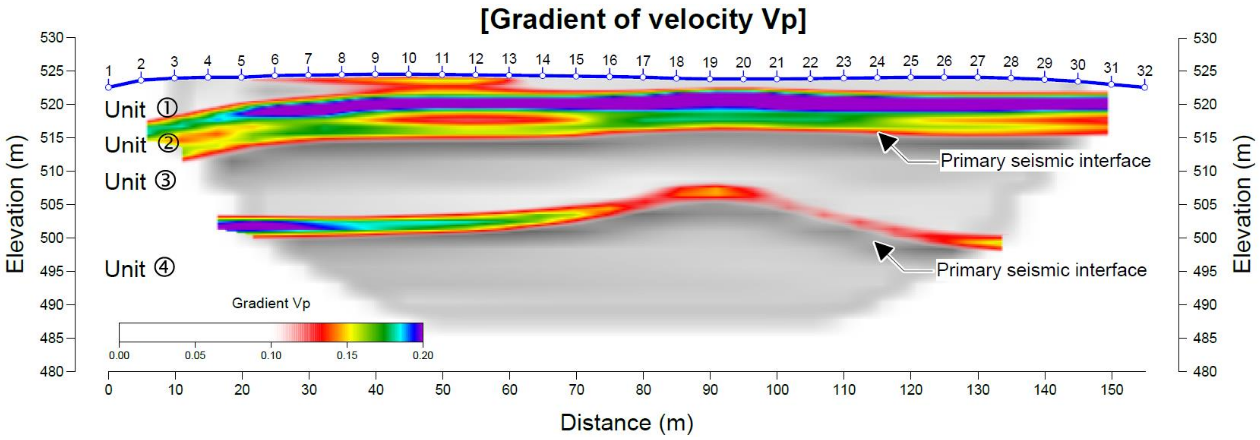


Figure 34 – SeSe2, gradient of velocity V_p

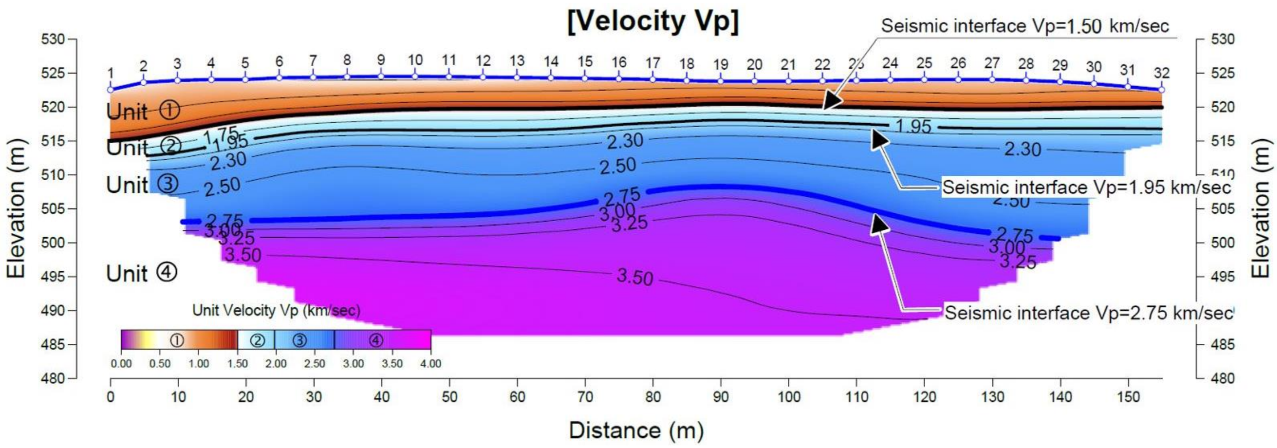


Figure 35 – SeSe2, velocity V_p

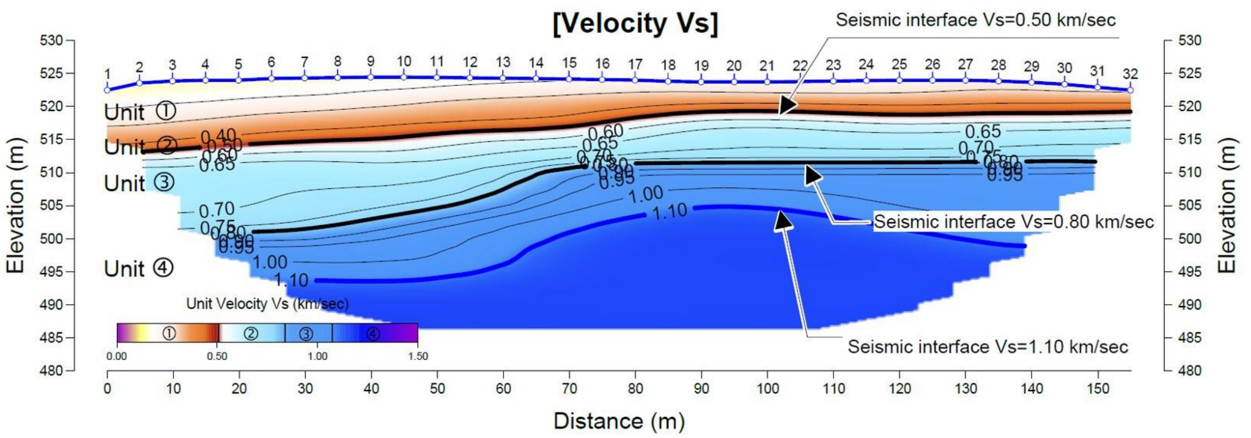


Figure 36 – SeSe2, velocity V_s

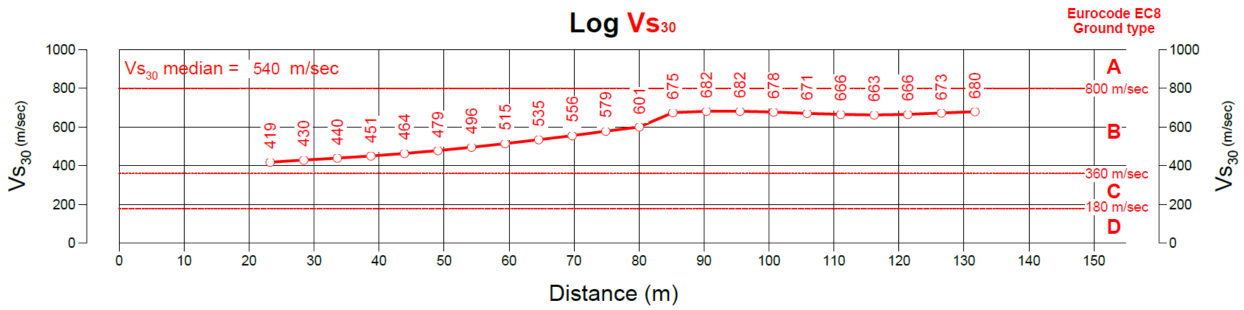


Figure 37 – SeSe2, log V_{s30}

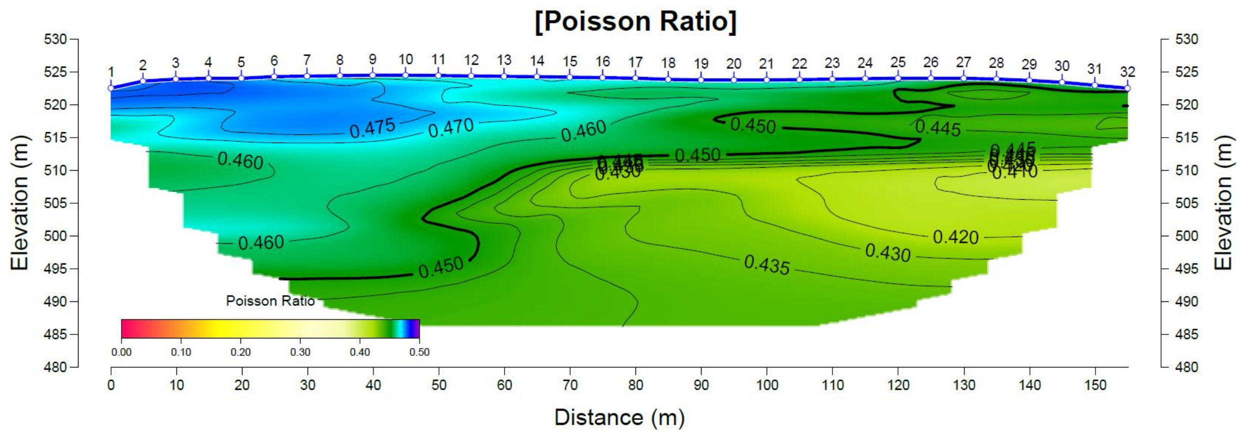


Figure 38 – SeSe2, poisson ratio

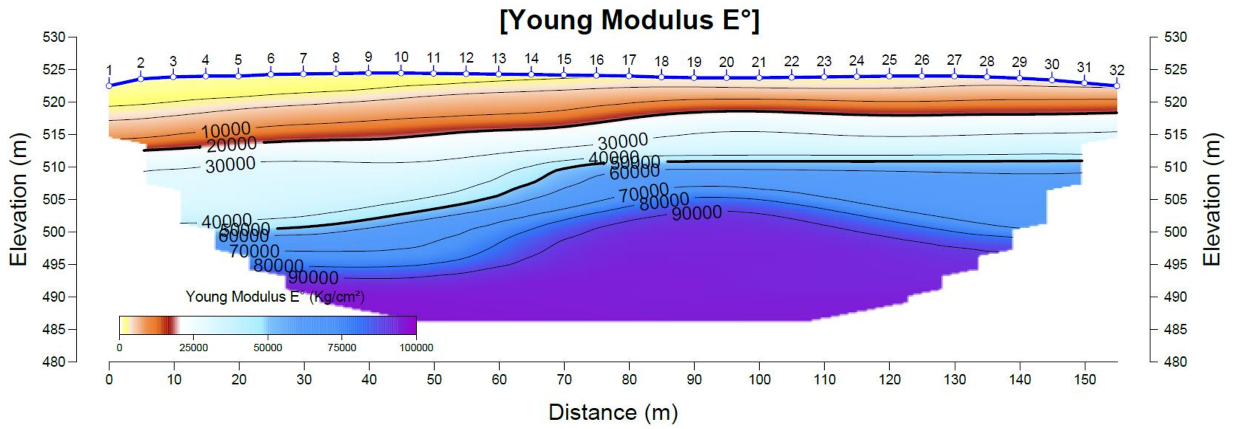


Figure 39 – SeSe2, Young's modulus E_0

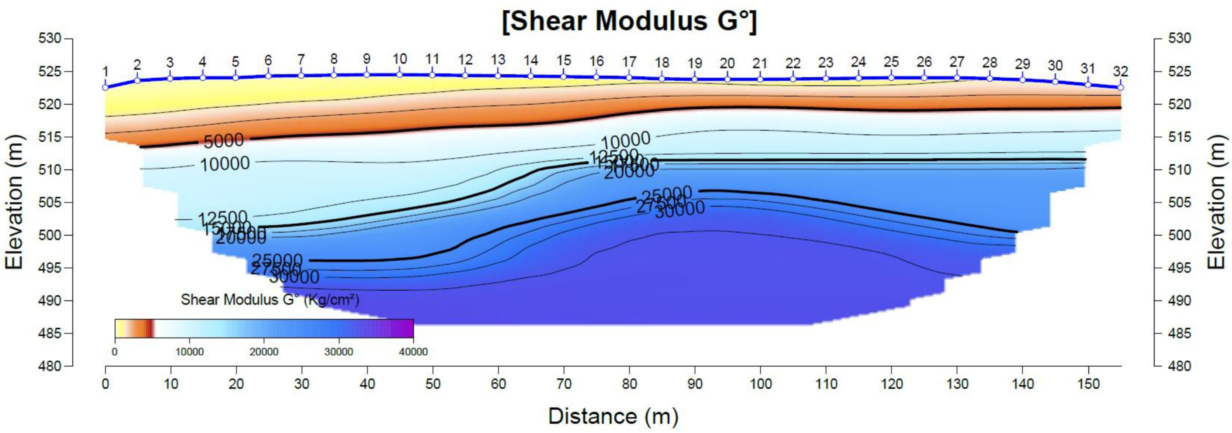


Figure 40 – SeSe2, shear modulus G_0

Geo-electric sections described in this thesis have been carried out with different acquisition geometries but with the same equipment:

- length of *150 m* with station spacing of *5 m* (a total of *31* stations) and length varying from *830 m* to *1310 m* with station spacing of *10 m* (from *84* to *132* stations in total);
- Geo-resistivimeter *96*-channels;
- Receivers electrode and connector;
- Power supply of *12 Volt battery 100*;
- Appropriate 2-D finite element methods (FEM) software;

A mixed Wenner-Schlumberger configuration was selected as the most suite for the survey. Acquisition time (*0,5 seconds* divided in *0,25* for measuring resistivity and *0,25* for the discharge curve) was tuned on the basis of the type of material and research to perform, while the maximum voltage was set on *400 V*.

As for the processing phase, electrical tomography consists of the 2-D inversion of surface electrical profiles obtained by the array measurements according to the method chosen (Wenner-Schlumberger) and determined based on the research goals as well as local electrical stability in the field.

Data processing followed a multi-step procedure. The first step is the reconstruction of resistivity pseudosections by means of contouring software and a preliminary filtering of measurements, especially in the occurrence of buried obstacles. The second step involved calculation of the real resistivity and chargeability values using 2-D inversion along with the elaboration of an adequate distribution model of resistivity in the subsurface.

The result of three examples, called in this thesis Electrical Section 1 (*ElSe1*), Electrical Section 2 (*ElSe2*) and Electrical Section 3 (*ElSe3*), particularly representative of the large investigation, are displayed in the following *figure 41 to 43*. Especially *figure 43* consists of three merged elements that have been acquired separately, but that have been assembled together in order to form a sole global geoelectric section.

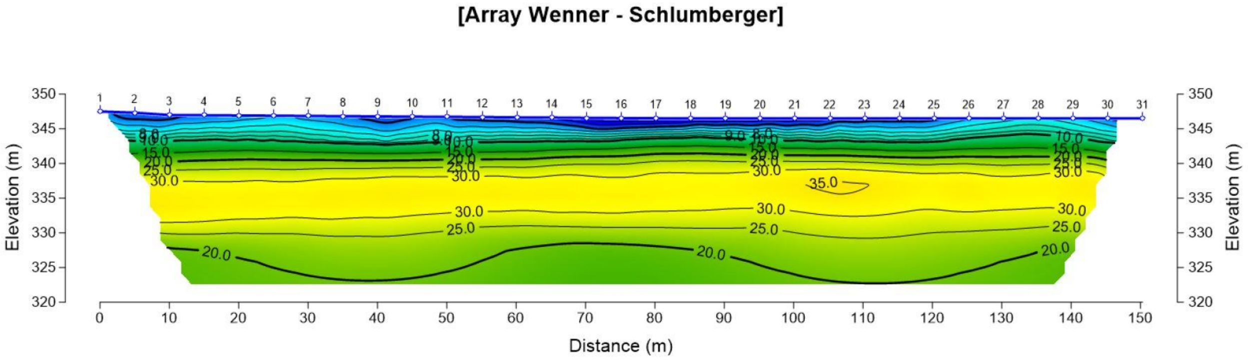


Figure 41 – ElSe1, resistivity field

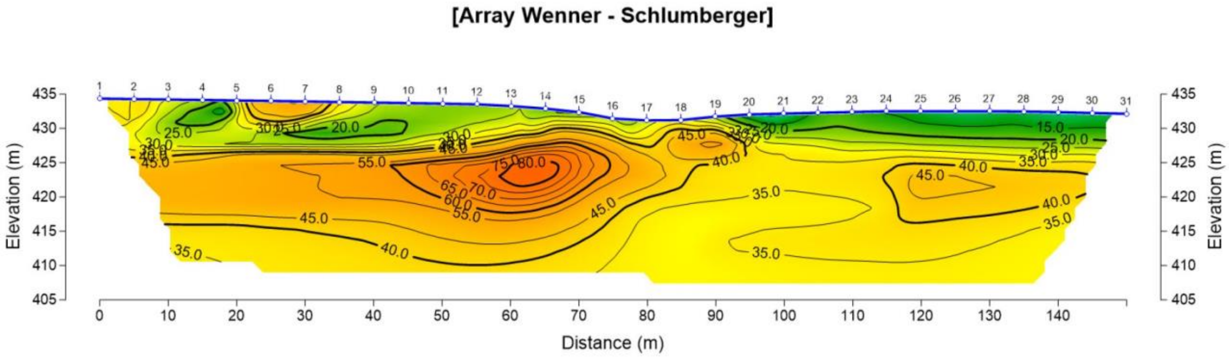


Figure 42 – ElSe2, resistivity field

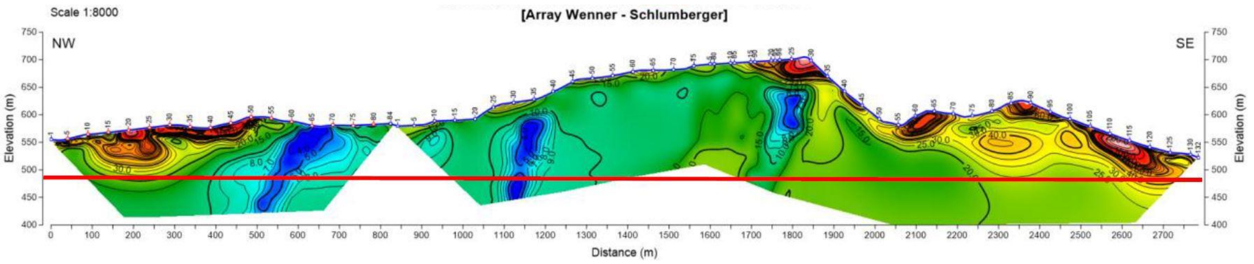


Figure 43 – ElSe3, resistivity field

2.2.2 ASSESSMENTS IN THE METAMATERIAL DESIGN PERSPECTIVE

Following the profiling activity, the first consideration coming out is that the practise of modelling the ground foundation as a medium of coherent behaviour for the site effects analysis can sometimes mislead and drawing conclusions that could lead to substantial errors.

Often, using approximations for the calculation of the seismic action is hardly the only solution to model the ground behaviour so that technical codes themselves come to help suggesting simplified methods. Such simplified methods, a fair number of times, fit quite well with real pictures of ground sub-surface turning out as a result of the investigation. Of course, the common practise of engineering design is adopting conservative approximations ensuring a certain safety margin for the outcomes of the analysis.

Predicting the effects of phenomena of waves propagation in a real soil deposit stricken by an earthquake is not a trivial work, as literature described (Chapter 1), also for ground foundations showing coherent behaviour. Even more, hardships improve in case of high level of heterogeneity and potential singularities that can result by the profiling activity.

The analysis conducted in the frame of the doctorate showed that the incidence of the oddity in the total number of the surveys is not negligible. Therefore, for an upgrade of the isolation technique with Metamaterials, the first issue to deal with is the variation rate and the lateral changes of the ground soil characteristics.

In fact, vertical variations are often taken into account in the models for the analysis of the ground in dynamic conditions because this circumstance is easy to occur, depending of the state of stress due to the weight of the ground. On the other hand, lateral changes as well as buried structures or singularities are much more complicated to be implemented, determining inexorably less accurate level of predictions.

Lower accuracy in the site effects evaluation can be determined as well by the typical large variability of the soil characteristics that has been brought to attention by means of the previous Figures. Nevertheless, with the purpose of obtaining a reliable sub-surface profile to be used for the evaluation of the site effect analysis, the most determinant factor is identifying precisely the bedrock positioning because possible inaccuracy in this point can entail substantial mistakes and cases where the bedrock shows sharp deflections or discontinuities are not so rare in nature.

Shortly after, the other determinant point is defining as appropriate the positioning of the water table and the estimation of the hydraulic regime in order to investigate as well the possible presence of risk factors due to global collapse mechanisms.

These preliminary considerations are all relevant aspects for the evaluation of the principal natural frequencies of ground foundation deposits, thus they became fundamental for the application of the seismic Metamaterial filter as well.

The same preliminary considerations highlighted the importance of creating a Geological Reference Model and a Geotechnical Model in order to be able to select the most suitable parameters of the ground, if any pointing out cases of potential instabilities, like liquefaction, ground subsidence or landslides. In any one of this occurrence none isolation system could face the calamity, consequently the best solution to secure a foundation site has to be found in the geotechnical engineering first of all.

Furthermore, considering the vast corpus of case histories that can turn out from the surveys, the best solution that, in general, can be proposed to make more effective and reliable Metamaterials embedded in the ground is providing an increase of soil mechanical parameters through the process of installation of Metamaterial devices themselves in the ground.

The effect of this solution can have several positive aspects. First of all, this could lead to a more homogeneous condition of the ground in the site, removing variability of the parameters and improving accuracy in the calculations. Moreover, this would bring to a more predictable behaviour of the soil under dynamic conditions.

In fact, all the proposals mentioned in this Paragraph base their analysis on elastic media, not taking into account the evolutive non-elastic behaviour of the ground. Nevertheless, non-elastic behaviour is an inherent characteristic of the soil that emerges vehemently when the soil undergoes to high levels of strain (medium or strong earthquakes).

In earthquake engineering, the selection of the appropriate parameters of the soil need careful consideration and a thorough understanding of the associated level of strain. This is because dynamic properties of the soil are strongly strain-dependent. The best way to describe the shear modulus degradation with the increase of cyclic strain is by means of the so-called modulus reduction curve. Modulus reduction curve normalizes shear modulus G with respect of the maximum shear modulus G_0, giving rise to the commonly referred modulus ratio.

Figure 44 [53] shows an example of this argument describing the change of the soil characteristics due to cyclic loading through the so-call soil thresholds. The more the soil undergoes to higher level of deformation, the more it loses its stiffness and increases its ability of dampening dynamic forces.

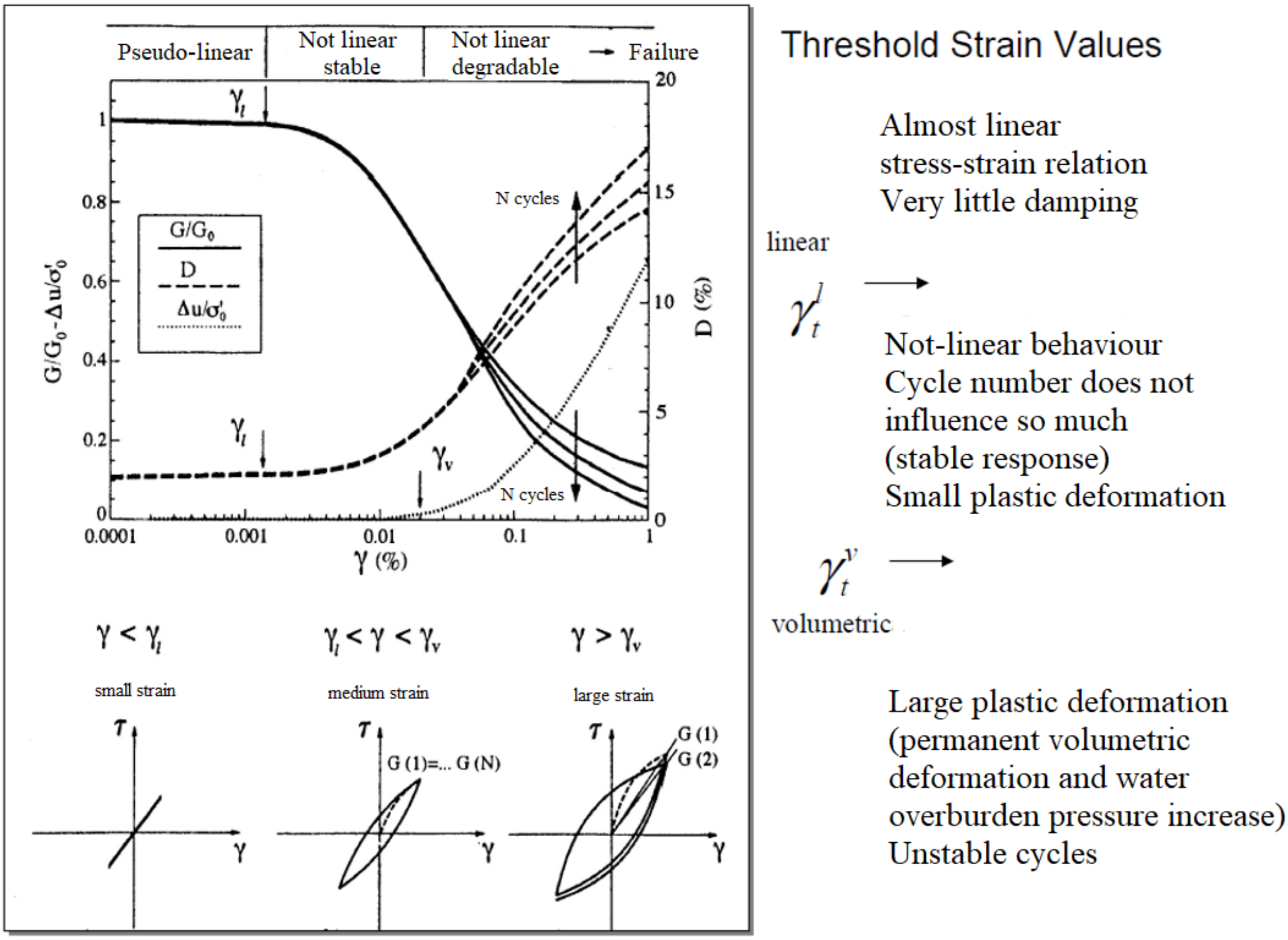


Figure 44 – Modulus reduction curve and cyclic threshold strain of soil under cyclic loading ([53])

Damping measures the energy dissipated in the soil particles by friction, heat or plastic yielding. The relationship of shear strain to damping is inversely proportional to the modulus reduction curve. Damping is often expressed as the damping ratio *D*, which is defined as the damping coefficient divided by the critical damping coefficient.

Often, engineering geotechnical or civil constructions induce low levels of strain in the soil foundation. However, in the occurrence of earthquakes, especially if they exhibit high magnitude and long duration, soil undergoes to heavy dynamic loading inducing large level of strains, as depicted in *figure 45*.

Earthquakes, depending on their magnitude, embrace a large range of strain. Additionally, they determine an evolutive behaviour of the soil foundation that, therefore, lead to variation in the relevant parameters, like shear modulus and damping ratio [54].

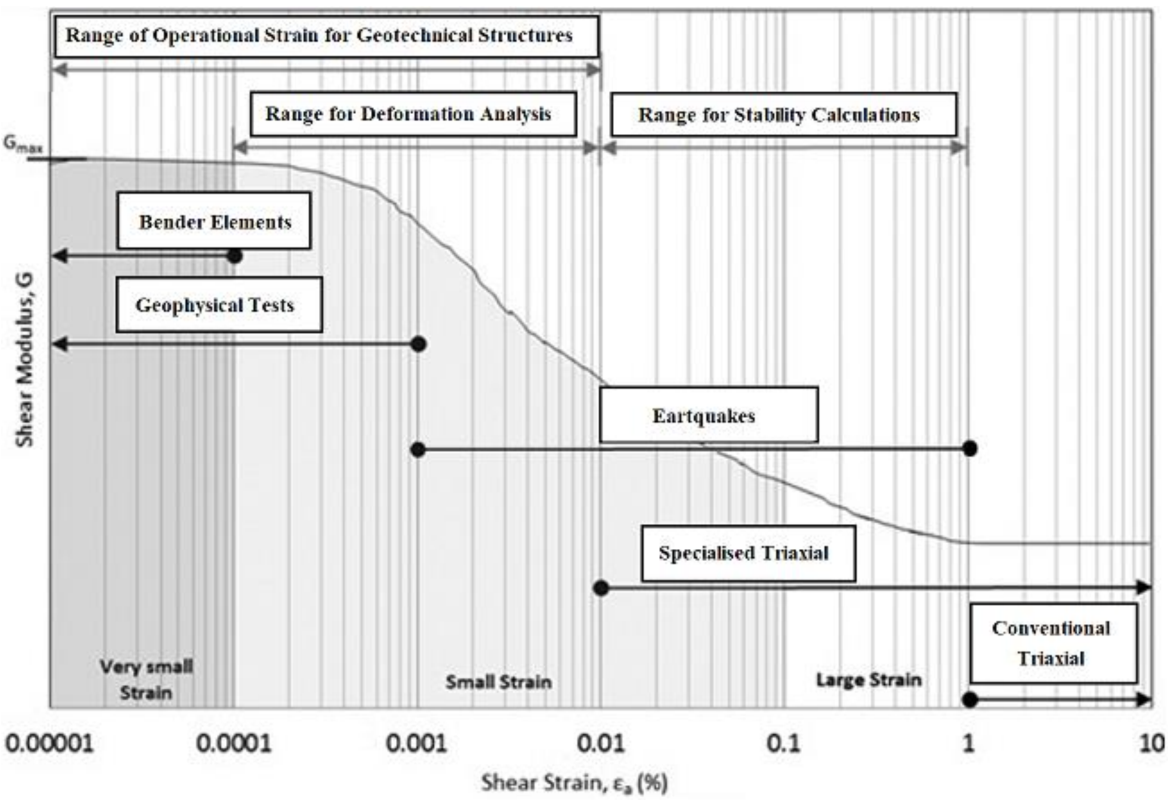


Figure 45 – Level of strain induced by different types of force in the soil ([54])

For instance, geophysical tests propagate seismic waves through the soil at a very low strain level (less than 10^{-3} percent), making practically impossible the measurement of strain. Such low level of strain allows the use of elastic theory to associate measurements with mechanical properties, because the most part the response is linear.

At intermediate levels of strain (~ 10^{-2} percent), this response starts becoming non-linear and at large strains (~10^{-1} *to 5* percent) the dynamic behaviour of soils remains non-linear, experiencing permanent deformation and, possibly, reaching an unstable condition.

Under intermediate and large strain, elastic parameters could not be any more properly representative for soil behaviour and the main features of the site transfer functions can vary considerably (as described in Chapter 1).

Many factors contribute to the stiffness of soils during cyclic loading, such as plasticity index, relative density, mean principal effective stress, over-consolidation ratio, number of cycles and void ratio. Intervene in these parameters can undoubtedly bring to select more stable and precise values for the evaluation of the earthquake effects in the ground foundation.

The installation of an artificial material in the ground, like a Metamaterial, could introduce in the soil a preliminary level of strain, density and over-consolidation, in order to reduce the range of variation of the soil parameters in occurrence of medium and strong earthquakes. In such a case, engineers could better manage all the problems related to wave propagation and tuning the isolation system to the deposits at a more accurate level. Moreover, the benefit of this action would be extended as well to any other kind of issue related to the ground stability.

After such a kind of intervention, the practise of using elastic parameters for calculations, as commonly happens, will be more appropriate. Additionally, performing analysis in elastic domain, rather than taking into account plastic/inelastic behaviour, is undoubtedly more convenient and manageable from the phenomenological point of view, leading to determine more reliable results.

Although this solution could provide a higher precision, the fact of following a cautioned approach in the natural vibration mode tuning between the seismic Metamaterials and the ground foundation is still a good point. Broaden the attenuation of waves in a larger range spectrum remains a must to be pursued.

A completely different approach has the proposal of Colombi et al [34]. On the basis of the analysis carried out above and considering the random principle taken into account in his proposal, the isolation through threes used as vertical resonators can be always performed in any context of project, varying from the small portion of land, like towns or single massive structure like plant, to extended infrastructures.

However, this solution should not be seen as the decisive resolution of the seismic threat but as a support for the mitigation of wave’s effect that can be easily put in place having as the only strict requirement the availability of space.

2.3 ISOLATION SYSTEMS CONNECTED WITH STRUCTURES: A NEW CONCEPT DESIGN BASED ON METAMATERIALS

Casablanca et al. [43] have published the latest version of seismic isolator based on the concept of Metamaterials in 2018. The group of researchers coming from the University of Messina proposed a special configuration of isolator created directly inside standard building foundations. The name "*composite foundation*" points out the fact that such a kind of isolator have been conceived exploiting the well-known characteristics of the technology of Metamaterials, embedding its realization to the typical ground foundation positioned right below the structures.

This new configuration has the advantage of including the isolation system within a necessary structural element performing its function without interfering with the surrounding of the building. Additionally, the massive use of the traditional materials, like concrete and iron, does not alter the construction technology of buildings also taking care of the cost-efficiency of the isolation system. Ultimately, this kind of technology is applicable only for new constructions.

The composite foundation integrates the physics of seismic Metamaterials based on the concept of periodic mass-in-mass systems. Therefore, Metamaterial carries out an action of seismic waves filter in a range of frequency that is supposed to be the one of the first vibration mode of the building to protect. Inside the bandgap, seismic energy can be reduced dramatically, preventing the building to incur in structural hardships under quakes.

The schematic of the composite foundation is displayed in *figure 46*. It is formed by a set of *20 cm* thickness reinforced concrete slabs, whereby the cross-section is sized $1 \times 1\ m^2$, linked among them by special ultralow connections and constrained at the corners by mechanical bonds. The ultralow surfaces consist of a suitable combination of layers of steel and Teflon. Their role is to disconnect each layer providing an appropriate damping rate (very low in this case) to the system. The bonds at the corners are composed of rubber with low Young's modulus with a *15 cm* diameter steel tube inside that keep the correct alignment to the foundation.

Connections are very crucial parts of the composite foundation because they provide low damping rate and ensure relative displacements between every layer. That is what provide deformability to the foundation in dynamic conditions, when the system undergoes seismic waves having frequency inside the bandgap.

Each slab exhibits a grid of nine cylindrical inclusions regularly arranged, with the centres *30 cm* spaced to one another. Such inclusions host a full-body steel block linked to the concrete by means of natural rubber with low Young's modulus, similarly for the mechanical bonds at the corners.

Figure 46 c) shows a magnification of the inclusions. Such inclusions act as internal resonators that, thanks to their shape smaller than the wavelength of the propagating waves, can interfere with them.

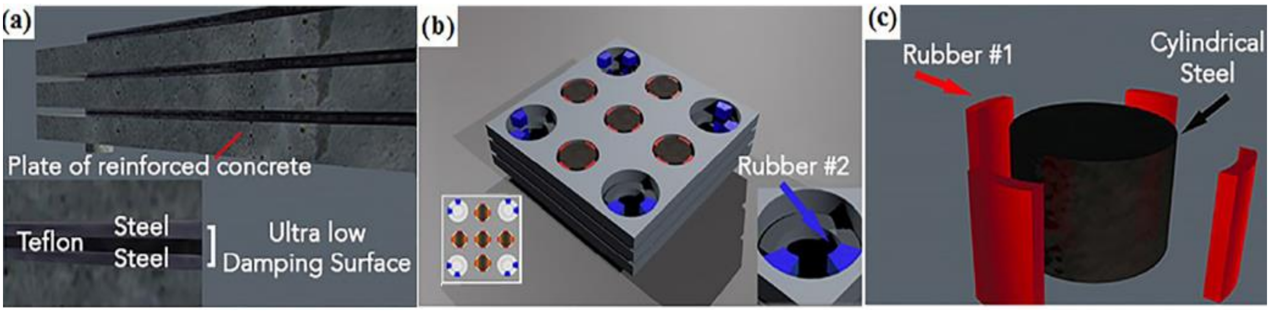


Figure 46 – Composite Foundation schematic: a) vertical layered structure with disconnection elements zoom, (b) top view with spatial distribution of resonators and detail of the corner bond, (c) detail of the internal resonator ([43])

Even in this case, rubber internal connection between the iron resonators and the concrete plates is a decisive element that allows the steel mass to play with the seismic energy. How it is possible to infer, every single part of the composite foundation, especially connections, plays a fundamental role in the behaviour of the whole Metamaterial foundation.

Since the leading idea on the functioning of the composite foundation is the periodic mass-in-mass model, every choice in terms of material and configuration of the components aims to correspond to the theoretical model.

The working principle is shown in *figure 47* and it refers to the periodic mass-in-mass system without dampers:

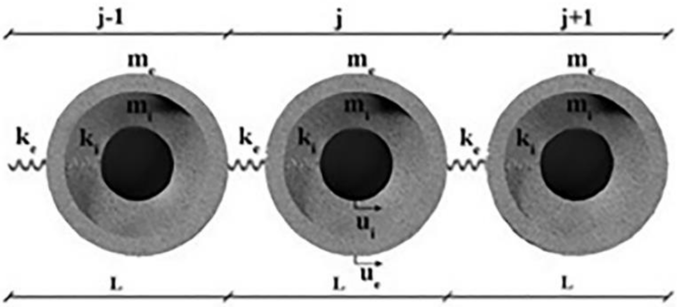


Figure 47 – Periodic mass-in-mass chain: m_e mass, k_e stiffness, u_e displacement for the external cell, m_i mass, k_i stiffness, u_i displacement for the internal cell, L the spatial unit length ([43])

The choice of the mass-in-mass system is due to the better efficiency that this configuration gives rise as regard wave attenuation within the band-gap.

Indeed, the presence of the internal resonators allows this system to push down the acoustic branch (the lowest edge) of the blocked frequency range. From the dynamic point of view, such a kind of Metamaterial does not present negative refraction index but negative effective mass inside the bandgap in dynamic conditions.

The dispersion law for the selected schematic shows an initial value of the bandgap $f_{BG,i}$ expressed by *equation 2.3* and a final value $f_{BG,f}$ expressed by *equation 2.4*:

$$f_{BG,i} = \frac{1}{2\pi}\sqrt{\frac{[k_i(m_e + m_i)]}{m_e m_i}} \qquad \text{Eq. 2.3}$$

$$f_{BG,f} = \frac{1}{2\pi}\sqrt{\frac{\left[k_i(m_e + m_i) + 4k_e m_i - \sqrt{[k_i(m_e + m_i) + 4k_e m_i]^2 - 16 m_e m_i k_e k_i}\right]}{2 m_e m_i}} \qquad \text{Eq. 2.4}$$

Figure 48 depicts a practical application of the composite foundation, with the presence of two load distribution plates, one at the top and one at the bottom of the seismic devices to improve load distribution and, at the same time, the dynamic response among the system composed by the building, isolators and the ground.

It is noticeable the presence of a clearance space between the foundation and the retaining walls to allow the isolation system to move freely without any possible obstruction coming from the surrounding ground.

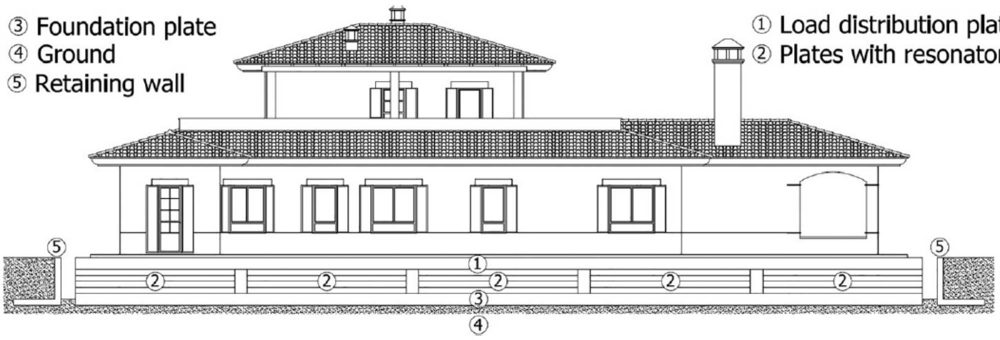


Figure 48 – Schematic of the composite foundation built beneath a residential house ([43])

According to such principles, a prototype of composite foundation has been assembled in order to test its behaviour under dynamic condition, simulating the occurrence of an earthquake.

The final experimental parameters of the mass-in-mass chain computed using the physical and mechanical values of the prototype components are displayed in *figure 49*, along with the corresponding dispersion law:

m_e	317 kg
m_i	245 kg
k_e	155 x 10^3 N/m
k_i	1080 x 10^3 N/m
L	0,2 m
$f_{BG,i}$	≈ 5 Hz
$f_{BG,f}$	≈ 14 Hz

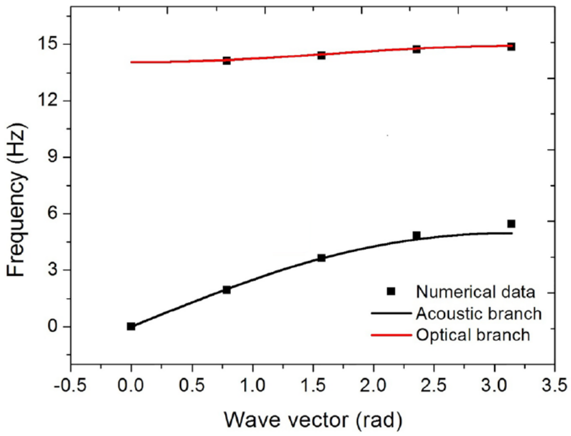


Figure 49 – Dispersion relationship calculated by numerical simulation and theoretical formulation using the physical and mechanical values of the prototype components ([43])

The prototype consists of four layers device, free to move over a fixed slab base. Such fixed layer is the base that in a real building would be the distribution plate. Each layer is disconnected from the others by an ultralow damping surface as indicated in *figure 46* above.

Obviously, also the base plate and the first layer is equipped with the same low-damping disconnection in order to reduce as much as possible friction between the steady slab and the moving one. Damping rate expected by the disconnections is very low and experimental tests confirmed a value of approximately 3 % from them.

All the set-up of the full-scale laboratory test is described in *figure 50*, where it is possible to observe in the details the schematic of the test as well as every components of the prototype, the machine tests and sensors system.

The measurement system is equipped with sensors to register the displacements and the accelerations of each plate and the accelerations of the internal resonators. Sensors consist of displacements transducers and PBC piezotronic triaxial accelerometers connected with the central chassis that processes the data. Low-cost MEMS accelerometers, whereas, have been used to detect resonator' motions inside the grid of concrete, using special care for the arrangement of the instruments because of the little space available between the resonators and the concrete support.

The input displacement consisted of a time-domain signal of the form $u_0 \sin(\omega t)$ with a nominal displacement $u_0 = 3{,}5\ mm$ and with an angular frequency w varying from *0,5* to *8 Hz*. The external force was applied to the second plate (first layer of the device) by a hydraulic actuator. Sensors *T1*, *T2*, *T3* and *T4*, therefore, record the time-domain signals at the same frequency as that of the input signal. The result of the test are shown in subsequent *figure 51* for input frequencies of *2, 4, 5, 6,* and *7 Hz*.

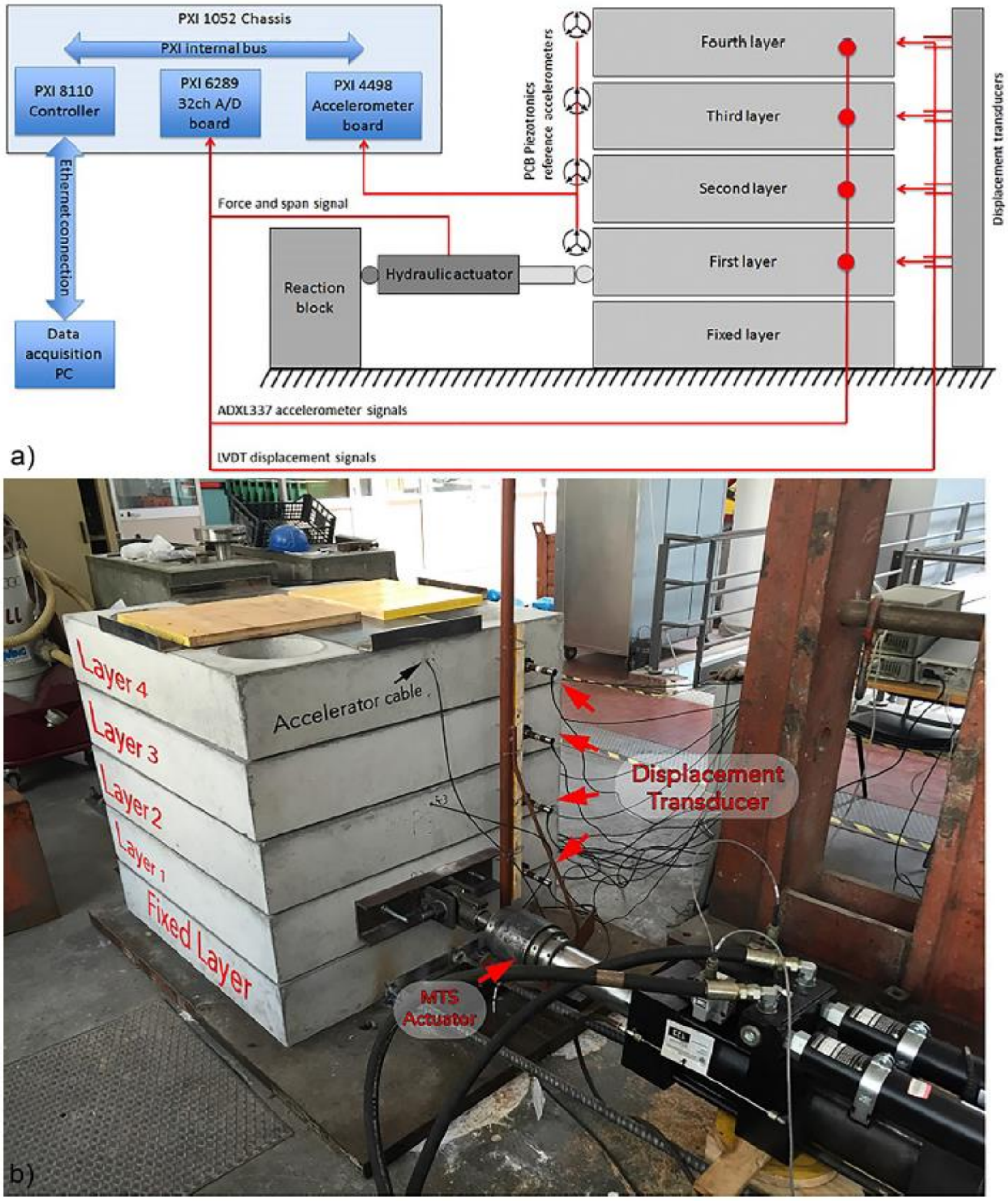


Figure 50 – Determination of the composite foundation isolating capabilities: (a) schematic of the laboratory test with all measure instruments and machineries used; (b) photograph of the composite foundation ready to be tested ([43])

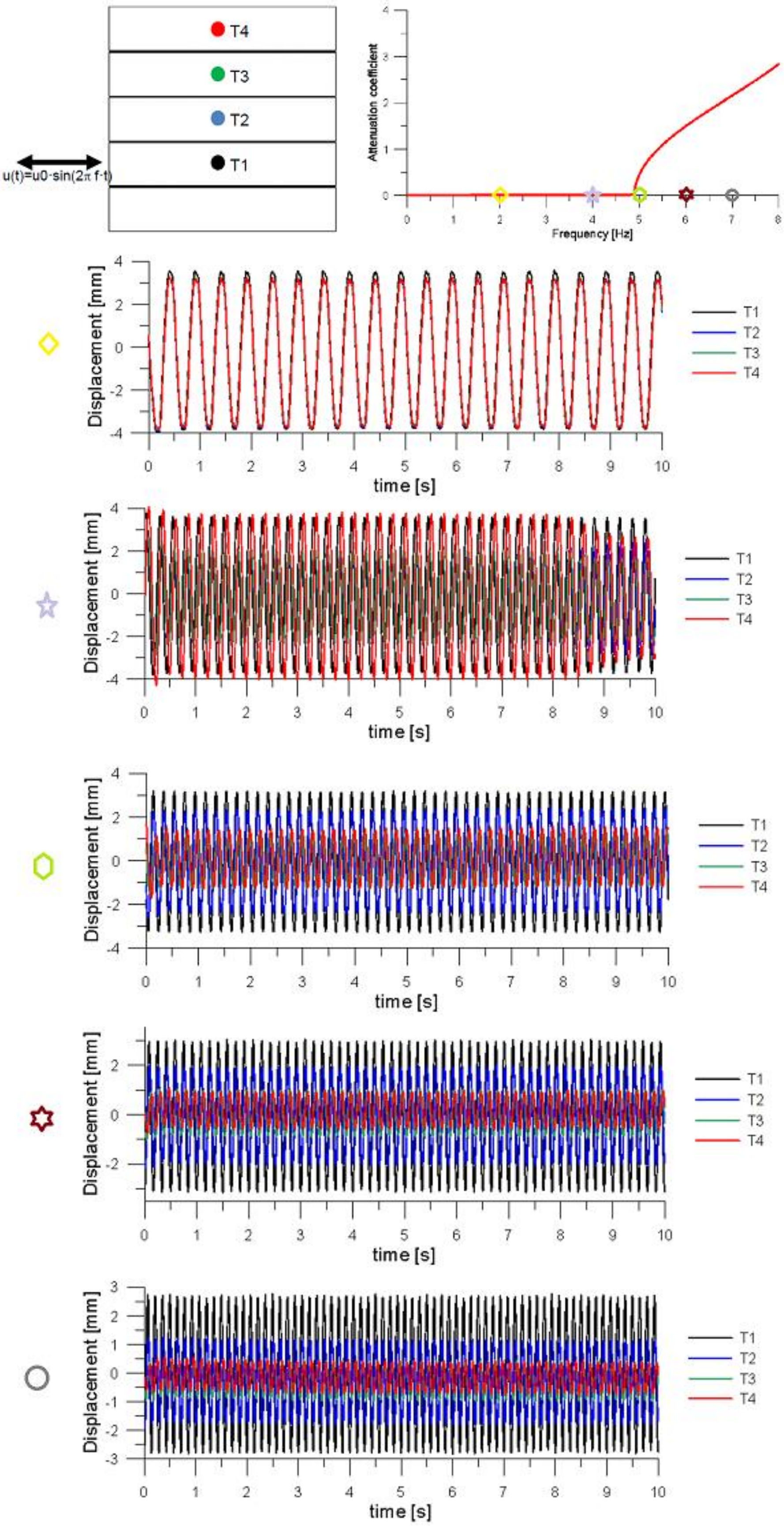


Figure 51 - Time-displacement charts for input signals of 2,4,5,6 and 7 Hz

From *figure 51*, it is possible to differentiate the motion at each layer of the prototype and, above all, it is possible to appreciate that as soon as the signal frequency gets close to the bang-gap region, the Metamaterial attenuates the amplitude.

The maximum attenuation as per experimental results is higher the *50 %* of the initial displacement, a very good find for the research that proves the effectiveness of such technology.

More experiments have been carried out adding weight of *1000 kg* on top of the prototype in order to represent the effect of a construction load. The results showed the same qualitative outcomes.

The direct conclusion of the latter is that a building provided with a composite foundation can have strongly mitigated its structural stress coming from horizontal vibrations in the range of frequency of the isolation system.

A consideration is expressed as regard the working mechanism of the composite foundation. This point is what mainly differentiate more common base isolation systems (consisting of elastomeric bearings, to give an example) to the composite foundation.

As many times mentioned, disconnections among each layer of the composite foundation provide internal deformability to the Metamaterial device when the external force shows frequency in the range of the designed bandgap. In this moment, Metamaterial gets into the action creating a counter-phase movements of the internal resonators that oppose to the seismic solicitations presenting a damping effect.

Ultimately, Metamaterials do not provide the system building-isolator of deformability but they only deliver a remarkable attenuation capability. On the other hand, the principal effect of conventional bearings is increasing system deformability in order to push the building period of resonance to higher values, where usually seismic action is less powerful and resonance phenomenon is unlikely to occur.

Therefore, the big difference between these two techniques of seismic base isolation is in the usage of disconnections: in conventional bearings, disconnections provide deformability to the whole building-foundation while in Metamaterials they allow the development of the isolator dissipating feature.

Referring to the test, the dissipation contribution of the internal resonators at every layer of the composite foundation is clearly visible in *figure 51*. Ideally, as the theoretical model of the periodic mass-in-mass system shows, with an infinite sequence of layers, the 100% of the seismic energy can be dissipated during the propagation through the Metamaterial device. Therefore, seismic action could be even wiped out for good.

Disconnections play a crucial role in this working mechanism because they activate the relative motion among the concrete slabs and, consequently, the counter-phase internal movement that break down seismic energy. Once energy propagates across the isolator the weight of the internal masses and the stiffness of the internal springs give rise to dissipation in the range of the internal resonance frequency.

However, disconnections can represent a weak point of this kind of devices because of degradation. Indeed, the issue of degradation of natural rubber exposed to the air deserves special attention because of the long lifetime required to isolators. Aging is a very relevant characteristic for rubber by virtue of the change in the mechanical properties that this phenomenon implies in the material.

Approximately this leads to the same uncertainties expressed for the ground connected Metamaterials, with the only difference that rubber change degradation is anyway predictable and more easily manageable since rubber texture is artificially designed in the industries.

There are many causes for rubber degradation over the time, like ultraviolet rays, ozone, salt water or acid rain, but the dominant agent is the high temperature effect, even more taking into account the isolator working conditions. High temperature entails a decrease of the antiseismic capabilities of devices that exploit the elastic characteristic of rubber components. Fortunately, deterioration involves a very extended time period at room temperature to affect the longevity of natural rubber compounds.

The effect of aging in rubber is to decrease substantially the failure strain and the tensile strength along with an increasing of the Young modulus. In other words, the effect of aging is to make the rubber more rigid and hard, making it less effective in allowing deformation and energy dissipation.

2.4 DYNAMICAL NON-LINEAR BEHAVIOUR IN SEISMIC METAMATERIALS

2.4.1 EXPERIMENTAL TESTS ON NATURAL RUBBER

Considering the fundamental role that disconnections play in the effectiveness of the composite foundation, an experimental laboratory campaign has been planned in order to investigate thoroughly the basic features of what is perhaps the most relevant component of the Metamaterial isolator, namely rubber.

To this purpose an appropriate type of rubber has be selected, *Natural Rubber SH35*, a relatively common typology of rubber, easy to find in the market, already used in construction environment and, above all, soft enough to guarantee a not rigid behaviour for disconnections.

The laboratory test campaign consists of carrying out three kind of tests: tensile test, compression test and tear test. All of them have been performed in accordance to the International Standard (ISO), using proper shaped samples, under controlled conditions. Interpretation of data, as usually, depends upon the interest of the users; therefore, the parameter sets extracted by the row laboratory data can be used for further calculations.

The scope of the tensile test is the determination of the tensile stress-strain properties of vulcanized and thermoplastic rubbers: tensile strength T_s, elongation at break E_b, stress at a given elongation, elongation at a given stress, stress at a yield and elongation at yield. The referring standard for it is the *British Standard BS ISO 37:2005* [55].

Under such code, standard test pieces of rubber, dumb-bells shaped or rings shaped, are stretched in a tensile-testing machine at a constant rate of traverse of the driven grip or pulley. *Figure 52* shows the latter sequence by the factual test carried out.

The samples used for the test are dumb-bell shaped, *type 1*, as prescribed by the standard itself. Readings of force and elongation are taken, as required, during the uninterrupted stretching of the test piece and when it breaks.

The test is accepted as valid when the crack occurs in the central part of the piece for dumb-bells shaped samples. A minimum of *3 test pieces* shall be tested, in the case under study *5 pieces* were utilized.

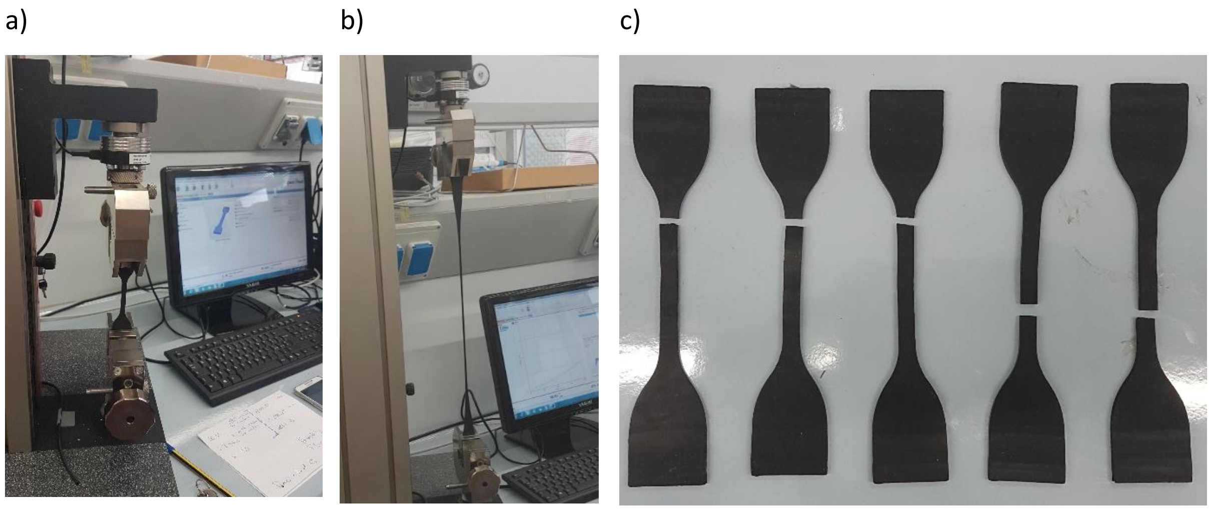


Figure 52 – Tensile test execution: (a) initial phase, (b) elongation phase, (c) crack of the 5 test pieces

Apart from the parameters indicated earlier, elastic moduli can be measured from the test results comparing stress and deformation in the pieces at different levels of charge. Many relevant kind of elastic moduli have been computed in order to better describe the experimental behaviour of the material under test:

- $E_{0\text{-}100}$, elastic moduli measured through linear regression from the initial values to the break;
- $E_{0\text{-}50}$, elastic moduli measured through linear regression from the initial values to the point of the *50%* of the charge for the break;
- $E_{50\text{-}100}$, elastic moduli measured through linear regression from the point of the *50%* of the charge for the break to the values of the break;
- E_{x3} , E_{x2} , E_x , elastic moduli measured through third-order polynomial regression from the initial values to the break.

Figure 53 shows the results of such calculation on the first specimen of the set:

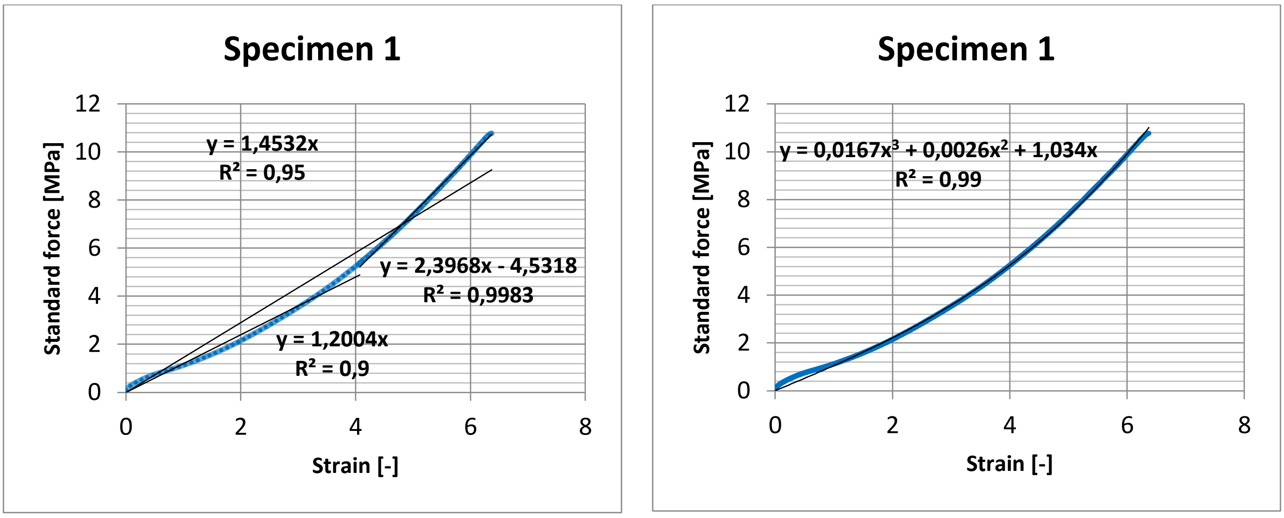


Figure 53 – Result of the tensile test for Specimen 1, calculation of elastic moduli: on the left, E_{0-100}, E_{0-50}, E_{50-100} through linear regression, on the right, E_{x3}, E_{x2}, E_x through third-order pol. regression

Table 7 summarizes the results for all the specimens including the final calculation of the parameters obtaining by averaging the experimental values:

	Specimen 1	**Specimen 2**	**Specimen 3**	**Specimen 4**	**Specimen 5**	**FINAL AVERAGE VALUES**
Ts [MPa]	10,7735	9,0499	10,12089	10,5497	10,7282	**10,2444**
E_b [%]	636,94	559,83	600,04	596,43	581,65	**594,98**
E_{0-100} [MPa]	1,4532	1,3572	1,4194	1,4708	1,5346	**1,4470**
E_{0-50} [MPa]	1,2	1,1418	1,194	1,2305	1,29	**1,2112**
E_{50-100} [MPa]	2,3968	2,353	2,4677	2,6145	2,685	**2,5034**
E_{x3} [MPa]	0,0167	0,0354	0,0311	0,0333	0,0372	**0,03074**
E_{x2} [MPa]	0,0026	-0,1171	-0,1043	-0,1072	-0,1257	**-0,09034**
E_x [MPa]	1,034	1,1826	1,2168	1,2399	1,3277	**1,2002**

Table 7 – Summarize of the tensile parameters for each specimen and calculation of the final results

The scope of the compression test is to measure the ability of rubber to retain their elastic properties at specified temperatures after prolonged compression at constant strain under one of three specified sets of conditions: *25 %* of compression strain (depending on the hardness degree of the rubber) in the case under study. The referring standard for it is the International Standard *DIN ISO 815:1991* [56].

Under such code, standard test pieces of rubber, cylindrically shaped, undergoes to a prolonged compression at constant strain of *25 %*. The strain is imposed for a specified time (*24 hours*) before releasing the strain in the sample. In this phase, two measurements are supposed to be taken, the first after *30 seconds* (h_{sec} in *mm*) of the release and the second after *30 minutes* (h_{min} in *mm*) of the release. *Figure 54* shows the latter sequence by the factual test conducted.

At the end of the test, a check of the internal conditions of the piece are supposed to be carried out, cutting each specimen in the central part of it and controlling the texture and the possible existence of bubble or voids.

For the purpose of the examination, only the capability to retain the imposed deformation has been calculated, then deformations after *30 seconds* (ε_{sec} in *%*) and after *30 minutes* (ε_{min} in *%*) from the release, by subtracting the two related measurements of the samples with their own initial heights (h_{ini} in *mm*).

The samples used for the test are *type A* size, as prescribed by the standard itself. A minimum of *3 test pieces* shall be tested, in the case under study *5 pieces* were utilized.

a) b) c)

Figure 54 – Compression test execution: (a) specimen under constant strain of 25 %, (b) release of strain, (c) manual measurement of the specimen after the release

Table 8 summarizes the results for all the specimens including the final calculation of the parameters obtaining by averaging the experimental values:

	Specimen 1	Specimen 2	Specimen 3	Specimen 4	Specimen 5	FINAL AVERAGE VALUES
h_{ini} [mm]	12,16	12,16	12,16	12,16	12,16	**-**
h_{sec} [mm]	12,10	12,11	12,10	12,11	12,11	**-**
E_{sec} [%]	6,0	5,1	5,5	4,5	4,9	**5,2**
h_{min} [mm]	12,16	12,16	12, 16	12, 16	12, 16	**-**
ε_{min} [%]	0,0	0,0	0,0	0,0	0,0	**0,0**

Table 8 – Summarize of the test parameters for each specimen and calculation of the final results

The scope of the tear test is to determine the tear strength of vulcanised or thermoplastic rubber. The test consist in measuring the force required to tear a specified test piece. The tearing force is applied by means of a tensile testing machine, operated without interruption at a constant rate of traverse, until the test piece breaks. The referring standard for it is the International Standard *DIN ISO 34-1:2010(E)*. [57]

Under such code, there are three possible methods to operate on the basis of the shape of the specimen. In the case under study, trouser test pieces were used then *Method A* was the corresponding one.

The force is applied at the two shreds of the specimen in order to create a concentration of stress in the cut or nick of it, as shown in *figure 55*. The tear strength *Ts* expressed in *KN/m* is given by the ratio between the force *F* in *N* and the median thickness *d* in mm of the test piece.

Depending upon the method employed, the force used to calculate the tear strength can be the maximum one or the median one (as in the case of study).

A minimum of *5 test pieces* shall be tested, in the case under study *6 pieces* were utilized, but one of them (*Figure 55 c)*) manifested a failure crack perpendicular to the cut, returning a too short test essay.

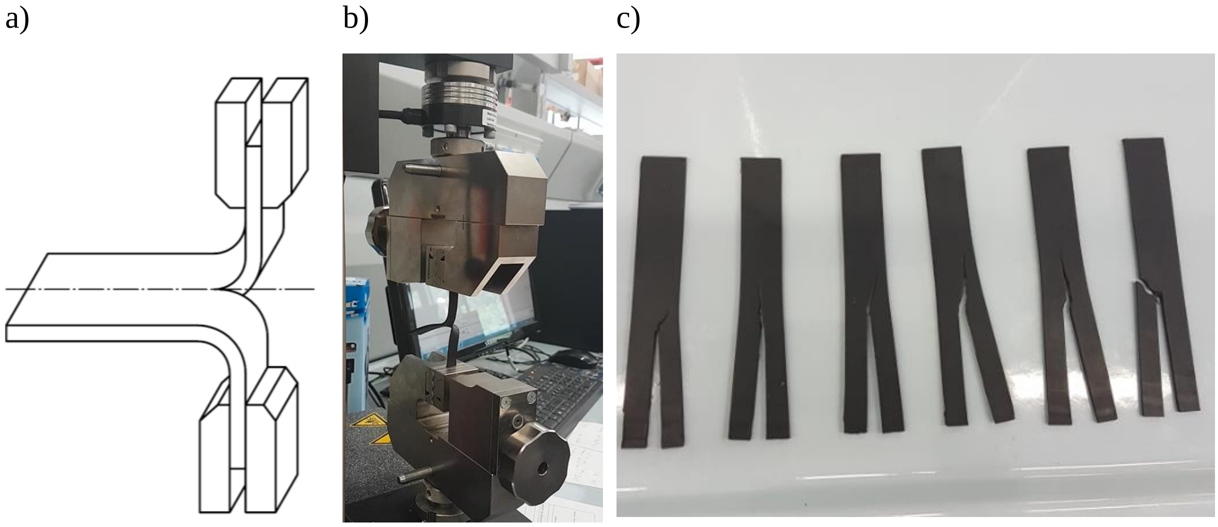
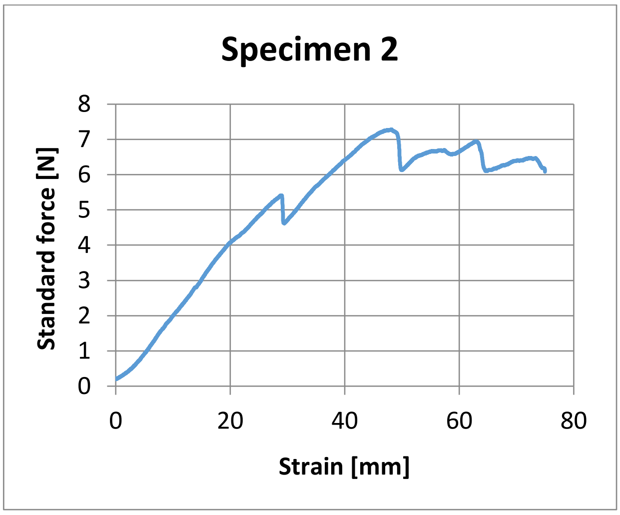


Figure 55 – Tear test execution: (a) schematic of the test, (b) practical test execution, (c) crack of every test pieces, 5 validated and 1 (at the extreme right) non-compliant

Figure 56 shows the full load path of the test of for two specimens validated (number *2* and *5*):



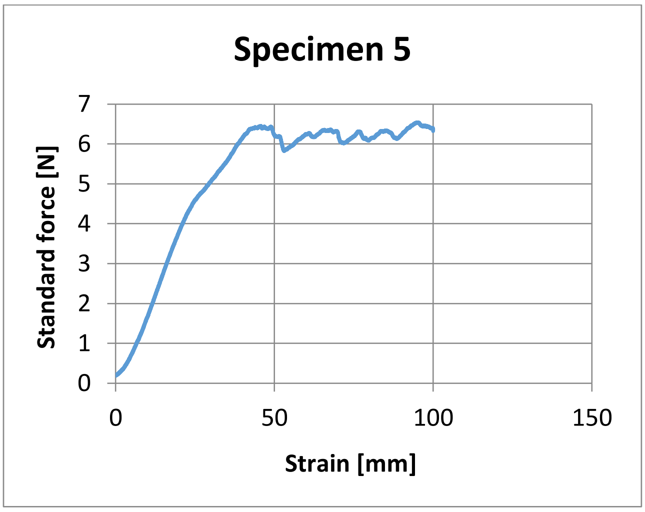


Figure 56 – Full load path for the tear test of specimen 2 and specimen 5

Table 9 summarizes the results for every specimen, including the final calculation of the parameters obtaining by averaging the experimental values:

	Specimen 2	Specimen 3	Specimen 4	Specimen 5	Specimen 6	FINAL AVERAGE VALUES
Ts [KN/m]	3,469	1,774	1,633	0,114	2,232	**1,844**
F_{med} [N]	6,939	3,548	3,266	0,228	4,464	**-**
F_{max} [N]	7,131	6,869	6,308	6,542	8,705	**-**
d [mm]	2	2	2	2	2	**-**

Table 9 – Summarize of the tear parameters for each specimen and calculation of the final results

2.4.2 METAMATERIALS WITH NON-LINEAR DISCONNECTIONS

The latest developments on Metamaterials led to the investigation of the dynamical non-linear behaviour of this kind of technology. To this regard, the group of researchers of the University of Messina conceived an analytical theory based on anharmonic approach, focusing on a seismic Metamaterial that consists of a one-dimensional periodic mass-in-mass system.

The idea of introducing the non-linearity in the mass-in-mass system via an anharmonic expansion of the potential energy came from some previous reference works, the most important of which are the works of Fermi et al. [58], Maradudin & Fein [59], Lowell et al. [60] and Tsurui et al. [61].

The aim of this research is to deepen the knowledge about non-linear effects in mass-in-mass Metamaterials in order to implement such expertise in the realization, development and improvement of Metamaterial isolator, like as an example the composite foundation.

The analytical model for the dynamics of a non-linear mass-in-mass chain takes a cue on the anharmonic description according to the Taylor expansion up to the fourth-order power of the potential energy and the kinetic energy.

The dynamics of the model is studied by coupling a non-linear oscillator and a linear oscillator moving together to represent the global behaviour of the system. *Figure 57* displays the basic schematic:

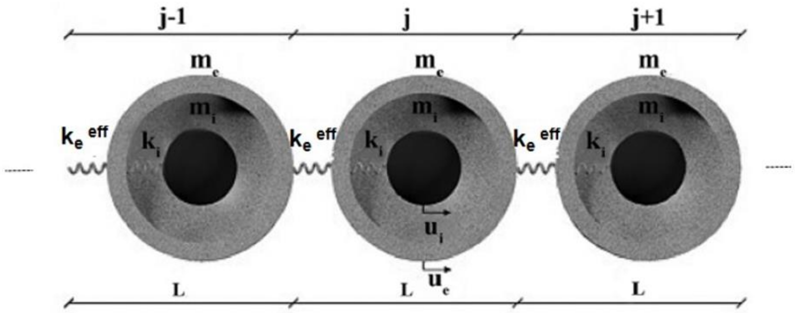


Figure 57 – Schematic for the non-linear periodic mass-in-mass chain

This is very similar to *figure 47*, unless for the introduction of k_e^{eff}, namely the effective spring constant deriving from the global non-linear behaviour given by the two different components, the harmonic one through the constant $k_e^{(2)}$ and the anharmonic one through the constant $k_e^{(4)eff}$. To derive the secular equation in the non-linear case, the following approximations are assumed:

- trial solution shaped as a periodic wave having a form resulting from the overlapping of two counter-propagating plane waves, either a cosine or a sine, as per Lowell et al. [60];
- considering the average solutions over the whole space-time period. As a result, the third-order term, linearly proportional to the cosine (sine) function, is nullified;
- considering the wave amplitude of the external mass as a "parameter".

According to such approximations, the secular equation for the anharmonic case is *equation 2.5*:

$$\omega^4 - \left(\frac{k_i}{m_i} + \frac{k_e^{(2)}}{m_e}\left(\frac{k_i}{k_e^{(2)}} + 2(1-\cos\kappa L) + \frac{k_e^{(4)\,\mathrm{eff}}}{k_e^{(2)}}\left(\frac{1}{6}\left(1-\cos 3\kappa L\right) + \frac{1}{2}\left(\cos 2\kappa L - \cos\kappa L\right)\right)\right)\right)\omega^2 + \frac{k_e^{(2)}}{m_e}\frac{k_i}{m_i}\left(2(1-\cos\kappa L) + \frac{k_e^{(4)\,\mathrm{eff}}}{k_e^{(2)}}\left(\frac{1}{6}\left(1-\cos 3\kappa L\right) + \frac{1}{2}\left(\cos 2\kappa L - \cos\kappa L\right)\right)\right) = 0. \qquad \text{Eq. 2.5}$$

Of which the dispersion law, in compact form is *equation 2.6*:

$$\omega = \frac{1}{\sqrt{2}}\left(A \pm \left(A^2 - 4B \right)^{\frac{1}{2}} \right)^{\frac{1}{2}} \qquad \text{Eq. 2.6}$$

Where parameters *A* and *B* are expressed as per *equation 2.7*:

$$A = \frac{k_i}{m_i} + \frac{k_e^{(2)}}{m_e}\left(\frac{k_i}{k_e^{(2)}} + 2(1-\cos\kappa L) + \frac{1}{2}\frac{k_e^{(4)\,\mathrm{eff}}}{k_e^{(2)}}\left(\frac{1}{3}(1-\cos 3\kappa L) + (\cos 2\kappa L - \cos\kappa L) \right) \right)$$

$$B = \frac{k_e^{(2)}}{m_e}\frac{k_i}{m_i}\left(2(1-\cos\kappa L) + \frac{1}{2}\frac{k_e^{(4)\,\mathrm{eff}}}{k_e^{(2)}}\left(\frac{1}{3}(1-\cos 3\kappa L) + (\cos 2\kappa L - \cos\kappa L) \right) \right) \qquad \text{Eq. 2.7}$$

Subsequently, with the intent to evaluate the effect of non-linearity in a more realistic system, studies by means of numerical calculations have been performed.

Input parameters used come from the composite foundation characteristics, except for the external spring expressing non-linearity. For such parameters, the result of a compression test (the main working condition of this element in a seismic Metamaterial) in samples of soft Natural Rubber have been selected. The parameters for the non-linearity, $k_e^{(2)}$ and $\bar{k}_e^{(4)\mathrm{eff}}$, have been extracted from the numerical interpolation of the resulting constitutive law.

Figure 58 displays a table listing the input parameters together with a comparison between the experimental behavior of the rubber and the numerical interpolation of the restoring force, according to the following third-order polynomial equation (*Equation 2.8*):

$$F = k_e^{(2)} x + \bar{k}_e^{(4)\mathrm{eff}} x^3 \qquad \text{Eq. 2.8}$$

Where *x* is the generic displacement. The experimental results confirmed a strongly non-linear behavior (hardening) as marked as the displacements increase. The fitting parameters for $k_e^{(2)}$ and $\bar{k}_e^{(4)\mathrm{eff}}$ are shown in bold in the table of *figure 58*:

m_e	317 kg
m_i	245 kg
$k_e^{(2)}$	**1,14 x 10⁵ N/m**
$\bar{k}_e^{(4)\mathrm{eff}}$	**2 x 10⁸ N/m³**
k_i	1080 x 10³ N/m
L	1 m

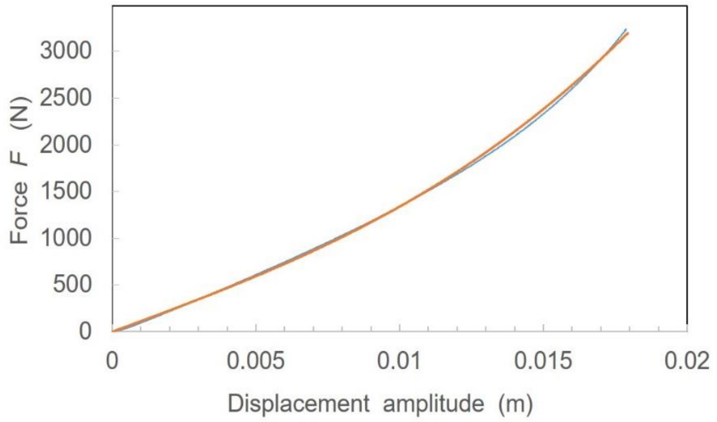


Figure 58 – Input parameters used for the numerical simulations of the non-linear behaviour of mass-in-mass chain, included of the fitted parameters (in bold) accounting for non-linearity. Comparison between the experimental behavior of the rubber (green line) and the numerical interpolation (red line)

Figure 59 portrays the results of numerical calculations of the dispersion laws for different values of external mass displacement *Re[Uₑ]* along with the corresponding fourth-order anharmonic constants $\bar{k}_e^{(4)\mathrm{eff}}$. The main remark is that the more the value of anharmonicity increases, the more the acoustic branch takes a curved shape.

Such trend is in favor of an antiseismic effect since the band gap is pushed down towards low frequencies because of the occurrence of an instability region due to the destructive interferences between incoming and reflected waves, during their propagation across the Metamaterial.

To better understand this point, *figure 60* shows the corresponding group velocity for all these cases. Here, as the non-linearity increases, the group velocity starts exhibiting a direction that deviates remarkably by its usual monotonic trend. This determines the onset (at *Re[Ue] = 0,02 m*) of the instability caused by the non-linearity that evolves more and more intensely.

All in all, it is possible to understand that the effect of anharmonicity is analogous to the effect of damping in a Metamaterial device. This behaviour leads, as a result, to the presence of a cut-off value for the external mass displacement in the case of non-linear behavior in contrast to the limitless amplitude reachable in the harmonic case. Such damping-like effect involves only the acoustic branch of the dispersion law without implying any change in the optic branch.

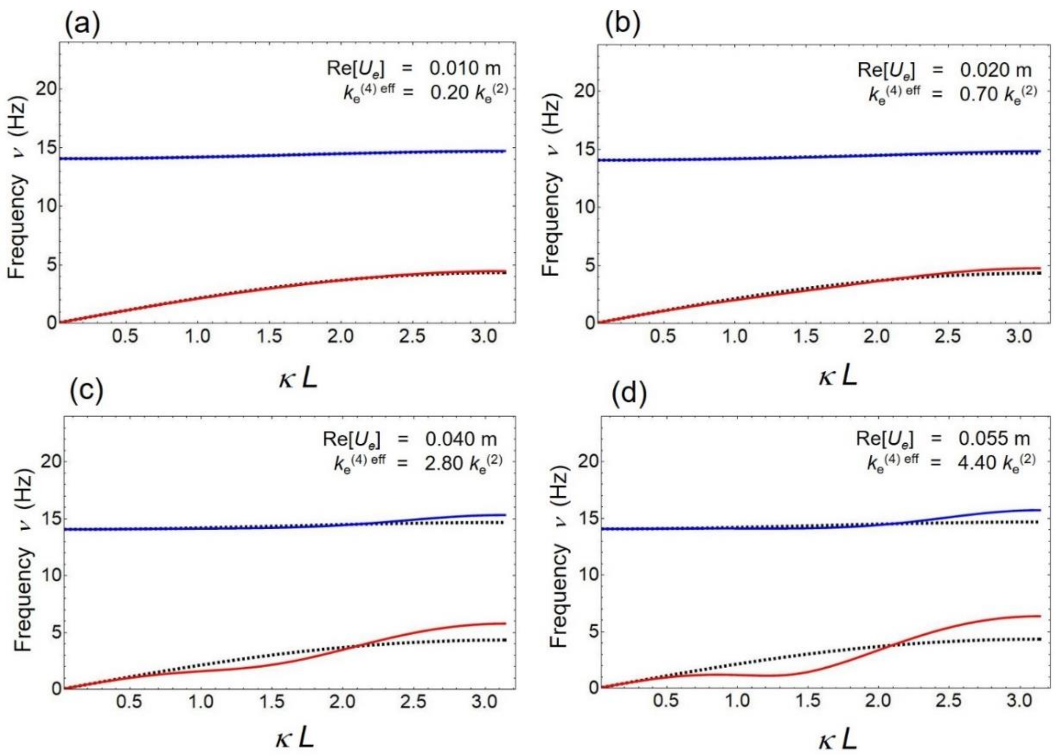


Figure 59 – Non-linear mass-in-mass dispersion laws calculated for different values of external mass displacement Re[Ue] and the corresponding fourth-order anharmonic constants $\bar{k}_e^{(4)\mathrm{eff}}$

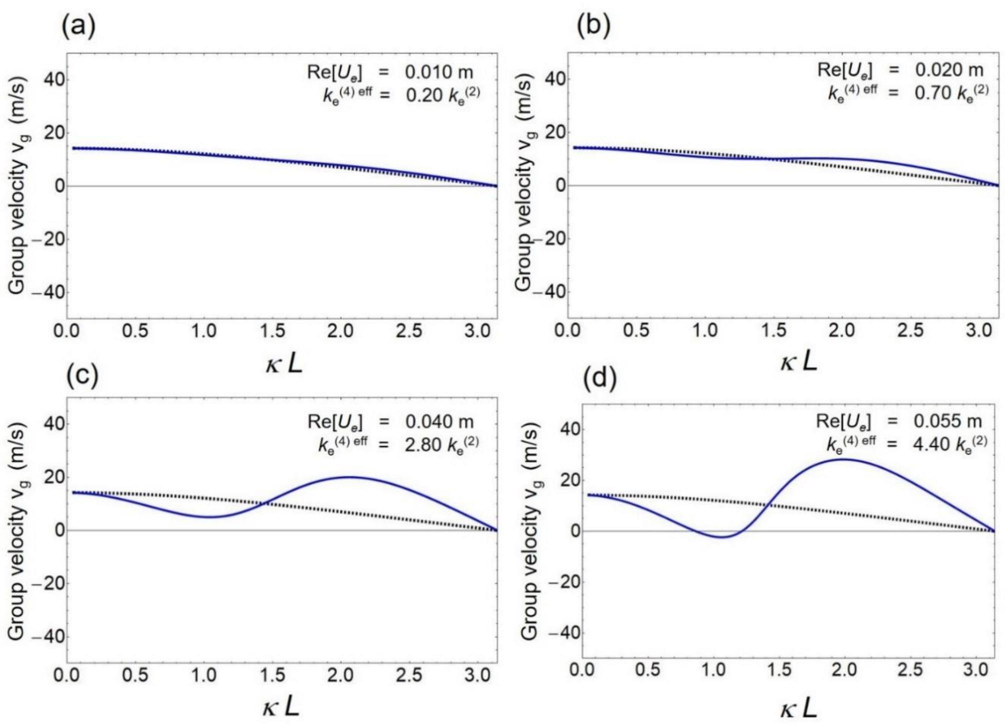


Figure 60 – Corresponding group velocity graphs to the cases in Figure 59

Conclusions

This thesis is the outcome of a three years study focused on the understanding and development of anti-seismic systems based on the concept of Metamaterial.

Metamaterials are a type of composite material with unique properties that can have vast application in many sectors. The aim of this work was to explore Metamaterial applications in earthquake engineering, pointing out their own qualities and their own flaws, if any.

With this aim, leading concepts of earthquake engineering design were presented in *Chapter 1*. Specifically, in *Paragraph 1.1* an overview of the historical attempts in this filed was introduced to show the way how science tried to face this issue over the time and how earthquake engineering concepts have been evolving to nowadays.

In *Paragraph 1.2*, the different approaches used in the last decades were discussed in order to call attention to divergences and benefits of using different anti-seismic techniques. Among these, base isolation technique represents the last achievement, validated by a relatively recent insertion in the regulatory framework for design and construction that opened the way to its large-scale dissemination. All the relevant aspects of such technique were covered, focusing on the requirements that standard and technical codes prescribe for its application.

In *Paragraph 1.3*, a meticulous analysis of practicability and efficiency for the base isolation technique was carried out in order to emphasize the cases where implementing such technique can be beneficial or conversely not convenient.

In *Paragraph 1.4*, the design phases for determining seismic actions and conditions of ground foundation were discussed, strictly adhering with the advices of the rules in force. In this context, the subject of ground response analysis was examined in the details in *Paragraph 1.5*, defining all the relevant parameters that drive to the evaluation of the transfer functions for site foundation deposits, in simplified cases and in situations that are more realistic.

This part represents the essential basis to figure out the most appropriate actions to be taken with the aim of mitigating earthquake effects. In this regard, Metamaterials represent a new frontier with enormous potential for future developments. Indeed, isolation systems based on the concept of Metamaterial have increasingly been proposing and, in the near future, they can constitute a serious step forward for the conception of protection systems in earthquake engineering.

Different proposals of isolation system based on the concept of Metamaterial were discussed along all *Chapter 2.* After a first overview of the most interesting proposals in *Paragraph 2.1*, two typologies of them were categorized and developed: systems connected with the ground and systems connected with structures. For both of them, working principles were described and remarks were expressed on the basis of all experiences collected, like field investigations, experimental tests, comparison with other systems, discussions with specialists in the field, ext.

About systems connected with the ground, an extensive investigation on real ground foundation examples were carried out, using principally geophysical methods like seismic refraction and electrical resistivity surveys. The more significant results were displayed in *Paragraph 2.2*.

The analyses conducted revealed the possible criticalities for the realization of this type of isolation system. In fact, catching accurately deposit natural frequencies is still complicated and uncertainties can bring to mismatches that would make the isolation system ineffective.

The best solution to manage this point is trying to enlarge as much as possible the frequency range of attenuation of Metamaterial systems in order to make them effective, even in case of small errors in the assessments of the deposit natural frequencies. This is because heterogeneities, lateral variation of parameters rather than complex geometries or subsurface structures are so common in practice, as proven by the vast corpus of case histories collected during the investigation phase.

A solution that, in general, can be proposed to make more effective and reliable Metamaterials connected to the ground is providing an increase of soil mechanical parameters through the process of installation of the Metamaterial in the ground. Indeed, the presence of an artificial material in the ground could introduce a functional level of strain, density and over-consolidation, limiting both heterogeneities in the foundation deposit and variation of the soil mechanical parameters due to the inelastic behaviour possibly activated in occurrence of medium or strong earthquakes.

About systems connected structures, the proposal called "*composite foundation*" was examined in *Paragraph 2.3*. This consists of a special configuration of seismic isolator created directly inside standard building foundations and integrating the physics of periodic mass-in-mass systems. The capabilities of such a system were described based on the results coming from a first experimentation phase held on a prototype of composite foundation.

Its operational mechanism differs with the ones of more classic isolator devices because composite foundation does not provide the building-isolator system of deformability but instead it delivers a beneficial dissipating capability. In this context, disconnections play a crucial role because they activate the relative motion among the Metamaterial foundation components and, consequently, the internal counter-phase movement that breaks down seismic energy.

The latest developments in this technique led to the investigation of the dynamical non-linear behaviour of disconnections. This was covered in *Paragraph 2.4*. An experimental laboratory test session was carried out in order to investigate deeply the basic features of disconnections main component, namely rubber. According to the laboratory results, an analytical theory based on anharmonic approach was conceived, considering a Metamaterial model that consists of a one-dimensional periodic mass-in-mass system exploiting rubber non-linear behaviour.

The numerical outcomes on this research shown a trend in favour of an antiseismic effect, since the new Metamaterial band gap is pushed down towards low frequencies thanks to the presence of an instability region due to wave interferences. At the end of this examination, it was emphasised that the effect of anharmonicity is analogous to the effect of damping.

Ultimately, final considerations are now expressed about the different typologies of systems herein pictured, in the light of the fact that currently the challenge in seismic Metamaterial environment is to find the most effective configuration to face earthquake effects.

Following all the deliberations expressed above, in the opinion of the author of this thesis, the most convenient configuration to exploit Metamaterial capabilities in antiseismic engineering is through isolator devices, tuned directly to the building vibration modes.

Basically, the reason that brings to this conclusion is that material behaviour and techniques for building construction are clearly known for ages, thus getting to know of the vibration modes of the buildings is easier and the risk to incur in mistakes in defining them is lower. Consequently, tuning Metamaterial devices to the building frequencies results more certain.

However, the idea of protecting larger portion of territory through the use of Metamaterials connected with ground is surely valuable and convenient and it would provide advantageous progress when this technique will be set up.

Of course, both these branches of research are still in progress and taking full advantage of their potentialities requires more time. Nevertheless, this area of research is very promising and it can lead to brilliant improvements in earthquake engineering.

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