Earthquake Simulator Testing of a Steel Model Seismically Protected with Friction Energy Dissipators

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INTERNATIONAL CENTER FOR NUMERICAL METHODS IN ENGINEERING
Edificio C1, Campus Norte UPC
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The authors are also grateful to the Laboratory of Mechanical Engineering for building the steel model and the friction dissipators utilized in the tests.

SUMMARY

In this research the study of an experimental reduced scale steel model of a medium-rise building structure is presented. The model was dynamically protected with energy dissipators installed on rigidization diagonals. These devices are passive control elements able to reduce the earthquake forces in buildings. The energy dissipators utilized in the experimental tests principally involve the steel-steel friction to dissipate energy and to lower the seismic response. When the structure undergoes interstory drifts the devices dissipate energy increasing the damping of the structure.

A series of tests have been performed on a shaking table in the Applied Geophisics Laboratory of the Technical University of Catalonia, Barcelona, Spain. The model is a 3-D steel moment resisting frame with 5 stories. It is 1.70 m high, 1.44 m wide and 0.77 m deep. Along one direction there are two bays and in the other direction there is one.

The experimental program consisted of identification tests (to determine the natural periods and the dynamic characteristics of the model) and earthquake simulation tests (to study the dynamic behaviour of the model with and without the dissipators). A numerical analysis has been performed to extrapolate some results also for other kinds of inputs.

Results show the efficacy of the protection system.

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CHAPTER 1

Introduction

1.1 GENERAL

In engineering and architecture, structures must resist many loads and principally dynamic loads such as seismic motion, wind gusts, vehicle motion, machine vibration, explosions, impacts, and others depending on the specific functionality of the structure. Nowadays in the actual design of structures the dynamic problems are always more important: lighter and thinner structures (due to the use of more resistant materials, bolder design procedures, new requirements to take into consideration and more exact and powerful analysis methods) and a more intense social awareness of safety and comfort of structures. Therefore it is evident that it is necessary to study structures able to resist dynamic excitation.

In the case of excitation produced by earthquakes, over time, society has recognised the importance of restoring regular life in a community after a serious earthquake, that involves a marked risk of distress or destruction. A better approach is to prevent or mitigate damage in such an event so that social activity is not disrupted. To this end, it is very important to know the effect of the earthquakes on structures, the damage that they produce and how the structures recover their characteristics of resistance and capacity of deformation after the seismic event occurs.

After a seismic event the damages in a structure could be of two types: structural and non-structural. The non-structural damage, such as cracked masonry walls and fallen mortar, can be rapidly restored even if sometimes at very high cost. Structural damage belongs to a more important category than the previous one. In this case it is necessary to rehabilitate or to reinforce the structure taking into account various aspects, and principally to decide whether it can be repaired or it is condemned to be pulled down.

To increment both the resistance and the capacity of deformation and energy dissipation of reinforced concrete and steel structures, the concept and the use of passive control systems has been developed. Most demands of energy dissipation and of deformation are concentrated in these devices, avoiding the formation of fall mechanisms not desired and not considered in the design.

1.2 CONCEPT OF STRUCTURAL CONTROL

The control of structures was born as a necessity to reduce the effects of vibrations on structures and on structural parts of them. From a broader perspective, control of structures not only refers to civil structures but also to mechanical systems and so on.

Traditionally, seismic motions were resisted by a correct design of the structure, with the aim of reducing the absolute accelerations (for equipment safety and human comfort purposes) and the relative displacements between the construction and the ground (for structure safety purposes). Generally, the horizontal components of the ground and the structure motions are more harmful than the vertical ones. Traditional seismic design has several limitations: usually heavy structures are obtained, response could not be lessened beyond some value, it is not possible to reduce relative displacements and absolute accelerations at the same time, for strong inputs the structure itself is damaged. To overcome all these difficulties, control systems have been proposed; they can be classified in: passive, active, semi-active and hybrid.

Passive systems.

They are the most utilised in practice because a lot of theoretical and experimental studies have been carried out.

They consist of some «inert» mechanical devices (here "inert" does not mean inactive but that they are not powered and that their behaviour can not be modified on-line) which are connected to the structure to dissipate and/or to deflect energy.

In general passive systems are not able to adapt to the unexpected characteristics of the excitation and in some cases their effect might even be damaging (for instance, as the natural frequencies of the structure are shifted new resonance can occur). However, if the main features of input are known, passive systems are extremely efficient.

Active systems.

These systems behave similarly to passive ones but, instead of inert devices, there are highly powered mechanisms (actuators) that are able to push the structure to counteract the input effect. Hydraulic cylinders driven by servo-valves are examples of actuators. An active control system is composed of: a set of sensors to measure on-line the response of the structure (mostly displacements and accelerations), the above mentioned actuators, a source or reservoir of energy to power the actuators and a controller (typically a computer) that decides the amounts of forces to be exerted by the actuators. These systems operate automatically as a closed loop where the measured response (by the sensors) is used by the controller to calculate (following some strategy -control algorithm-) the forces to be applied (by the actuators) to reduce the vibrations.

The feasibility and reliability of active systems are controversial because: (i) to accelerate the massive civil engineering constructions large forces are required (consequently, a huge energy source is needed) and (ii) energy supply can be interrupted during a strong input (earthquake, wind, etc.). Therefore, active systems might be more appropriate for reducing the response under minor (or even, frequent) earthquakes. In building structures the feasibility of active control is higher for wind input than for strong seismic excitations because the required control forces are smaller (roughly hundreds of tons compared to thousands).

Semi-active systems.

Their operation is similar to that of active systems but the actuators are only able to restrain the structure instead of having also the capacity of pushing it (in other words, they can take energy from the system but can not exert energy on it). Consequently, these devices are much smaller (for example hydraulic cylinders with an on/off valve) than those required for active systems, and only a minor amount of energy is required to operate them (typically about 25 W, so some batteries can supply it). Obviously, these systems are more feasible and reliable than the active ones since they do not depend on an external source of energy. It can be said that the performance is better than in the passive case and only slightly worse than in the active one.

Hybrid systems.

These systems consist of a combination (series of parallel) of active (or semi-active) and passive systems. The efficiency of such co-operation lies in the fact that the passive system can provide the gross reduction of response (by absorbing or deflecting energy) while the active one is used for further lowering (for protection of sensitive equipment, for example) of displacements or accelerations.

As no big control forces are required, hybrid systems are more feasible and reliable than active ones.

It is remarkable that none of these devices (active, semi-active or passive) are part of the main structure, so they can be temporarily removed for inspection, replacement or repair. As described next, the case of base isolation is slightly different.

A comparison among the four categories described in this section shows that the passive ones are more feasible and reliable since devices are simpler and do not depend on any source of energy. Though they are not as incredibly efficient as the other systems, if properly designed, they can provide excellent results in a wide range of situations. Passive control is clearly more spread than the other systems. This paper presents an overall picture of passive devices proposed to date .

Figure 1.1 shows a classification of the most common systems to control the vibration of structures. The list is not exhaustive because this field is still under research and new mechanisms and devices are being introduced.

Passive control systems could be defined as «inert» mechanisms added to the structure to improve its behaviour in response to the dynamic forces associated with wind and especially earthquakes. The behaviour of these systems is based on deforming inelastically in response to the excitation.

Passive systems are much cheaper and simpler than the active ones and have experimented a huge growth in the last years. This reasearch focuses on Energy Dissipating Devices (EDDs) which are a kind of passive control system for civil engineering structures.

It is possible to classify the mechanisms of passive control of the seismic response in four classes:

1) Mechanisms for the seismic isolation (base isolation) of vibrations. Generally they are put on the base of the structure, with the aim of reducing the seismic forces entering the structure [Jolivet, et al., 1977]. The structure is partially uncoupled from the foundation by using flexible bearings instead of traditional (rigid) connections. Vibration to be controlled can be transmitted from the ground to the structure or, conversely, from the structure to the ground (vibrating elements isolation). (see Figure 1.2).

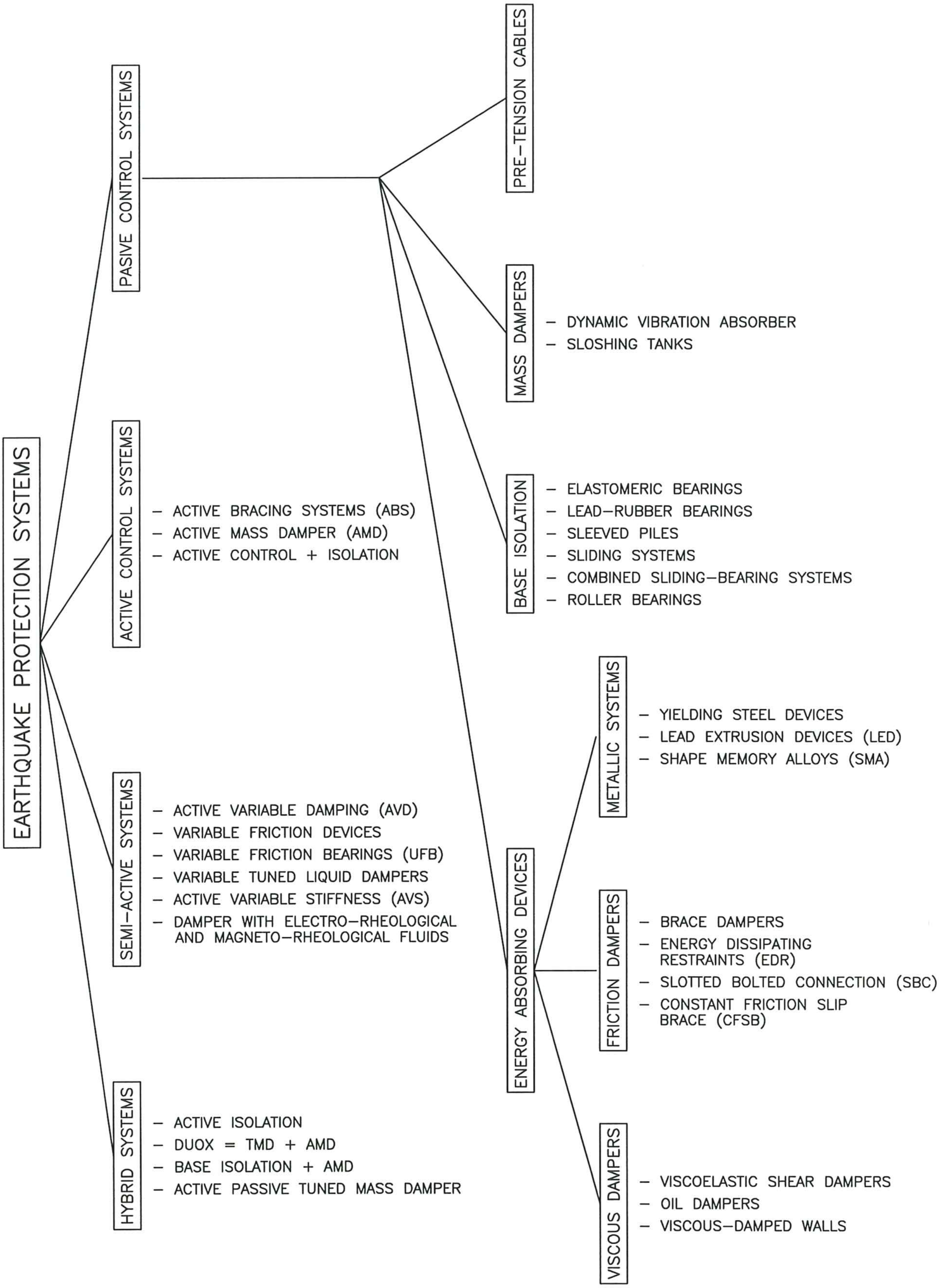

Figure 1.1. Earthquake protection systems.

2) Energy dissipation systems. Energy is dissipated by external devices fixed to the structure. Input: earthquakes, wind, etc. They are installed in the structure with the aim of dissipating most seismic energy entering the same structure [Aiken, et al., 1993, Bergman, et al., 1993, Tsai, et al., 1993].
3) Mass dampers. They consist in one or more masses added to the structure, generally at the top floor. These masses are made with such dynamic properties that they reduce the response of the structure. Energy is transformed into kinetic energy (translation or rotation) in massive devices (possessing big mass or big moment of inertia).
4) Pre-tension cables. These cables stiffen the structure and increase the axial load in the columns reducing, in some cases, the rotation of their ends.

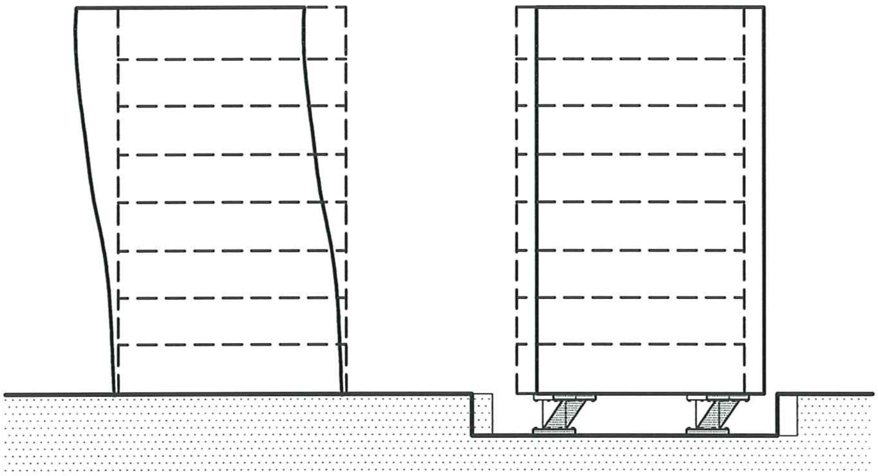

Figure 1.2. (a) Fixed base building; (b) base isolated building.

The requirements that a mechanism of passive control of the seismic response should guarantee are listed below:
a) After the seismic event, the structure must return to its original position (high deformations must not be permitted).
b) The daily life of the people living in the structure must not be altered during the checking and restoration phases.
c) Utilising mechanisms of this type the initial cost of construction and of the structural reinforcing of the existing building should be reduced.
d) The mechanical properties of the devices must not vary substantially with time.
e) Maintenance and inspection must not be required except after the occurrence of a severe earthquake; in this case the operations requested should be simple.

To accomplish all these requirements, a complicated device is not useful, while a simple, economic device with stable behaviour under seismic action is preferable.

The traditional seismic design takes into account that some structural elements enter the inelastic range of behaviour, and the hysteretic energy involved will contribute to reducing the value of the demand of responses during a destructive earthquake [Ohno, et al., 1992]. In fact the conventional design of seismic-resistant structures is based on the concept of ductility and structural redundancy. The forces induced by a severe earthquake are significantly reduced as a function of both concepts, connected with the energy dissipation capacity of the structural elements [Bozzo and Barbat, 1995b]. A seismic resistant rational design guarantees that for a certain global structural ductility demand the ductility capacity of the elements is not exceeded. Due to uncertainties with non-linear analysis is difficult to precisely estimate the local ductility demands in each section of a structure. Moreover, the traditional building design offers few guarantees of avoiding damage to the non-structural elements during a severe earthquake, so even the restoration of the principal elements could be difficult. Therefore in the last twenty years several energy dissipation and base isolation systems have been proposed to localise non-linear behaviour in certain pre-defined areas of a structure.

Passive systems have experimented a huge growth in the last years and hundreds of structures incorporate such protection devices. Outside Europe such systems have been mostly used in Japan, New Zealand and U.S.A.; in the European Union passive systems have been mostly used in Italy and in France. Presently passive control is expanding everywhere and constitutes a usual design alternative, it is remarkable that base isolation has been incorporated to the aseismic design code in the U.S.A. and energy dissipation design code in the U.S.A. and energy dissipation will be incorporated soon.

Along with the extended use of passive control systems an important research effort has been done in both public and private research institutions. However, the use of the passive devices has grown faster than the research and nowadays there is a strong need of investigation in this area. Particularly, there is lack of experimentation (both in reduced and full scale) and of simple, accurate and reliable design and analysis tools.

The dynamic behaviour of passively controlled structures is usually nonlinear but, since the main purpose of the control systems is to keep the response of the structure inside the yielding limits, nonlinearities are concentrated in the control devices. Full nonlinear analysis of all the structure is time consuming and usually expensive, and results are not always reliable since some devices exhibit strongly nonlinear and irregular behaviour which is difficult to deal with. Besides, 3D problems are not easily simulated. Therefore, passive systems are usually designed from simplified linear equivalent models. Techniques for an analysis that considers any local nonlinearities have been developed recently [Inaudi and De La Llera, 1992] for some simple cases. Anyway in the field of passive control there is still a huge need of research and it is very important to have a reliable, accurate and computationally efficient simulation tool since a lot of numerical experiments has to be performed.

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CHAPTER 2

Energy Dissipating Systems

2.1 GENERAL

Traditional techniques for the design of building structures, are based on experimentation and on the damage observed during earthquakes. Structural ductility and redundancy are the basis of modern design criteria, giving the possibility of significantly reducing seismic forces. The result is the design of economic structures which perform satisfactorily during a severe earthquake. However ductility means both damage in the structural and non-structural elements. Furthermore the damage may cause the temporal or total arrest of the building. Therefore, in recent years research is mostly oriented towards finding techniques which reduce seismic forces, without creating damage in the structure, or concentrating it in certain pre-determined points.

In conventional design practice, energy dissipation is intended to occur in some detailed critical regions of the structure, usually in the beams near or adjacent to the beam-column joints. Inelastic behaviour in these regions often results in significant damage to the structural members, and although the regions may be well detailed, their hysteretic behaviour will degrade with repeated inelastic cycling. Further, the large interstory drifts required to achieve significant hysteretic energy dissipation in critical regions usually result in substantial damage to non-structural elements such as in-fill walls, partitions, doorways, and ceilings.

A modern tendency in seismic design is to find structural systems which localise the ductility request in some "weak" points able to dissipate energy in a stable form and which, moreover, may be easy to repair. To this aim, Energy Dissipating Devices have been utilised in antiseismic design; they are external devices connected to the structure in such a way that when it vibrates they deform and dissipate energy; after major earthquakes, such mechanisms can be easily replaced. In this way, no ductility in the main carrying-load system is required since damage (permanent strains) is concentrated in the dissipators. The idea of the hysteretic energy dissipator is based on the same principle, but the structural element which plastically deforms is only the device itself; in this way the other structural elements are kept in the elastic range of behaviour [Asano, et al., 1982].

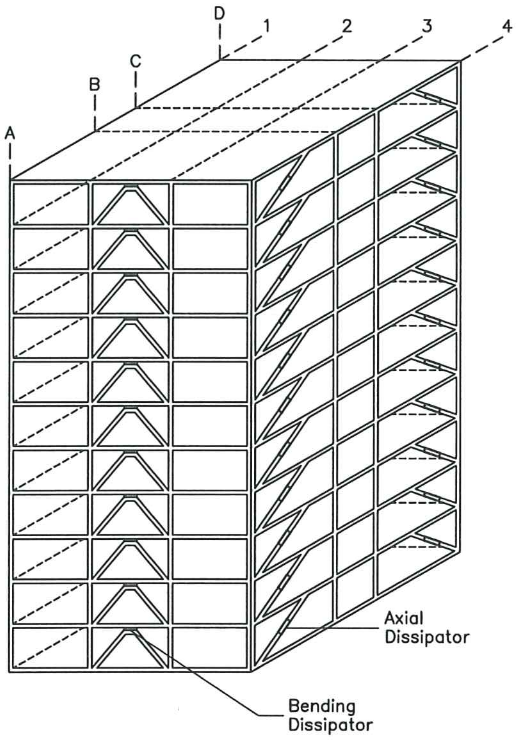

Figure 2.1. Building with energy dissipators.

To this end, in the international context, mechanical devices which utilise plastic deformation of steel to obtain this energy dissipation have been developed [Skinner, et al., 1980, Way, et al., 1986, Jolivet, et al., 1987, Castiglioni, et al., 1982, Ikonomou, 1982, Robinson, 1982, Aiken, et al., 1993, Bergaman, et al., 1993, Tsai, et al., 1993]. Friction represents another source of energy dissipation. However, this would be a difficult source of energy dissipation to quantify. Another disadvantage of the friction devices is that most of them are not capable of auto-centering; after the occurrence of an earthquake a permanent displacement between the two frictioning parts could appear, causing high maintenance costs.

There is a certain tendency to consider that energy dissipators do not reduce the seismic forces so significantly as base isolators. This is the case for some systems and always as a function of their design and number. For example, it requires a large number of viscoelastic (VE) dampers with relatively large dimensions in order to increase significantly the damping in a building [Pong, et al., 1994]. VE devices seem more advantageous for vibration control of wind and environmental. Other systems, such as those based on metal plastification, are relatively simple to construct and it is easy to change their dimensions. In general, the significant reduction of structural response to severe earthquakes utilising energy dissipators depends on their number, position in the building, the kind of dissipator and its design. Conceptually, the seismic design of structures with plastification of steel devices is similar to the conventional design of buildings based on ductility with the additional requirement of the limited number of available devices.

2.2 CLASSIFICATION OF ENERGY DISSIPATION DEVICES

The addition of mechanical damping devices to a building causes an advantageous improvement of the structural response.

In conventional design practice, energy dissipation is intended to occur in specially detailed critical region of the structure, usually in the beams near or adjacent to the beam-column joints. Inelastic behaviour in these regions often results in significant damage to the structural members, and although the regions may be well detailed, their hysteretic behaviour will degrade with repeated inelastic cycling. Further, the large interstory drifts required to achieve significant hysteretic energy dissipation in critical regions usually result in substantial damage to non-structural elements such as in-fill walls, partitions, doorways, and ceilings.

In summary, up to now, the Energy Dissipating Devices which have been developed are based on the following principles:

1) Plastic deformation of metals.
2) Friction.
3) Elastomers with high damping capacity.
4) Devices which act on the base of viscous fluids.

In the following the most common Energy Dissipating Devices, based on the already mentioned mechanisms, will be described in detail.

2.3 DESCRIPTION OF DISSIPATING SYSTEMS

2.3.1 Metallic Systems

These energy dissipating systems take advantage of the hysteretic behaviour of metals when deformed into their post-elastic range. A wide variety of different types of devices have been developed that utilise bending, shear, or extensional deformation modes into their plastic range. An important desirable feature of these systems is their stable behaviour, long-term reliability, and generally good resistance to environmental and temperature factors.

2.3.1.1 Yielding steel systems

These devices utilise mild steel to dissipate energy. Mild plates are modelled with different shapes so that the yielding is spread uniformly throughout the material. The result is a device which is able to sustain repeated inelastic deformations in a stable manner, avoiding stress concentrations and low cycle fatigue.

The Betchel Adding Damping And Stiffness (ADAS) is a known yielding steel system. ADAS elements are an evolution of an earlier use of X-plates, as damping support for piping systems [Stiemer, et al., 1981]. This device is composed of X-shaped metallic plates arranged in parallel, as shown in Figure 2.2(a) and (b). The dimensions of each plate are shown in the figure while its number could change depending on the amount of energy to dissipate. Plastification is produced at the same instant in each plate and their shape follows the fixed bending diagram. Optimisation of the global structural response can be achieved by varying the number of plates used in each device.

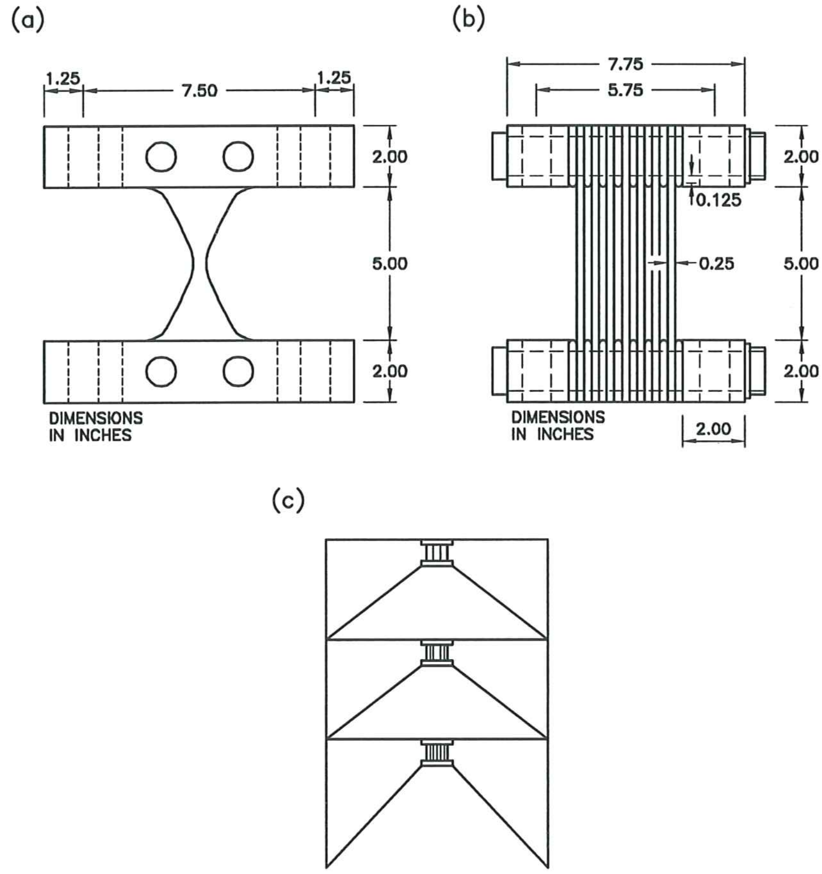

Figure 2.2. ADAS system. (a) Dimensions. (b) Typical connection with parallel plates. (c) Conventional localisation of the connection in a frame.

A convenient location of these connections is in the diagonal-beam joint (Figure 2.2(c)). In this case the bracing system must be substantially stiffer than the surrounding structure. The introduction of such a heavy bracing system into a structure may be prohibitive otherwise the system is not efficient [ ]. Alternatively they can be placed in coupled walls in reinforced concrete, as shown in Figure 2.3(a). Extensive experimental studies have investigated the behaviour of individual ADAS elements and structural systems incorporating ADAS elements [Bergman and Goel, 1987; Whittaker, et al., 1991]. The tests showed a very stable hysteretic performance without any significant degradation after many load cycles. Figure 2.3(b) shows typical results of hysteretic curves for this connection, after a load cycle of one hundred. The principal characteristics which affect the behaviour of an ADAS device are its elastic stiffness, yield strength, and yield displacement.

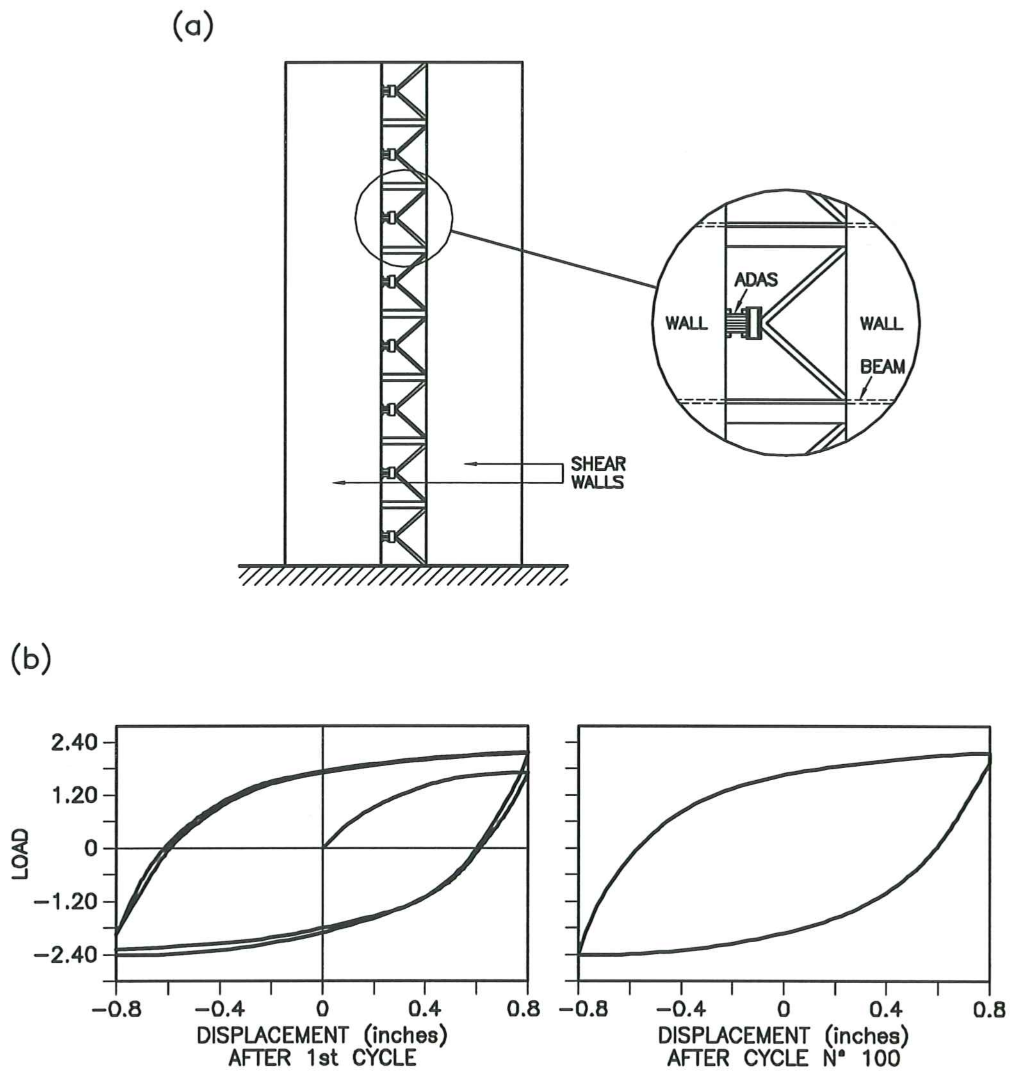

Figure 2.3. (a) Localisation of an ADAS system in reinforced concrete walls.
(b) Stable hysteretic response.

The device in Figure 2.4 is made with a metal pad with a trapezoidal section and it is called T-ADAS. In this case the forces acting are perpendicular to the plane shown. The triangular-plate concept for energy dissipators was originally developed and used in several base isolation applications [Boardman, et al., 1983]. Then it was extended to building dampers in the form of triangular ADAS elements: T-ADAS [Tsai and Hong, 1992]. The T-ADAS device combines two desirable features: no rotational restraint is required at the top of the brace connection assembly, and there is no potential for instability of the triangular plate due to excessive axial load in the device.

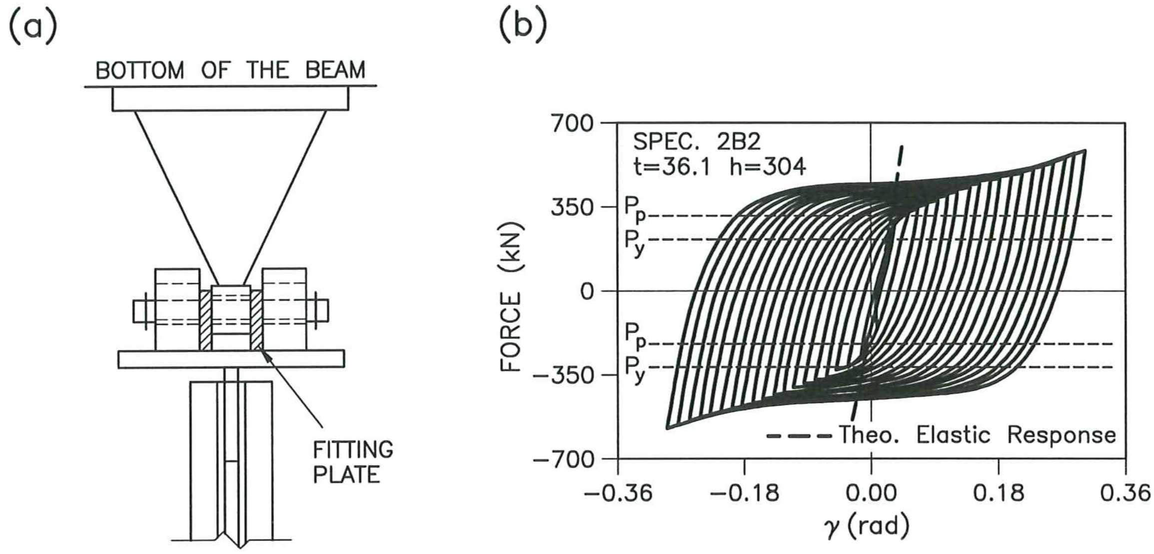

Figure 2.4. T-ADAS device. (a) Diagram (b) Hysteretic behaviour.

Figure 2.5 shows other bending dissipation devices proposed for seismic design and developed by Kajima Corporation, in Japan.

The dissipator shown in Figure 2.5(a) is called Joint Damper System and it is very simple to build. The bar has a constant section or, as in the figure, a variable section, to optimise the dissipation in its length. The bar is double fixed in its bigger sections at the ends, where the highest bending moments are induced. This device controls the response of two or more adjacent structures with different vibration characteristics. In general, the energy-absorbing capability is best utilised by concentrating installation of the dampers at the point where large relative displacements occur. This system is intended to deform in the same manner as an ADAS X-plate but in multiple directions.

The dissipator in Figure 2.4(b) is a single-tapered steel tube called Bell damper. It is a omni-directional damper with a constant bending moment along its length. The device in Figure 2.4(c), is called plane system or "honeycomb". It is principally for high rise buildings and it is formed by steel plates with honeycomb-shaped openings installed between storys to function only for acting forces parallel to the shown plane.

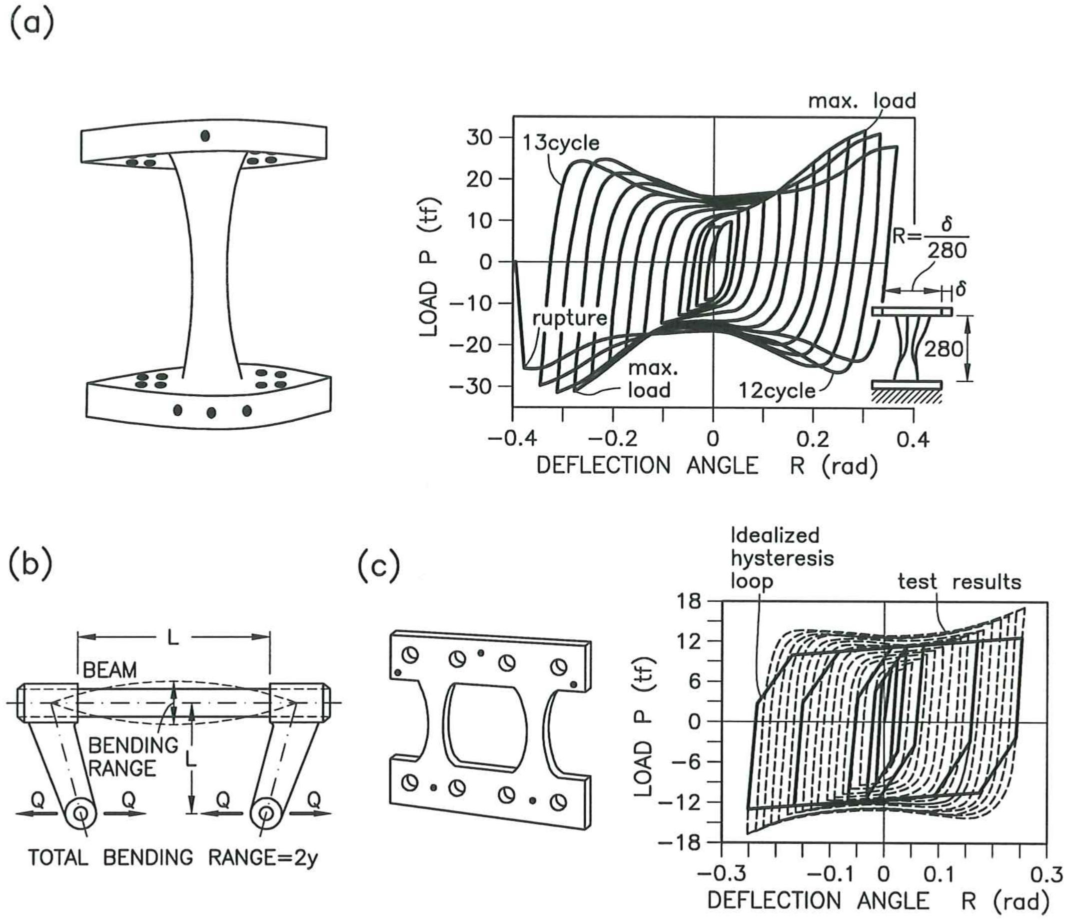

Figure 2.5. Dissipators for metal plastification. (a) Joint Damper System. (b) Bell Damper. (c) Honeycomb.

During an earthquake, these devices plastify and change the dynamic characteristics of the building. The fundamental period, for example, will be different in each instant of time and the dynamic forces will change. Moreover, there is a reduction of the seismic forces due to energy dissipation. These elements behave in the same way as the plastic hinge in conventional design. Besides energy dissipation devices could be combined with base isolation systems, both in building structures and in bridges (Figure 2.6), with the favourable effect of reducing base displacements.

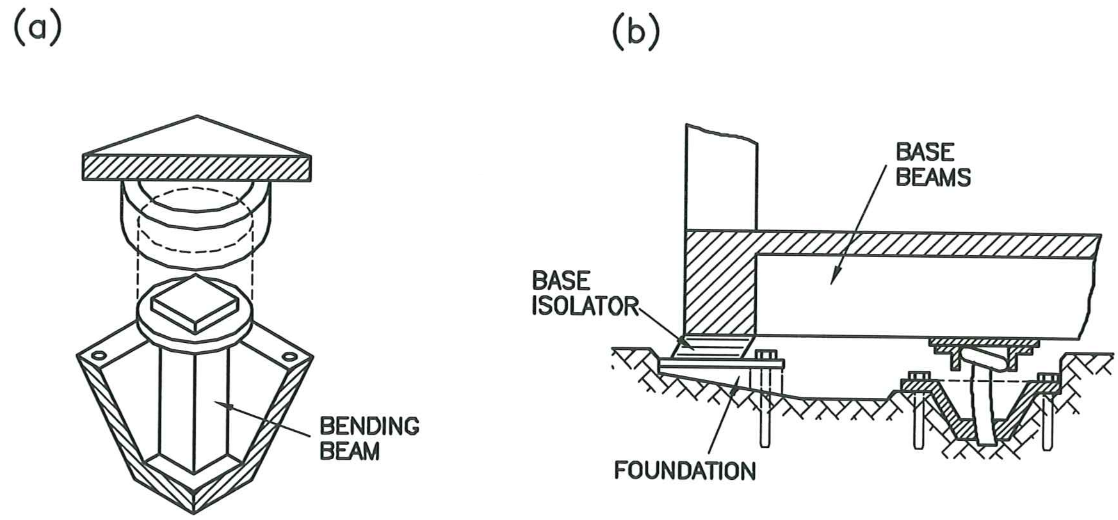

Figure 2.6. (a) Bending beam device. (b) Combination of base isolation and dissipators.

Figure 2.7 shows a torsion dissipator and its location during a shaking-table test. A combination of base isolation systems and energy dissipators to reduce base displacements is shown in Figure 2.7(b). Torsion devices are advantageous because utilise all the length of the beam, simply optimising the dissipation.

Figure 2.8 shows a panel-type dissipator and its corresponding hysteretic curve. As indicated previously, this device is made of a metallic plate with holes. The energy dissipated for cycle is stable, without a degradation of resistance.

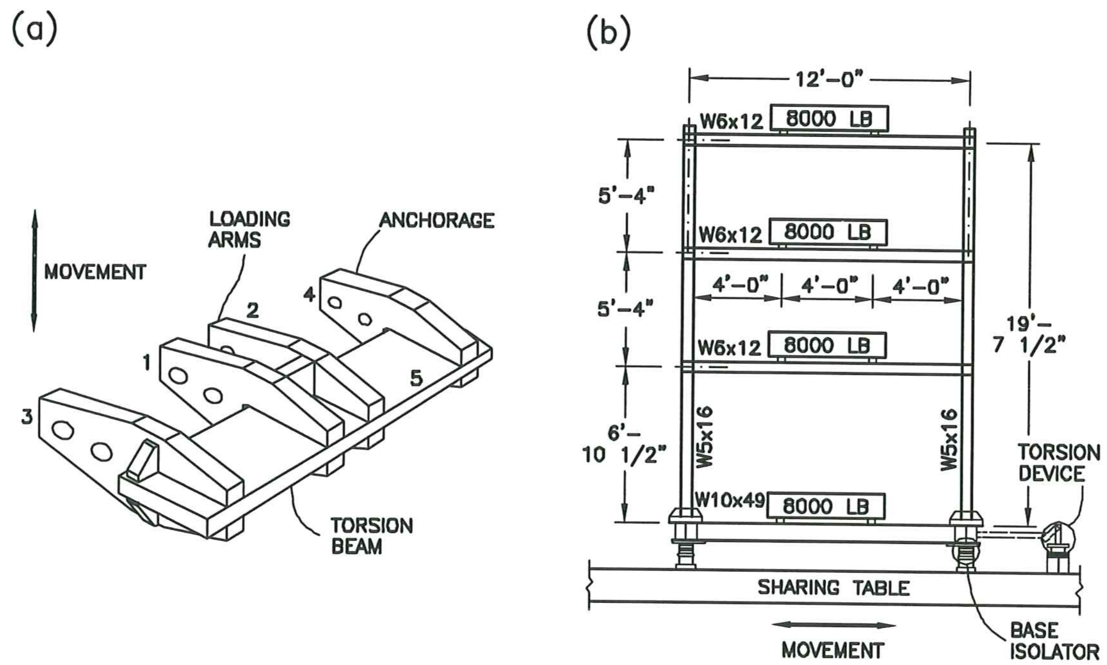

Figure 2.7. (a) Torsion dissipator. (b) Combination of base isolation and dissipator.

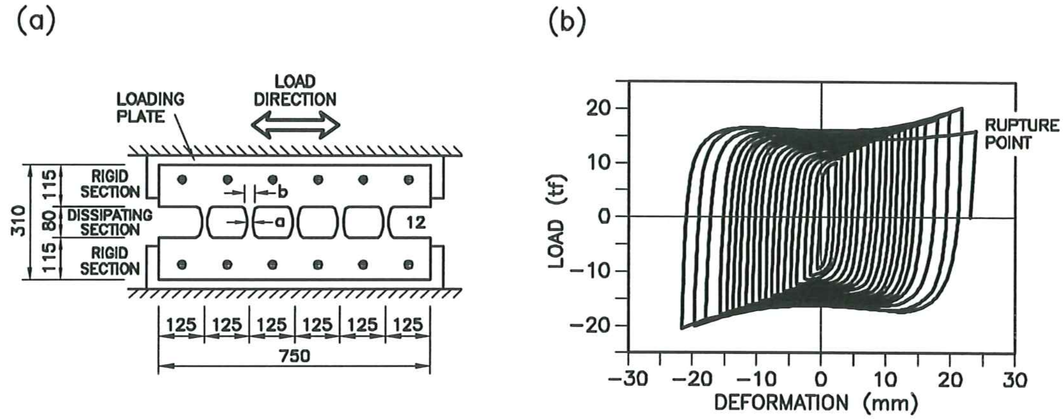

Figure 2.8. Panel-type dissipator. (a) Dimensions. (b) Hysteretic response.

The Shear Link (SL) device is an energy dissipation device suitable for the seismic protection, developed at the University of Girona [Cahis, et. al. 1997, Cahis, et. al. 1998, Bozzo, et. al. 1998]. It has a rectangular shape and an H section. Overall it is similar to the link of an eccentric braced frame and consequently it looks like a beam with stiffeners, as shown in Figure 2.9. The four stiffeners of the soul avoid its buckling under large shear-strain cyclic loading. Steel is ductile and permits a deformation up to 20% during a monothonic traction test. The total height of the connection is about 15.6 cm plus 1.2 cm for each base plate. One of the important advantages of the system is that the dimensions can be easily varied in order to achieve different yielding forces.

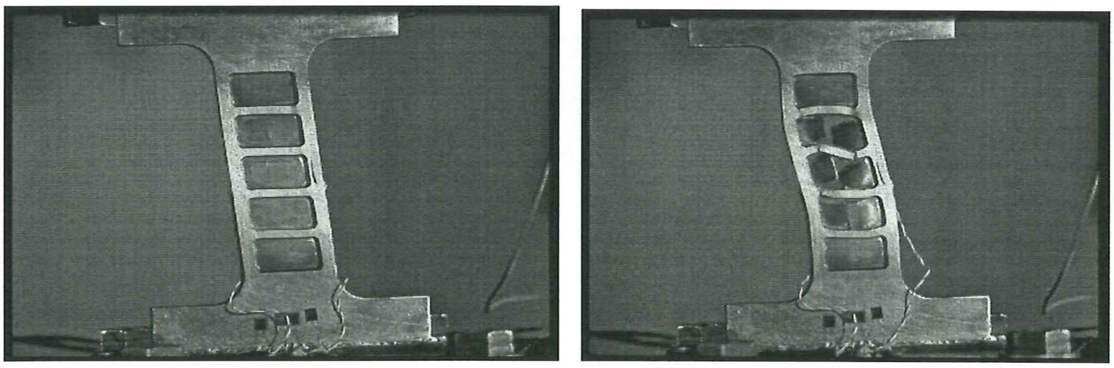

Figure 2.9. The Shear Link (SL) dissipator (a) Device working at a "shear mode". (b) Device working at a "bending mode" after the failure of the steel plates which provides additional factor of safety.

Preliminary tests on an SL device show a stable hysteretic curve (Figure 2.10). A series of dynamic characterisation tests have been performed at ISMES on a single-story one-bay full scale frame protected with SL devices.

The study is not over yet and the results are being conducted towards a new concept in stiff devices, rather than flexible ones, with a variety of dimensions and strengths. However, the tests indicate that the Shear Link device effectively reduces lateral displacements and forces. Similar to eccentric link braces, the plastification of the device is very stable under severe cycle loading. Resonance is not observed provided the connection plastifies, which occurs at low lateral displacements.

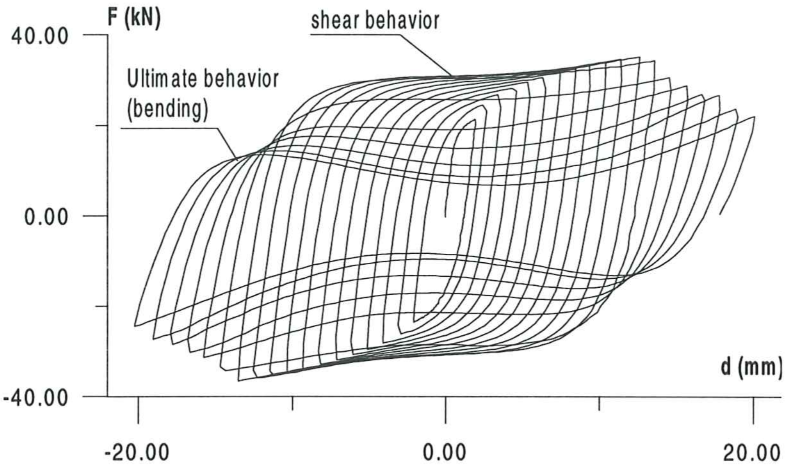

Figure 2.10. The experimental hysteretic curve of the Shear Link (SL) dissipator.

Another Shear Link dissipator made of aluminium was developed by Rai and Wallace [1998]. This device is designed to yield in shear at a lateral force less than that required to buckle the compression brace, providing significant energy dissipation potential. Aluminium was chosen because of its low yield strength, enabling the use of thicker webs, reducing the problems of web buckling. It is usually installed between the tips of the diagonal bracings and the beam from the floor above. Figure 2.11 shows a scheme of the Aluminium Shear Link and its hysteretic behaviour.

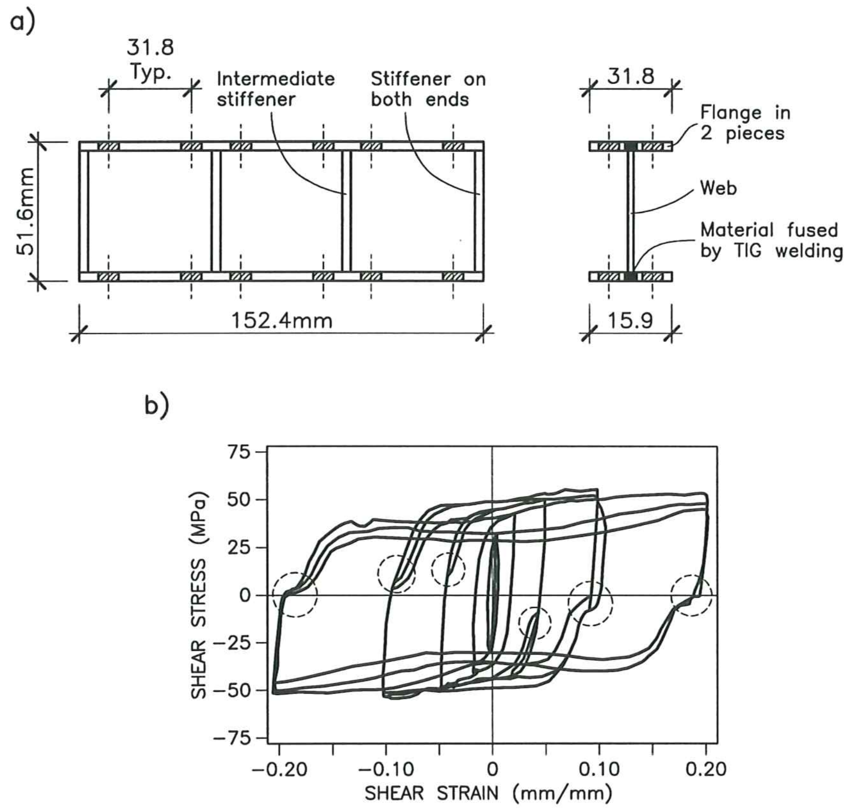

Figure 2.11. Aluminium Shear Link. (a) Scheme. (b) Hysteresis loop.

Another type of damper developed by Sakurai, et al. [1992] is called Lead Joint Damper (LJD). The device is a short lead tube with a wide hysteresis cycle as shown in Figure 2.12. The form of the LJD is determined such that it yields by shearing deformation and it is axially symmetrical in the horizontal directions. The upper and lower portions of the damper are for attachment to the structural members.

The mechanism of the seismic response reduction of LJDs is activated when two buildings with different fundamental periods are subjected to relative displacements produced by external forces such as an earthquake, causing a deformation of the joint damper set up between them. The damper absorbs the vibration energy by means of this deformation, and thus enables the response reduction of at least one of the two buildings.

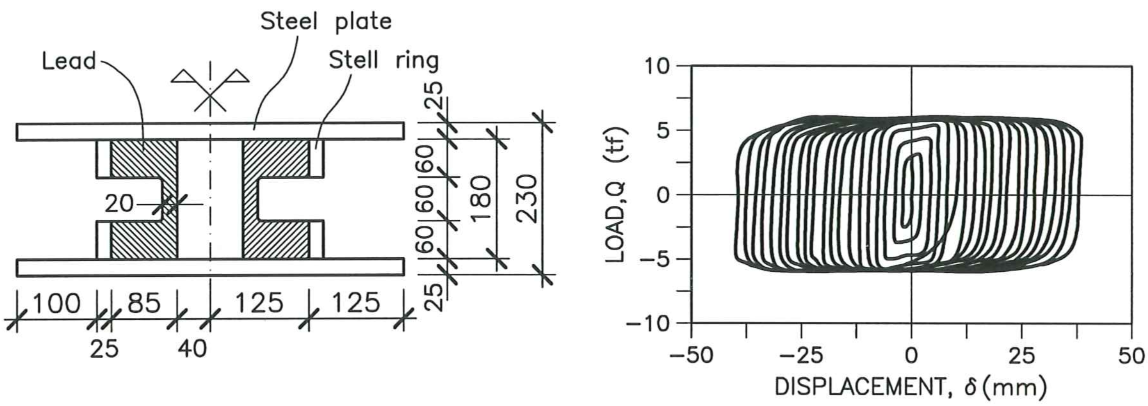

Figure 2.12. Lead Joint Damper. (a) scheme. (b) Hysteretic behaviour.

2.3.1.2 Lead Extrusion Devices (LEDs)

Extrusion is a mechanical process where a soft material changes its shape. This process is utilised in industry to make different products and it was initially proposed by Robinson and Greenbank [1975] for the seismic protection of bridges in New Zealand. Figure 2.13(a) shows a scheme of the device, where the lead flows through a hole which forces its change of section dissipating energy as heat. As shown in Figure 2.13(b), the resulting hysteretic curve for a device subjected to more than one hundred of load cycles is very stable. The hysteretic curve does not present either equivalent damping or stiffness and its analysis should be, in general, non-linear. An advantage of this characteristic is that the dissipated area in each load cycle is close to the maximum that it is frequency-independent. Skinner et al. [1993] give a simple analysis and design procedures for buildings with these devices.

2.3.1.3 Shape Memory Alloys (SMAs)

Shape Memory Alloys (SMA) are alloys that plastically deform at relatively low temperature and upon exposure to higher temperature, they will return to their shape prior to the deformation. Thus the material can undergo large hysteretic deformations without permanent effects: a thermal treatment permits it to recover plastic strain. The shape change occurs at a unique transformation temperature determined by the alloy composition. SMAs seem particularly good for passive damping devices because a stress induces in the material a micro-mechanical phase transition causing inelastic deformations with a large energy absorption. SMAs also exhibit a crystallographic phase change behaviour in response to applied heat or stresses. Therefore it is possible to achieve a large hysteretic deformation with residual strains that can be recovered by applying a suitable heating temperature [Attanasio, et al., 1997].

Figure 2.14 shows a device for passive seismic protection made with SMAs and produced by ALGA S.p.A.. It is a symmetrical two-span frame which is hinged at the base. The mechanism of energy dissipation is based on the elasto-plastic flexure of the SMA beam.

Although a relatively wide variety of alloys are known to exhibit the shape memory effect, only those that can recover substantial amounts of strain are interesting for applications in energy dissipators. One of them is the nickel-titanium alloy. It exists in one of two stable crystalline phases: a high temperature phase, called austenite, and a low temperature phase, called martensite. In the austenitic phase, the crystal structure is body-centered cubic. When cooled below its transformation temperature, the austenitic structure undergoes a diffusionless shear transformation to a highly-twinned martensite crystal structure.

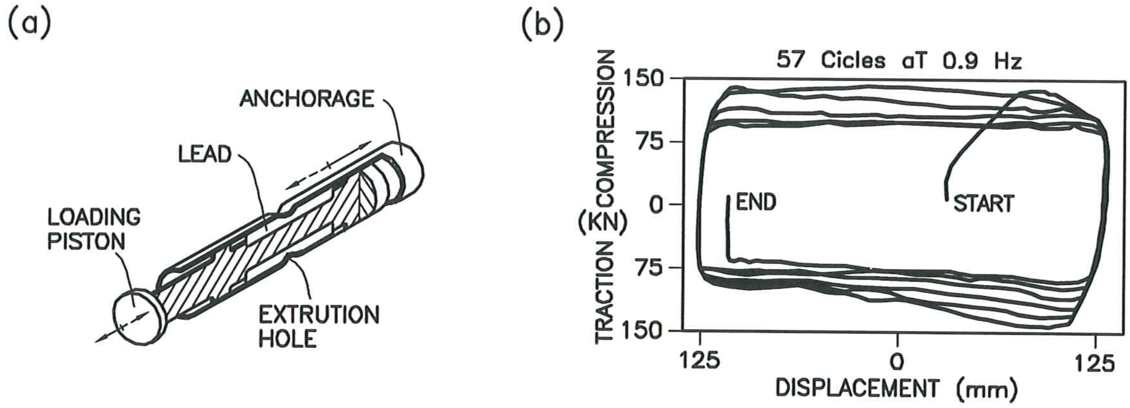

Figure 2.13. Lead extrusion device. (a) Scheme. (b) Hysteretic behaviour.

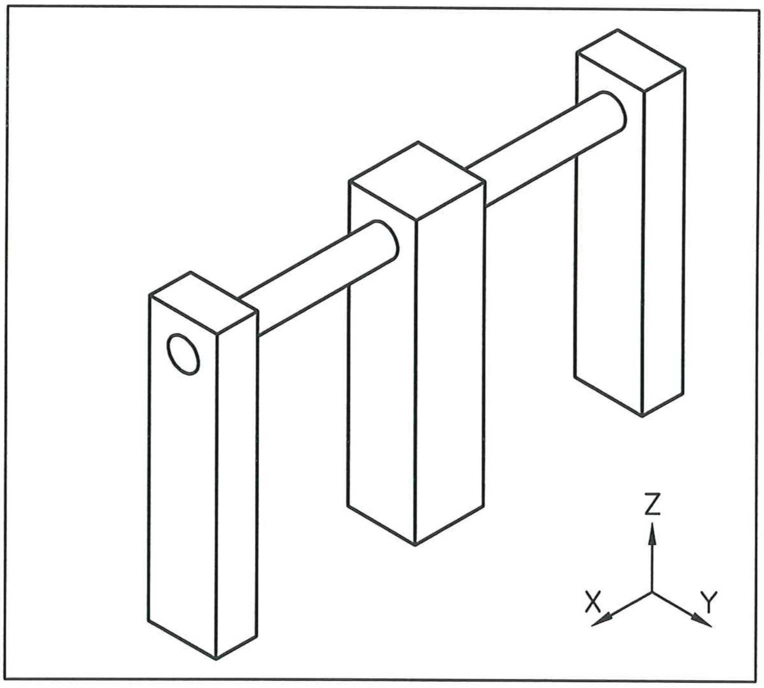

Figure 2.14. Scheme of a device made with Shape Memory Alloys.

In this martensitic phase, the twinned structure can be mechanically deformed to a parallel crystal structure. This deformation of approximately 8% remains as long as the alloy is below its transformation temperature. However, when the deformed martensitic structure warms through its transformation temperature, it returns immediately to the austenitic form. If unrestricted, the alloy will return to its original shape. These microstructural changes are summarised in Figure 2.15.

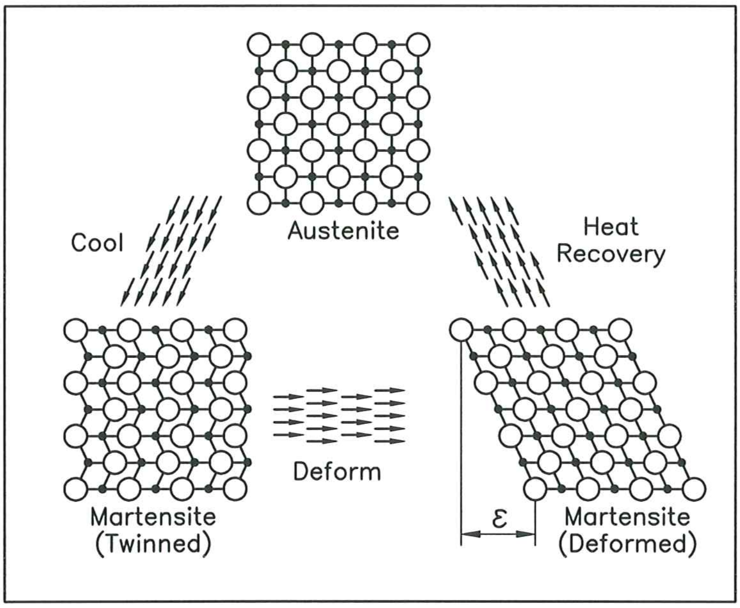

Figure 2.15. Microstructural changes during the shape memory cycle.

By varying the alloy composition, properties such as yield strength, ultimate strength and transformation temperature can be controlled, providing the potential for the development of simple devices which are self-centering and which perform repeatedly for a large number of cycles.

Several earthquake simulator studies on structures with SMA energy dissipators have been carried out. At the Earthquake Engineering Research Center of the University of California a 3-story steel model was tested with Nitinol (nickel-titanium) tension devices as part of a cross-bracing system [Aiken, et al., 1992]. At the National Center for Earthquake Engineering Research [Witting and Cozzarelli, 1992] a 5-story steel model was tested with copper-zinc-aluminium SMA devices. In this second study, devices with torsion, bending, and axial deformation modes were investigated. A typical hysteresis loop from these tests is shown in Figure 2.16. Results showed that the SMA dissipators were effective in reducing the seismic responses of the models.

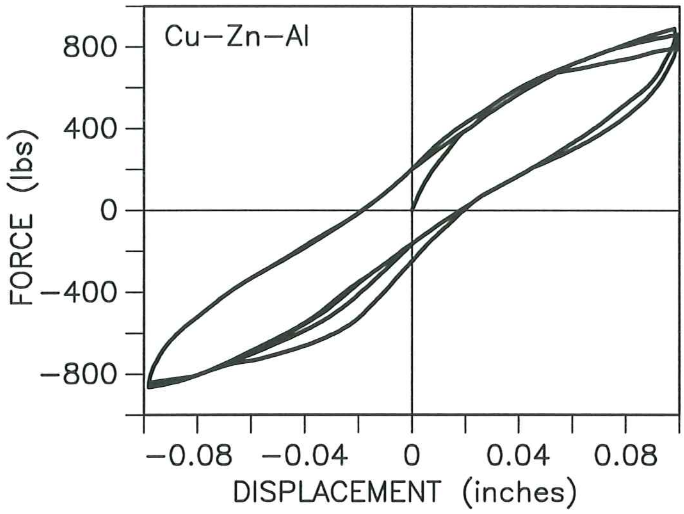

Figure 2.16. Cu-Zn-Al (Torsion) hysteresis loop.

Shape memory devices must be designed in such a way that the device deformations do not occur beyond the elastic limit strain (into the plastic range), resulting in a permanent yield in the material. The elastic limit strain varies with the SMA, but usually it is of the order of 5%. Some members of the SMA family also exhibit excellent fatigue resistance. Nitinol, among the family of SMAs, has outstanding corrosion resistance, even superior to that of stainless and other corrosion-resistant alloys.

2.3.2 Friction systems

Figures 2.17 to 2.19 show three systems of friction energy dissipation. The friction systems have the advantage of producing hysteretic stiff-plastic curves, where the energy dissipated in each cycle is maximum, as shown in Figures 2.17(b) and 2.18(b). These devices are usually placed at the crossing of the diagonals in a frame.

In Figure 2.18(a) a device developed by the Oiles Industry and Co., Ltd. of Tokyo, Japan, is shown. It is made with steel and neoprene and is installed in the intersection of the diagonals. The experimental study of a frame with these elements, shows an increment of the damping and a modification of the stiffness if compared wit the same frame without the devices. The principal disadvantage is its high cost in comparison with other systems.

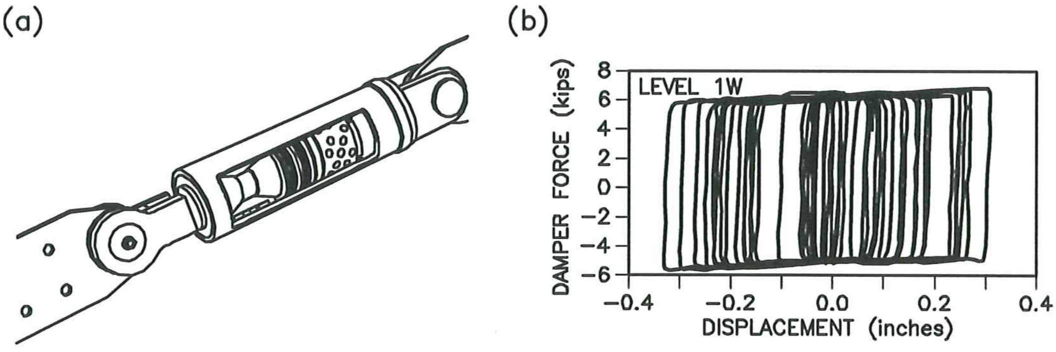

Figure 2.17. Sumimoto friction dissipator. (a) Section. (b) Hysteresis loops.

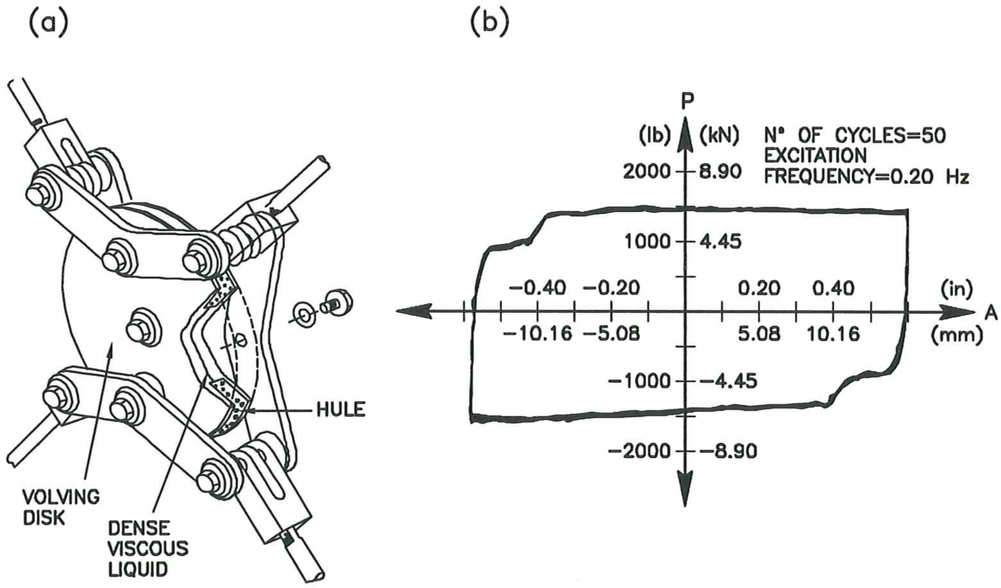

Figure 2.18. "Brace damper". (a) Section. (b) Hysteresis loop.

A friction device manufactured by Pall Dynamics, Ltd., is shown in Figure 2.19. It has been installed in three buildings in Canada, one retrofit and two new buildings [Pall, et al., 1987, 1991; Venzina, et al., 1992]. The Pall device is intended to be mounted in X-bracing. Several earthquake simulator studies of multi-story steel frames incorporating Pall devices have been performed [Filiatrault and Cherry, 1987; Aiken, et al., 1988], and a design methodology has been developed for friction-damped structures [Filiatrault and Cherry, 1990].

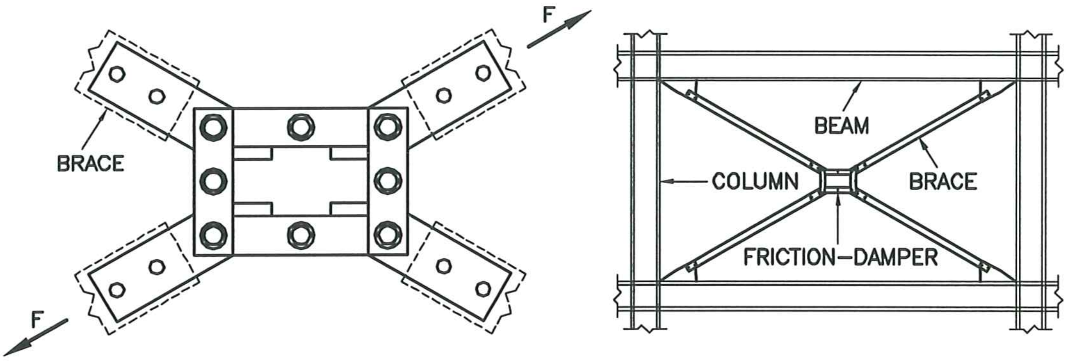

Figure 2.19. Pall Friction Device. (a) Section. (b) Typical installation.

Energy Dissipating Restraint (EDR) is a type of friction device developed and tested by Fluor Daniel, Inc. [Richter, et al., 1990]. The characteristics of this device are that it has a self-centering capability, the friction force is proportional to the relative displacement between the ends of the device, and it can be configured to provide energy dissipation for a wide range of seismic ground motions. Several hysteresis behaviours are possible such as those shown in Figure 2.21. The mechanism of the EDR is based on sliding friction through a range of motion with a stop at the end of travel. The friction surfaces in this device are bronze wedges sliding on a steel barrel. The variable parameters are the number of wedges, spring constant, gap, and spring precompression. A scheme of the EDR is shown in Figure 2.20 [Nims, et al., 1993].

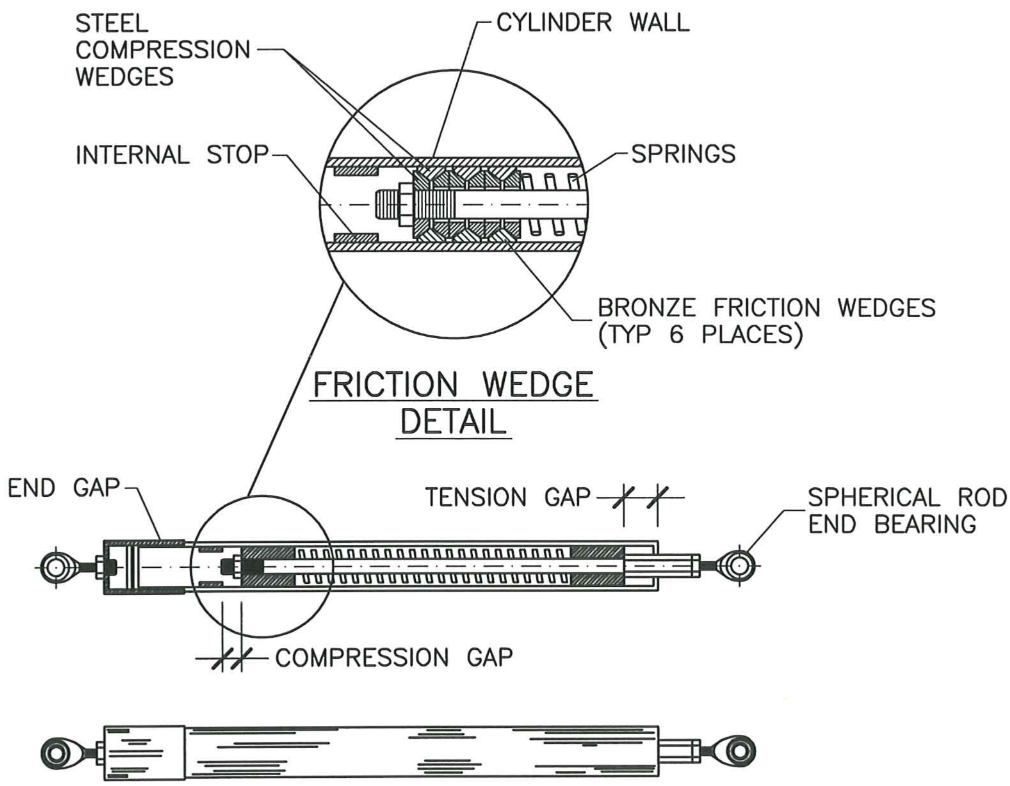

Figure 2.20. External and internal views of Energy Dissipating Restraint.

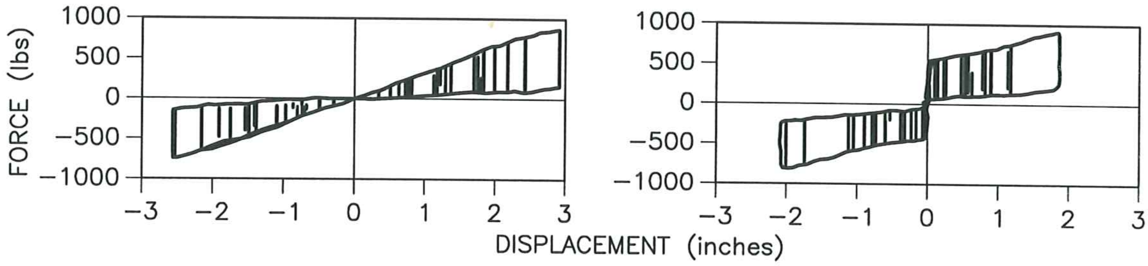

Figure 2.21. Fluor-Daniel EDR Hysteresis Loops.

A series of simple friction devices have been developed by Giacchetti, et al. [1989] and Tyler [1985] using a brake pad material on a steel friction interface. Roik, et al., [1988], Fitzgerald. et al., [1989] utilise even simpler friction schemes which allow slip in bolted connections. A refinement of the slotted bolted connection has been developed using a brass on the steel-steel friction surfaces (Figure 2.22) [Grigorian, et al., 1992].

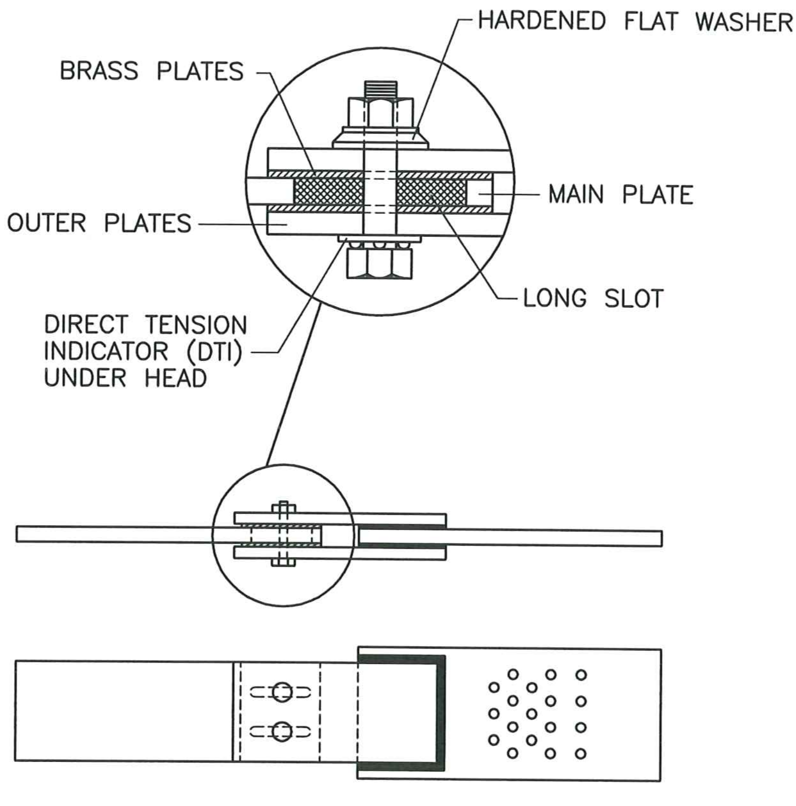

Figure 2.22. Diagram of a Slotted Bolted Connection.

Another device belonging to this category is the Constant Friction Slip Brace (CFSB) which will be utilised in the following parametrical study. CFSBs are installed in the diagonal braces of a frame and are pre-loaded on the friction interface. The location of CFSBs in a frame structure is shown in Figure 2.23. The operational principle of CFSB is to allow braces to deform through slippage along the friction interface at a defined level of axial brace load, and thus preclude buckling. By adjusting the pre-load on the friction interface, various slip loads are obtained and the maximum level of axial brace forces are controlled.

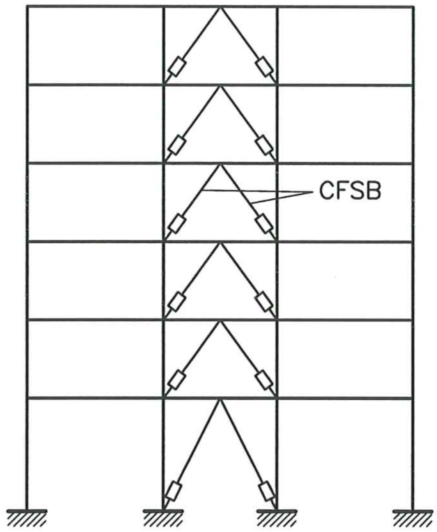

Figure 2.23. Location in a frame of a Constant Friction Slip Brace.

The friction force in these connections is equal to the product of the normal force and the friction coefficient. The normal force could vary in each connection utilising, for example, dynamometric instruments to produce a variation of the friction in height which could minimise the response. The friction coefficient, on the other hand, changes during sliding in function of the nature of the sliding surfaces, their relative velocity and the normal pressure at each instant. In general, the increment of the contact pressure produces a reduction of the friction coefficient. For material such as teflon, its variation range lays between 0.05 and 0.15 while for steel-steel contact it increases up to 0.3. The effects of the change of the friction coefficient during the displacement in passive protected structures could be important, as for the restoration of buildings which may enter the inelastic range. The variation of the ductility demand for these structures is not linearly independent on the variation of the friction coefficient.

The energy dissipation through friction gives an economic and simple alternative to reduce the seismic action. Positive characteristics of friction devices are the long-term reliability and maintenance. A negative characteristic is the possibility to introduce high frequencies as the devices undergo stick-slip behaviour. The possibility to change the hysteretic characteristics of each connection in simple form permits to find designs which minimise the response. On the other hand, as previously indicated, friction varies significantly during sliding, therefore its representation and numerical analysis is more difficult than in other systems. Different numerical studies indicate that the variation of the friction coefficient during sliding in base isolated structures is not significant if the structure lies in the linear elastic range [Bozzo and Barbat, 1995; Bozzo, et al., 1996]. These results suggest to utilise conservative values of the friction coefficient if a constant value has to be used.

2.3.3 Viscoelastic (VE) dissipators

These devices are made with metal plates jointed through a viscoelastic material (Figure 2.24(a)). They are some of the first devices utilised in buildings for vibration control of wind effects. Viscoelastic copolymers developed by the 3M Company have been used in a number of structural applications such as, for example, double-layer shear dampers using 3M material were installed in the 110-story twin towers of the World Trade Center in New York City [Mahmoodi, et al., 1989]. A total of 10000 dampers were installed in each tower and Figure 2.24(b) shows a typical location. Since then viscoelastic damping systems have been adopted in several tall buildings for the same purpose [Keel and Mahmoodi, 1986; Mahmoodi and Keel, 1989]. The seismic forces reduction obtained with these systems is exclusively based on the increment of structural damping. The dynamic characteristics of the building such as the fundamental period, do not change significantly since these devices present an equivalent stiffness and damping characteristics. The force-deformation curves in these elements have a defined slope, independent from the deformation level, as shown in Figure 2.24(c). The width of the hysteresis curve defines the equivalent damping. These systems are efficient for vibration control, but a significant reduction of the seismic forces could be obtained only if a high number of dissipators are installed. Moreover, their energy-absorbing capacity is highly effected by ambient temperature as it decreases while the temperature increases.

Anyway, their low unit cost and simple numeric analysis through linear equivalent characteristics are important factors which have motivated extensive experimental studies [Lin, et al., 1988; Aiken, et al., 1990; Kirekawa and Asano, 1992]. It must be pointed out that [Pong, et al., 1994] to significantly reduce the base shear utilising these systems, the geometric characteristics must be carefully chosen. The thickness of the damper plays an important role in the seismic resistance. It should be as small as possible without inducing large strain. Moreover, there is a high dependence of the equivalent damping on the environmental temperature during the test [Soong and Mahmoodi, 1990]. On the other hand, the energy dissipated is not so significantly affected by the change of temperature during the test as the equivalent damping.

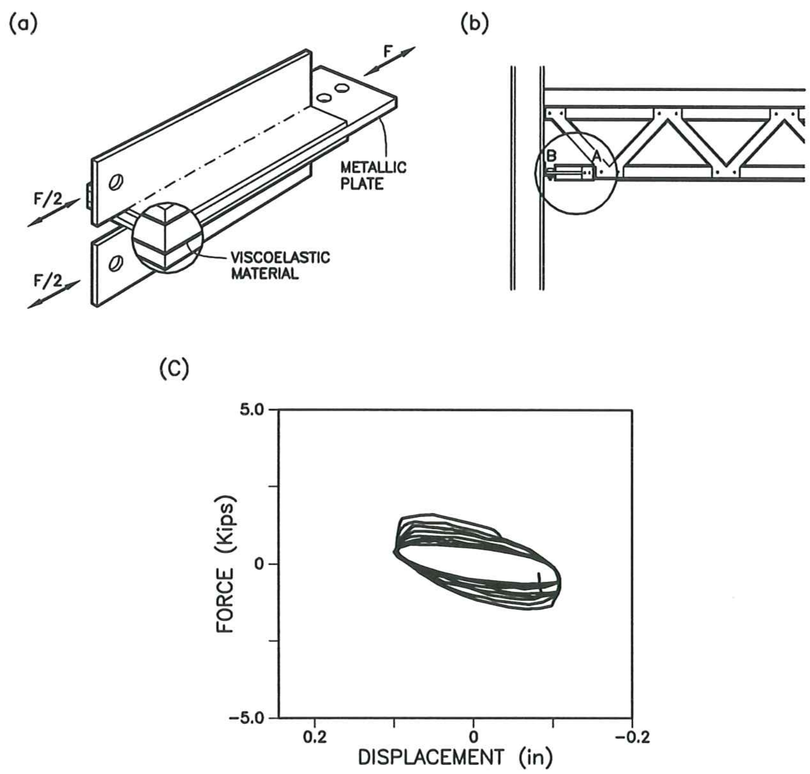

Figure 2.24. Viscoelastic dissipators. (a) Scheme. (b) Location in the twin towers. (c) Hysteretic behaviour.

Due to a stable slope under different displacements of the hysteretic curve it is possible to perform a linear elastic analysis, changing the damping. This behaviour has the advantage of allowing simple design procedures, similar to those of the conventional seismic design. Based on this behaviour, Soong and Mahmoodi [1990] stated optimisation procedures for the design of buildings with these systems.

A kind of viscous dissipator is the so-called viscous-damping (VD) wall system which has been developed by Oiles and Sumimoto Contruction (Figure 2.25). Results of shaking-table tests on a full-scale, 4-story frame showed very large response reductions of the frame with VD walls compared to the one without dissipators [Arima, et al., 1988]. A 4-story, reinforced-concrete test building with VD walls was constructed in Tsukuba, Japan, in 1987. Observed accelerations are 25 to 70 percent lower than those of the buildings without VD walls [Arima, et al., 1988]. A VD wall system in a 15-story building constructed in Shizouka City, Japan, will provide between 20 and 32 percent damping to the building, and achieve response reductions of the order of 75 to 80 percent [Miyazaki and Mitsusaka, 1992]. Another type of wall damping system has been developed and tested by Kumagai-Gumi Corporation. It is a super-plastic and silicone rubber VE shear damper that is included at the top connection of a wall panel to the surrounding frame [Uehara, et al., 1991]. Earthquake simulator tests of a 1/2-scale, 3-story steel frame showed significant response reductions in the VE damped model; as large as 50 percent in story accelerations and 60 percent in story displacements.

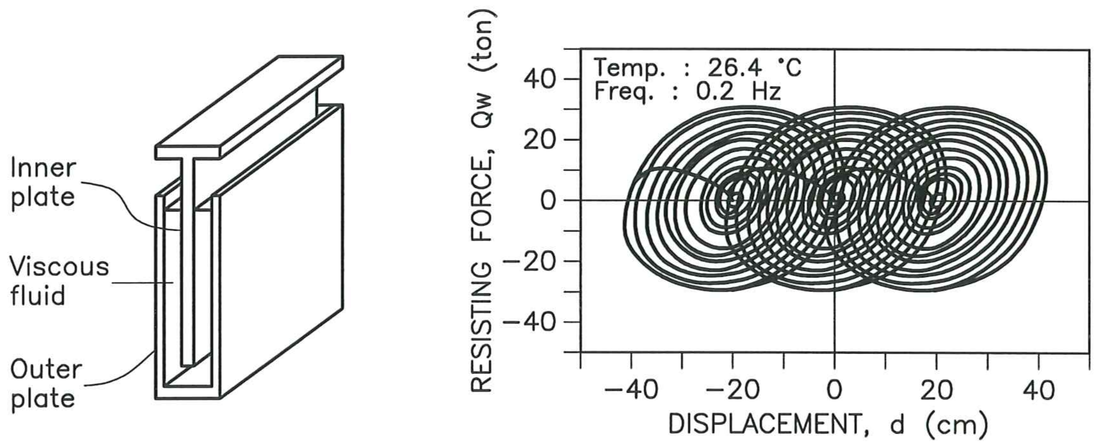

Figure 2.25. VD wall. (a) Scheme. (b) Hysteresis loop.

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CHAPTER 3

Analysis of Buildings with Dissipators

It is often necessary to evaluate the final design of buildings with energy dissipating devices by mean of inelastic dynamic response calculations. The aim is to verify the reliability of the damping assumptions taking into account the frequency and temperature dependencies of the devices.

3.1 NONLINEAR THEORY FOR DISSIPATING DEVICES

3.1.1 Introduction

The numerical simulation of structures with dissipators needs efficient nonlinear algorithms for a step-by-step analysis [Nagarajaiah, et al., 1991]. Many procedures have been proposed to realise this integration in the time. The procedures more utilised are those called monolithic. They do not distinguish the degrees of freedom of the structure from those corresponding to the dissipators. The algorithms based on static condensation reduce the order of the nonlinear problem to the number of its unelastic degrees of freedom [Lager, et al., 1986]. In this way it is possible to obtain a high reduction of the d.o.f. of the structure. Apart from the static condensation it is possible to uncouple the equations of motion of the dissipators and the structure in an iterative form. These iterative methods, combined with the corresponding linearisation of the nonlinear terms are called iterative schemes for blocks [Barbat, et al., 1996] and are showed in the following in more detail.

3.1.2 Scheme of the monolithic step-by-step integration

The equation of motion is

$$\boldsymbol{M\ddot{D}} + \boldsymbol{C\dot{D}} + \boldsymbol{f(D)} = \boldsymbol{F} \tag{3.1}$$

where $\boldsymbol{f(D)}$ represents the nonlinear forces. The procedures of the monolithic simulation solve the nonlinear static problem in each time interval, considering the complete system, in a way similar to the procedures of the inelastc analysis of conventional structures. In each step, the solution is obtained utilising the tangent stiffness matrix or utilising the equivalent pseudo-forces. The evaluation and decomposition of the tangent matrix at each step is a computing expensive procedure; therefore these methods are not applied to base isolation. A more efficient alternative is to evaluate the equivalent pseudo-static forces at each time step. In this case the equation of motion is:

$$\boldsymbol{M\ddot{D}} + \boldsymbol{C\dot{D}} + \boldsymbol{K}_0\boldsymbol{D} = \boldsymbol{F} - \boldsymbol{F}_N \tag{3.3}$$

were the vector $\boldsymbol{f(D)}$ is linearised in the following way

$$\boldsymbol{f(D)} = \boldsymbol{K}_0\boldsymbol{D} + \boldsymbol{F}_N$$

The stiffness matrix $\boldsymbol{K}_0$ is linear and the pseudo forces vector $\boldsymbol{F}_N$ is time independent. The matrix $\boldsymbol{K}_0$ could be the initial tangent stiffness matrix.

The principal advantage of utilising pseudo-forces is that the original stiffness matrix is decomposed only once at the beginning of the integration, because this operation is the one which contributes more to the numerical inefficiency. In each step, the nonlinear problem is limited to the evaluation of the equivalent forces. If these forces correspond to a few degrees of freedom the procedure can be efficient as the calculations are limited to the element level.

3.1.3 Scheme of integration for blocks

The monolithic algorithms requires discretisation procedures of the equations of motion which include their linearisation and coupling [Su, et al., 1989]. The global system is not easy to solve due to the different stiffnesses of the; therefore it is necessary to utilise very small time steps. These procedures require a great number of iterations and their convergency is slow. To this aim schemes of iteration for blocks which reduce the number of iterations and the time of convergency has been proposed [Codina, 1992]. The procedures with iterative blocks, as presented in detail in the following, deal with the nonlinearity of the problem as an iterative actualisation of the force $\boldsymbol{f}$ in the protection devices.

The following equations represent a generic coupled problem [Codina, 1992]

$$\begin{bmatrix} \boldsymbol{A}_{11} & \boldsymbol{A}_{12} \\ \boldsymbol{A}_{21} & \boldsymbol{A}_{22}(\boldsymbol{y}) \end{bmatrix} \begin{bmatrix} \boldsymbol{x} \\ \boldsymbol{y} \end{bmatrix} = \begin{bmatrix} \boldsymbol{q}_1 \\ \boldsymbol{q}_2 \end{bmatrix} \tag{3.4}$$

where $\boldsymbol{x}$ and $\boldsymbol{y}$ are unknown vectors, $\boldsymbol{q}_1$ and $\boldsymbol{q}_2$ are load vectors and $\boldsymbol{A}_{ij}$, i,j =1,2 are matrices, where $\boldsymbol{A}_{22}$ is function of $\boldsymbol{y}$. The equations of the system (3.4) are linearly coupled. The matrix $\boldsymbol{A}_{22}$ is linearised in the following way

$$\boldsymbol{A}_{22}(\boldsymbol{y}^{(i)})\boldsymbol{y}^{(i)} \approx (\boldsymbol{A}_{22}^{L}\boldsymbol{y}^{(i)} + \psi(\boldsymbol{y}^{(i-1)})) \tag{3.5}$$

where A_{22}^L is the linearised form of A_{22}. Starting from equation (3.4) it is possible to state the following monolithic procedure, similar to the one previously showed for the pseudo-forces

$$\begin{bmatrix} \mathbf{A}_{11} & \mathbf{A}_{12} \\ \mathbf{A}_{21} & \mathbf{A}_{22}^L \end{bmatrix} \begin{bmatrix} \mathbf{x}^{(i)} \\ \mathbf{y}^{(i)} \end{bmatrix} = \begin{bmatrix} \mathbf{q}_1 \\ \mathbf{q}_2 - \psi(\mathbf{y}^{(i-1)}) \end{bmatrix} \tag{3.6}$$

Alternatively, starting from equation (3.6) and using the Gauss-Seidel iteration for blocks it is possible to write the following coupled equations

$$A_{11}x^{(i)} = q_1 - A_{12}y^{(i-1)} \tag{3.7}$$

$$A_{22}^L y^{(i)} = q_2 - \psi(y^{(i-1)}) - A_{21}x^{(i)} \tag{3.8}$$

This is a first alternative to state an iterative procedure for blocks. Equation (3.7) could be solved first assigning a value to $x^{(i)}$ and this value is utilised to solve equation (3.8) to obtain the vector $y^{(i)}$.

The monolithic analysis procedures do not take into consideration that energy dissipators are installed in certain points of the structure and that the number of the degrees of freedom associated to them is much lower than the total number of the degrees of freedom of the structure. A procedure more efficient from a numerical point of view and of interest in the calculation of buildings with dissipators, is to utilise techniques of sub-structures. This alternative was proposed for the dynamic analysis of structures with nonlinearities localised utilising the Ritz vectors [Leger, et al., 1986]. An alternative for the sub-structures technique is to decompose the equations of motion as

$$M = \begin{bmatrix} M_{11} & M_{12} \\ M_{21} & M_{22} \end{bmatrix} \qquad C = \begin{bmatrix} C_{11} & C_{12} \\ C_{21} & C_{22} \end{bmatrix} \qquad K = \begin{bmatrix} K_{11} & K_{12} \\ K_{21} & K_{22} \end{bmatrix} \tag{3.9}$$

defining the following transformation matrix

$$T = \begin{bmatrix} \Phi_{11} & -K_{11}^{-1}K_{12} \\ 0 & I \end{bmatrix}, \qquad \eta = TD$$

where Φ_{11} is the first modal matrix with dimensions (n x q) corresponding to the first q vibration modes of the structure with fixed base. The degrees of freedom 1 and 2 correspond to the linear and nonlinear regions of the system. This transformation matrix modifies the equations of motion to

$$M^*\ddot{\eta} + C^*\eta + K_N^* = F^* \tag{3.10}$$

where

$$M^* = T^T M T, \qquad C^* = T^T C T, \qquad K_N^* = T^T K T, \qquad F^* = T^T F$$

The transformation decouples the equations of motions and the mass, damping and stiffness matrices are written in the following way:

$$M^* = \begin{bmatrix} \Phi_{11}M_{11}\Phi_{11} & \Phi_{11}M_{12} - E \\ M_{21}\Phi_{11} - E^T & A \end{bmatrix} \tag{3.11}$$

$$C^* = \begin{bmatrix} \Phi_{11}C_{11}\Phi_{11} & \Phi_{11}C_{12} - \Phi_{11}C_{11}K_{11}^{-1}K_{12} \\ C_{21}\Phi_{11} - K_{21}K_{11}^{-1}C_{11}\Phi_{11} & B \end{bmatrix} \tag{3.12}$$

$$K^* = \begin{bmatrix} \Phi_{11}K_{11}\Phi_{11} & 0 \\ 0 & K_{22} - K_{21}K_{11}^{-1}K_{12} \end{bmatrix} \tag{3.13}$$

where

$$A = K_{21}K_{11}^{-1}M_{11}K_{11}^{-1}K_{12} - K_{21}K_{11}^{-1}M_{12} - M_{21}K_{11}^{-1}K_{12}$$

$$B = K_{21}K_{11}^{-1}C_{11}K_{11}^{-1}K_{12} - K_{21}K_{11}^{-1}C_{12} - C_{21}K_{11}^{-1}K_{12}$$

$$E = \Phi_{11}M_{11}K_{11}^{-1}K_{12}$$

The transformation vectorial base Φ_{11} includes the vibration modes or the Ritz vectors [Leger, et al., 1986]. The analysis is still nonlinear but the principal advantage of the transformation is the reduction of the problem size to only a few vibration modes. The proper vectors or the Ritz vectors are obtained at the beginning of the integration and the nonlinear problem is limited to the degrees of freedom of the isolators.

3.2 DESIGN OF BUILDINGS WITH DISSIPATORS

There are several aspects to consider in the design of buildings with energy dissipators. A first aspect is how to model the dissipation systems and which local parameters to utilise. A second aspect is how to modify these parameters along the building height to obtain a best response [Ruiz, et al., 1995]. Bozzo, et al. [1996] and Foti, et al. [1996] propose the design criteria which are shown in the following. At local level, the hysteretic response of these connections can be represented in an approximated form utilising expressions such as

$$\begin{aligned} F &= k\,d && \text{for } d \le d_y \\ F &= k\,d_y\left(\frac{d}{d_y}\right)^n && \text{for } d > d_y \end{aligned} \tag{3.14}$$

where F is the cyclic force, d_y is the yielding displacement, k is the element stiffness $k = \dfrac{F_y}{d_y}$, F_y is the yield force and n is a positive exponential to best represent the hysteretic characteristics of the connection. The value of n is less than one and is selected to fit the hysteretic curve to the experimental results of the connection. Alternatively to this simple model, Wen's model [1976] can be utilised. It is more flexible and precise than the previous one. The disadvantage is that its parameters do not have a specific physical meaning.

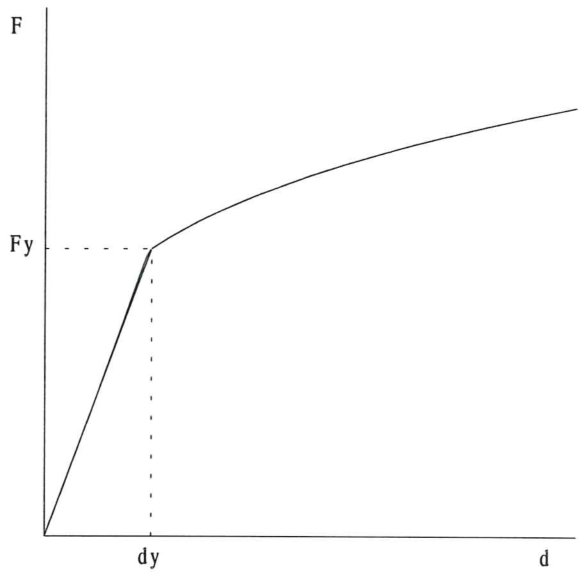

Figure 3.1. The general F-d curve described by equation (3.14).

The simplest dissipator which is adequately represented by equation (3.14) is the friction one. In this case the hysteretic response is rigid-plastic and, therefore, the parameters are $n = 0$, $d_y = 0$, and $F_y = \mu N$, where μ is the friction coefficient and N is the normal force in the connection. As the friction coefficient is function of the kind of material utilised in the connection, the only design parameter in these connections is the normal force N.

The dissipators based on metal plastification are also adequately analysed utilising this model. For example, as shown in the experimental results presented by Hanson, et al. [1992], the adequate design parameters for the ADAS connections are d_y= 0.6 cm and n = 0.25. It should be noticed that in this system the X-shaped metal plates are set in parallel; therefore, independently from its number in the connection, they yield at the same displacement d_y. Similarly to a friction connection, the only design parameter for ADAS is the yielding force F_y.

The second aspect to be considered in the design of buildings with energy dissipators is how to change the yielding force along the height of the building. Many procedures have been proposed especially for viscoelastic dissipators and, in a reduced measure, for dissipators based on metal plastification [Tsai, et al., 1993; Scholl, 1993]. Bozzo, et al. [1996] propose a simple procedure based on equivalent static forces.

Further, the criterion generally accepted that under a severe earthquake the building behaves in the linear elastic range and non-linearities are concentrated in the connections is applied. Considering that in high seismic areas an intermediate structural ductility level is necessary, an efficient design criterion is to accept that under extreme earthquakes the structure dissipates part of the energy. Numerical studies of buildings with base isolation subjected to severe earthquakes which show an inelastic structural response indicate that the efficacy of the isolation decreases if compared to the case of an elastic structural response [Bozzo, et al., 1996a; Bozzo and Barbat, 1995]. In buildings with energy dissipators it is possible to take advantage of this additional capacity of the structure, obtaining a higher safety coefficient for extreme earthquakes.

Bozzo, et al. [1996b] propose an equivalent non-linear one-degree-of-freedom model for the design of buildings. This equivalent model is obtained from the equation of motion for non-linear multi-degree-of-freedom structures:

$$\boldsymbol{M}\ddot{\boldsymbol{D}} + \boldsymbol{C}\dot{\boldsymbol{D}} + \boldsymbol{K}\boldsymbol{D} = -\boldsymbol{M}\boldsymbol{J}\,\ddot{u}_g(t)$$

From these equations and considering a change of variable

$$\lambda(t) = \frac{u(t)}{\ddot{u}_{g_{max}}}$$

it is possible to obtain the normalised equation

$$\boldsymbol{M}\,\ddot{\boldsymbol{\lambda}}(t) + \boldsymbol{C}\dot{\boldsymbol{\lambda}}(t) + \frac{\boldsymbol{V}(t)}{\ddot{u}_{g_{max}}} = -\boldsymbol{M}\boldsymbol{J}\,\ddot{u}_g(t) \tag{3.15}$$

Miranda [1990] indicates that, at least for a preliminary analysis, the inelastic response can be conveniently represented with the first vibration mode φ_1, which enables the matrix equation (3.15) to be simplified:

$$m^*\ddot{\lambda}(t) + c^*\dot{\lambda}(t) + \varphi_1^T\frac{V(t)}{\ddot{u}_{g_{max}}} = -m_1\ddot{u}_g(t) \tag{3.16}$$

or dividing by the equivalent mass to obtain the normalised equation

$$\ddot{\lambda}(t) + 2\omega\ \xi\ \dot{\lambda}(t) + \eta_{ubc}\rho(t) = -\frac{m_1}{m^*}\ddot{u}_g(t) \tag{3.17}$$

where

$$\eta_{ubc} = \varphi_1^T\frac{V_{ubc}}{m^*\ddot{u}_{g_{max}}}$$

Figure 3.2 describes this equivalent one-degree-of-freedom model, good for the preliminary design of buildings with dissipators. This model assumes that the hinges or energy dissipation sections appear all at the same instant. The design procedure presented in this section is based on the assumption that under the same load all the dissipators start to work at the same time. Therefore, under static loads and considering elasto-plastic dissipators -that is friction devices or devices without post-yielding stiffness-, the proposed equivalent model is correct. In reality the devices have a certain hardening and under dynamic loads do not work all at the same time, forming hinges at different instants. Of course, this simple model permits to conveniently dimension multi-degree-of-freedom buildings with dissipators [Bozzo, et al., 1996].

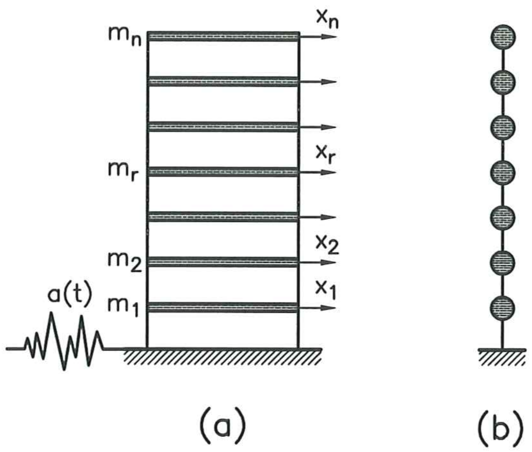

Figure 3.2. Multistory building and equivalent non-linear one-degree-of-freedom model.

3.2.1 Hysteretic modelling

In the linear theory for a structural system, the equations of motions are of the following form:

$$\boldsymbol{M}\ddot{\boldsymbol{u}}(t)+\boldsymbol{C}\dot{\boldsymbol{u}}(t)+\boldsymbol{K}\boldsymbol{u}(t)=\boldsymbol{R}(t) \tag{3.18}$$

where:
$\ddot{\boldsymbol{u}}(t),\dot{\boldsymbol{u}}(t),\boldsymbol{u}(t)$ are respectively the vectors of displacements, velocities and accelerations at the degrees of freedom of the system;
$\boldsymbol{M}$, $\boldsymbol{C}$ and $\boldsymbol{K}$ are respectively the mass, damping and stiffness matrices of the system;
$\boldsymbol{R}(t)=-\boldsymbol{M}\boldsymbol{J}a(t)$ are the seismic forces;
$a(t)$ is the ground acceleration;
$\boldsymbol{J}$ is a unit vector.

The standard mode superposition method of analysis is used to solve the dynamic equilibrium equations. This involves expressing the node displacements as

$$\boldsymbol{u}(t)=\boldsymbol{\Phi}_1\, y(t)_1+\boldsymbol{\Phi}_2\, y(t)_2+\ldots+\boldsymbol{\Phi}_i\, y(t)_i+\ldots+\boldsymbol{\Phi}_n\, y(t)_n \tag{3.19}$$

where $y(t)_i$ is the amplitude response of *i-th* mode. The substitution of equation (3.19) into equation (3.18) and the pre-multiplication by $\boldsymbol{\Phi}^T$ produces a set of uncoupled modal equations:

$$\boldsymbol{I}\ddot{\boldsymbol{Y}}(t)+\boldsymbol{\Lambda}\dot{\boldsymbol{Y}}(t)+\boldsymbol{\Omega}^2\boldsymbol{Y}(t)=\boldsymbol{F}(t) \qquad (3.20)$$

The assumption that $\boldsymbol{\Lambda}=\boldsymbol{diag}[2\lambda_i\omega_i]$ requires that the damping matrix C must be proportional, where λ_i is the modal damping ratio.

If a passive protection system has a non-linear behaviour, some terms due to non-linearity are present in the left-hand side of equation (3.18):

$$\boldsymbol{M}\ddot{\boldsymbol{u}}(t)+\boldsymbol{C}\dot{\boldsymbol{u}}(t)+\boldsymbol{K}\boldsymbol{u}(t)+\boldsymbol{f}_1+\boldsymbol{f}_2=\boldsymbol{R}(t) \qquad (3.21)$$

f_1 is the force produced by the elements with a hysteretic behaviour;
f_2 is the force due to the elements with a friction behaviour.

To find out a solution of the problem a mathematical model which adequately describes the forces f_1 and f_2 produced by the non-linear system is necessary. The hysteretic force could be calculated with constitutive models. These models can be described by both algebraic and differential equations. The differential equations have the advantage to fill a wide range of possible materials, from the elastic ones to the elasto-plastic ones with a high numerical efficiency.

The SADSAP model

The method of non-linear dynamic analysis used by the SADSAP program is designed to be applied to structural systems with a limited number of predefined non-linear elements. In addition, the structural system must be structurally stable without the non-linear elements (all structures can be made to satisfy this condition if linear elements with very small stiffness are added to the basic computer model). If a structural system satisfies these conditions it is possible to calculate the non-linear dynamic response with almost the same numerical effort as required by a linear dynamic analysis. Since the behaviour of these elements is velocity and history dependent they cannot be used in a normal static analysis.

The global dynamic equilibrium equations of a large elastic structure with non-linear or energy dissipating elements can be written in the following form:

$$\boldsymbol{M}\ddot{\boldsymbol{u}}(t)+\boldsymbol{C}\dot{\boldsymbol{u}}(t)+\boldsymbol{K}\boldsymbol{u}(t)+\boldsymbol{R}(t)_N=\boldsymbol{R}(t) \qquad (3.22)$$

where $\boldsymbol{R}(t)_N$ are the global node forces due to the sum of the forces in the non-linear elements. Also, equation (3.22) can be written in terms of the modal co-ordinates as

$$\boldsymbol{I}\ddot{\boldsymbol{Y}}(t)+\boldsymbol{\Lambda}\dot{\boldsymbol{Y}}(t)+\boldsymbol{\Omega}^2\boldsymbol{Y}(t)=\boldsymbol{F}(t)-\boldsymbol{F}(t)_N \qquad (3.23)$$

in which the non-linear modal forces are given by

$$F(t)_N = \boldsymbol{\Phi}^T \boldsymbol{R}(t)_N \tag{3.24}$$

The terms in equations (3.22) have the same meaning as before.

At any time "t" the deformations $\boldsymbol{\Delta}(t)$, in the local co-ordinate system of the non-linear elements, can be expressed in terms of the node point displacements, $\boldsymbol{u}(t)$, by a standard displacement transformation equations of the form:

$$\boldsymbol{\Delta}(t) = \boldsymbol{A}\boldsymbol{u}(t) \tag{3.25}$$

Since $\boldsymbol{u}(t)_N = \boldsymbol{\Phi}\boldsymbol{Y}(t)$, the deformations in the non-linear elements can be expressed directly in terms of the modal co-ordinates as

$$\boldsymbol{\Delta}(t) = \boldsymbol{B}\boldsymbol{Y}(t) \tag{3.26}$$

in which $\boldsymbol{B} = \boldsymbol{A}\boldsymbol{\Phi}$.

It is very important to note that the ***B*** matrix is not a function of time and is normally a small matrix since its side is "the number of non-linear deformation terms" time "the number of modes" required for the dynamic analysis. Therefore, it need be calculated only once prior to the integration of the modal equations and it can be easily retained in high-speed computer storage all phases of the non-linear solution.

At any time, given the deformations and history of behaviour in the non-linear elements, the forces in the non-linear elements $\boldsymbol{P}(t)$ can be evaluated from the basic non-linear properties of the elements. The non-linear modal forces are then calculated from:

$$\boldsymbol{F}(t)_N = \boldsymbol{B}^T \boldsymbol{P}(t)_N \tag{3.27}$$

The non-linear analysis method can now be summarised. The first step in the analysis is to conduct a normal static analysis and to calculate the eigen or Ritz vectors for the computer model without the non-linear elements. A standard time-history analysis is then performed by the mode superposition method. The modal equations are integrated by a method which is exact for a linear variation of the load during the defined time increment. If non-linear elements are present the loads in the non-linear elements are calculated at the end of each time step. The element loads are converted to modal loads and applied to the right hand side of the modal equations. Iteration is then performed within the time step until the loads at the end of the time increment converge.

For time history dynamic analysis it is possible to have any number of one-dimensional non-linear elements, each connected between any two node points, within the structural analysis model. The elements may behave axially, torsionally or in shear in two different directions. Several different types of non-linear elements are possible. For this study the plasticity element with energy dissipation has been chosen. The force-deformation relationship for this element is of form (3.14). Its parameters are defined in 3.14. For both the loading and unloading curves, the relative deformation d is calculated in reference to a permanent plastic deformation d_P. Or,

$$d = d_T - d_P$$

where d_T is the total deformation relative to the total displacements at the end node points of the element. Therefore, the deformations which occur during the loading in the plastic range are accumulated and the behaviour of the element is history dependent.

The Wen model.

In this case the hysteretic force f_1 is modelled as:

$$f_1 = f^{y} z \tag{3.28}$$

where f^{y} is the yielding force and z is an auxiliary variable defined by Wen in the following way:

$$\frac{dz}{dd} = A \pm (\nu \pm \gamma) z^{n} \tag{3.29}$$

where d is the translation in the direction of the motion and A, ν, γ are constants which describe a wide class of hysteretic cycles.

For friction elements, the pure friction force f_2 is defined by the following expression:

$$f_2 = -\mathrm{sgn}(d)\, \mu\, Q \tag{3.30}$$

where Q is the normal force to the friction surface.

The coefficient μ can be expressed by:

$$\mu = \mu_{max} - \Delta\mu\, e^{-\beta|\dot{d}|}$$
(3.31)

where β is a constant;
μ_{max} is the friction coefficient to a high displacement velocity;
$\Delta\mu = \mu_{max} - \mu_{min}$= cost, where μ_{min} is the friction coefficient to low displacement velocity.

3.2.2 Equivalent static models

Several equivalent models are currently in use for the study of the dynamic behaviour of structures. In the Uniform Building Code (UBC) of the United States of America, the equivalent static lateral force method is proposed for the earthquake-resistant design. This method assumes that the structure should be designed for a minimum total base shear force V given by the following formula:

$$V = \frac{ZIC}{R_w} W \tag{3.32}$$

where C is a factor which depends on the period of the building and the kind of ground:

$$C = \frac{1.25\,S}{T^{2/3}} \leq 2.75 \qquad (3.33)$$

with a minimum value for $C/R_w = 0.075$ except where scaling of forces by $3R_w/8$ is prescribed. Z is a factor related to the seismic zones of division of the USA; it varies between Z=0.075 for areas with less seismic risk to Z=0.4 for those with higher risk. S is the site coefficient depending on the characteristics of the soil at the site; its value varies between 1.0 and 2.0 in function of the kind of ground. Rock basements with a depth less than 200 ft have a value S=1.0; if the depth is more than 200 ft the factor S=1.2; for grounds with a depth higher than 70 ft and with more than 20 ft of medium clay but no more than 40 ft of soft clay S=1.5; for soft grounds with more than 40 ft of soft clay S=2.0. I is the occupancy importance factor related to the anticipated use of the structures: it is equal to 1.25 for buildings such as an hospital which must be operative soon after an earthquake, and equal to 1.00 for all other structures. R_w is a reduction design forces factor; it is a function of the ductility and the weight W of the building. It is a measure of the capacity of the structural system to absorb energy in the inelastic range through ductility and redundancy. Its value ranges from 4 to 12 and is based primarily on the performance of similar systems in past earthquakes. W is the seismic weight which includes the dead weight of the building and applicable percentage of other loads. T is the fundamental period of the building, which may be approximated from the following formula:

$$T = C_t\left[h_N^{3/4}\right] \qquad (3.34)$$

where h_N = total height of the building in feet;
C_t has different values depending on the type of building.

The base shear force V calculated with (3.19) is distributed at the various levels according to the following formulas:

$$F_x = \frac{(V - F_t)W_x\,h_x}{\sum_{i=1}^{N} W_i\,h_i} \qquad (3.35)$$

where F_t is a force applied at the last floor of the building, W_x is the weight of the floor at level x and h_x is the height from the ground. F_t models, in an approximated form, the participation of modes higher than the first and it is considered only when the fundamental period of the building is higher than 0.7 sec:

$F_t = 0.07\,TV \leq 0.25\,V$ for $T > 0.7$ sec
$F_t = 0$ for $T \leq 0.7$ sec

and, at the base,

$$V = F_t + \sum_{i=1}^{N} F_i \qquad (3.36)$$

where:

N = total number of stories above the base of the building;
F_x, F_i, F_N= lateral force applied at level x, i, or N;
F_t= portion of the base force V at the top of the structure in addition to F_N ;
h_x, h_i= height of level x or i above the base;
W_x, W_i= seismic weight of x-th or i-th level.

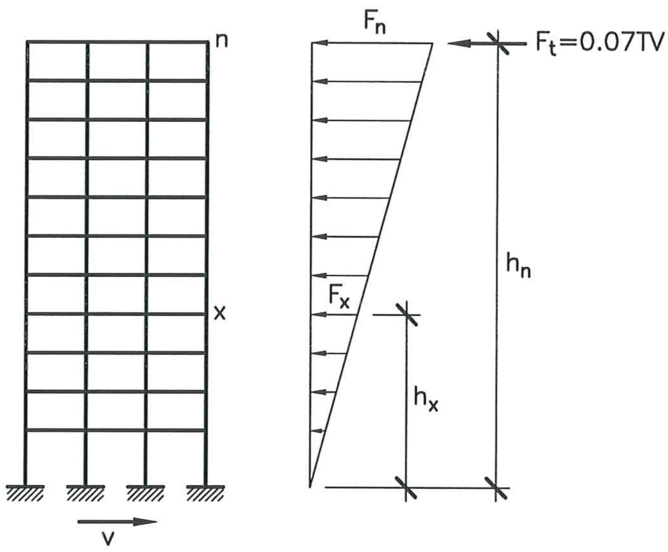

Figure 3.3. Distribution of lateral forces in a multi-story building.

The distribution of the lateral forces F_x and F_t in a uniform multi-story building is shown in Figure 3.3. The UBC-91 code stipulates that the force F_x at level x be applied over the area of the building according to the mass distribution at that level. It is assumed that the shear force distribution in height is considered regular for buildings which show a decrement of the plan dimensions at each floor less than the 25% of the base dimensions.

3.3 SCOPE OF THIS STUDY

The present study is intended to assess the efficacy of energy dissipators in the seismic protection of buildings from an experimental point of view.

Shaking-table tests have been performed on a steel model passively protected with a kind of friction energy dissipator at the Applied Geophisics Laboratory of the Technical University of Catalonia, Barcelona, Spain. The description and the results of these tests are shown respectively in Chapter 4 and 5.

Chapter 1 deals with a general description of structural control, while Chapter 2 presents a state-of-the-art of Energy Dissipation systems.

As conclusion from the dynamic tests it appears that energy dissipators provide an effective and reliable method for seismic resistant design. This study also offers the basis for further research in this field.

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CHAPTER 4

Experimental Tests. Description

4.1 GENERAL

In this chapter the efficacy of EDDs in the seismic protection of buildings is analysed from an experimental point of view by performing a series of shaking-table tests on models protected with two different kinds of devices. A series of tests have been performed on a reduced scale steel model protected with a friction dissipator.

The energy dissipators utilised in this model principally involve steel-steel friction to dissipate energy and reduce the seismic response. They are similar to the simple Slotted Bolted Connection described in 2.3.2.

Experimental tests have been performed on a shaking table in the Applied Geophisics Laboratory of the Technical University of Catalonia, Barcelona, Spain. The shaking-table measures in plan 1.50m x1.50m and has tri-axial motion.

The model is a 3-D steel moment-resisting frame with 5 stories. It is 170 cm high, 144 cm wide and 77 cm deep. Along one direction there are two bays and in the other direction there is one, (Figure 4.1).

The model allows changes in its stiffness and mass and has the possibility to incorporate different passive control devices. It was instrumented with accelerometers on each floor to measure directly the time-response accelerations. In the Appendix, Photo 1 shows the 5 story model installed on the shaking table.

The work has been divided in three parts. The first part deals with the design of the model and the characterisation of the experimental model utilising push-over inputs and random vibrations. The natural periods and equivalent viscous damping ratios were obtained. In the second part a series of earthquake simulator tests were performed on the model with and without energy dissipators. Results were determined comparing the time-response and frequency-response plots. In the third part a numerical analysis has been carried out on the model subjected to El Centro register.

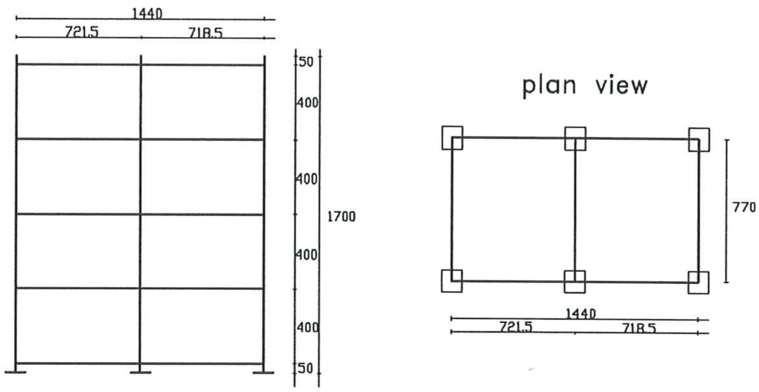

Figure 4.1. Vertical and plan sections of the model.

4.2 TEST STRUCTURE

The model consists of 6 columns and 35 beams made of F111 steel (σ_y=2300 Kg/cm2; σ_u=4000 Kg/cm2). The columns are continuous L-shape 20mmx20mmx2mm profiles and the beams are 20x10mm rectangular bars. The length of each beam is approximately 72 cm for the two-bay frames, and 77 cm for the one-bay ones. The diagonals are 20x12mm rectangular bars.

The static characteristics of the columns, the beams and the diagonals are shown in Table 4.1.

ELEMENT	SECTION AREA [cm2]	INERTIA [cm2]	SECTION WEIGHT [Kg/cm]
COLUMN	0.76	0.288	0.0059
BEAM	2.00	0.167	0.0157
DIAGONAL	2.40	0.288	0.01824

Table 4.1. Structural characteristics of the elements.

The dimensions of the elements verify all the resistance and stability conditions under a vertical concentrated static load of 4.5 Kg on each beam and a horizontal equivalent static load determined from the Uniform Building Code (UBC) 1991.

The column-beam-diagonal connection was an L-shape 50mmx50mmx2mm steel plate welded to the beam. In this way it is possible to utilise different kinds of beams and diagonals to change the stiffness and mass of the model, (see Figure 4.2). In the Appendix some photos of the node connections are shown.

Approximately 4.5 Kg of lead weight was added to each beam. The added load consisted of eight 200mmx50mmx1mm lead straps connected to the model with a steel plate and four bolts (Figure 4.3).

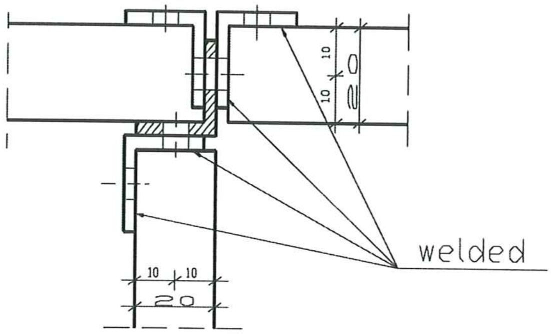

Figure 4.2. Column-beam-diagonal connection of the central column.

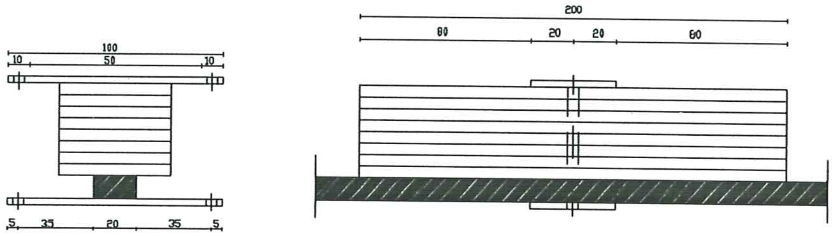

Figure 4.3. Design of the added load on each column.

4.3 FRICTION ENERGY DISSIPATORS

The energy dissipators utilised in the tests are friction-type dissipators. They dissipate energy through pure friction. Figure 6.4 shows the vertical section and a plan view of the dissipating devices utilised in the experimental model. The F-δ behaviour is a rigid-perfect plastic one as shown in Figure 4.4. Photo 7 and 8 in the Appendix show a single dissipator and Photo 9 shows the dissipators installed in the model.

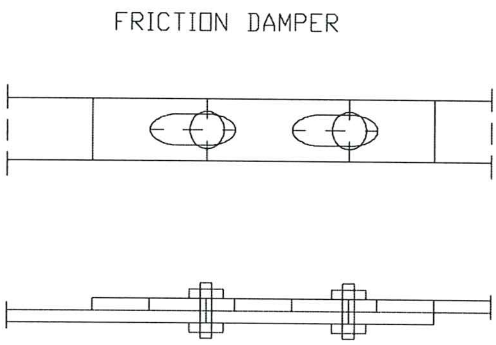

Figure 4.4. Plan view and vertical section of the friction damper.

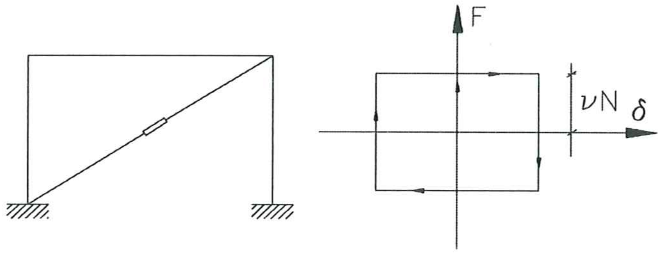

Figure 4.5. F-δ behaviour of a friction dissipator utilised in the test model.

4.4 TEST FACILITIES

The shaking table has a 1.50 x 1.50 m test-bed. It stands a maximum weight of 1000 Kg. The table and the added weight were supported on four adjustable dampers of air pressure. The table has three degrees of freedom: two horizontal and one vertical. It has an allowable frequency range between 0.0 Hz and 12.5 Hz and a maximum displacement of ± 3.8 cm.

4.4.1 Instrumentation

The actuators which drive the table are electrical engines Siemens with 370 V, designed for machine control. There were three engines for each degree of freedom of the shaking table. The engines were controlled by a numerical Siemens control system specific for the regulation of the electrical engines.

Accelerometers of the type 4384 from B&K were attached to each floor and to the base; only one accelerometer was put in correspondence of the fifth floor for the push-over tests on the bare and braced frames.

4.5 TEST PROGRAM

Tests have been performed on the frame without diagonals (bare frame), on the frame with a diagonal added between the second and third floors of one bay (braced frame), and on the frame with dissipators in substitution of the diagonals (protected frame), (Figure 4.6).

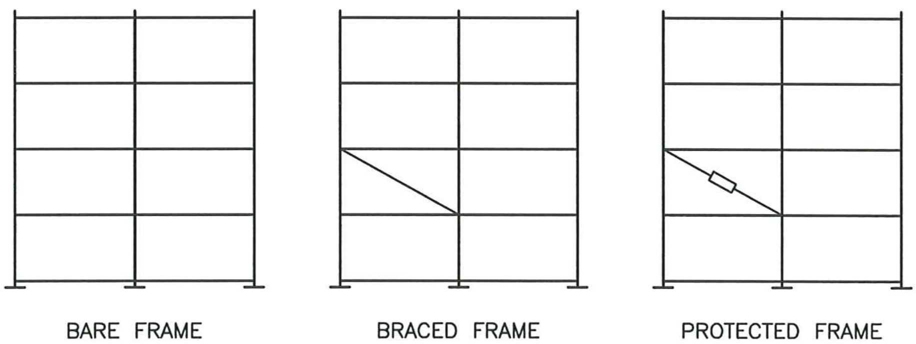

Figure 4.6. The three basic structural configurations.

4.5.1 Table motions

Preliminarily a series of push-over inputs (utilising a hammer) and random signals have been utilised to characterise the structure.

A series of shaking table tests were performed on the model utilising a scaled register of a real earthquake. In this case a synthetic record with the same spectra as El Centro 1940 earthquake has been utilised as seismic signal. The input consisted of a horizontal component in the direction parallel to the longitudinal frames. No rotational excitations were applied to the structure. This earthquake was chosen because has a relatively wide range of frequencies (between 1 and 5 Hz), and consists of about five significant cycles. It is commonly used by structural engineers and is considered a typical California earthquake record. The acceleration time history and its Fourier Transform are shown in Figure 4.6.

4.5.2 Test sequence

1) Push-over test for the braced frame. A diagonal was installed between the second and the third floor of the two-bay frames.
2) Random vibration for the braced frame. The base peak acceleration was about 5mm/sec2.
3) El Centro register. Scale factor S=1.5. Braced frame.
4) Random vibration. Braced frame. Peak acceleration ≅ 5 mm/sec2.
5) Random vibration. Bare frame. Peak acceleration ≅ 5 mm/sec2.
6) El Centro register. Scale factor S=1.5. Bare frame.
7) Random vibration. Protected frame. The dissipators are installed in substitution of the diagonals. Peak acceleration ≅ 5 mm/sec2.
8) El Centro register. Scale factor S=1.5. Protected frame.
9) El Centro register. Scale factor S=2.1. Protected frame.

The main objective of the experimental analysis was to study the efficacy of the friction EDDs when a structure is subjected to base motions.

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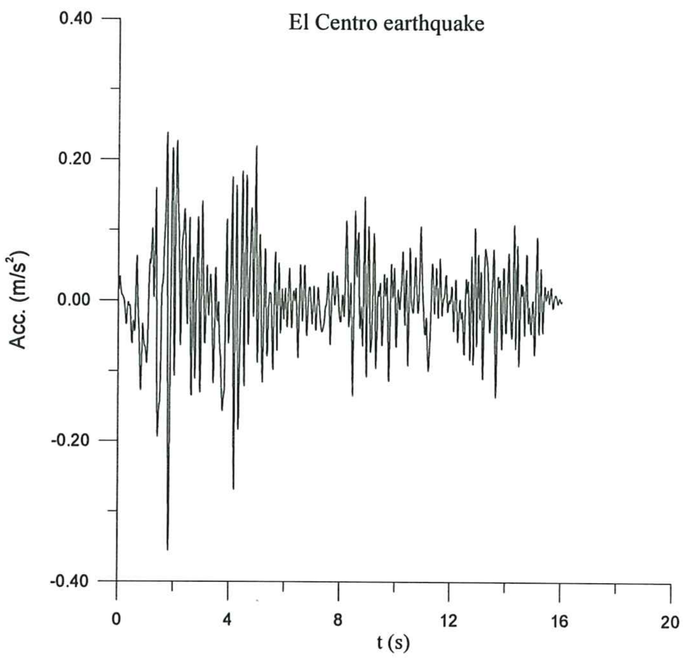
(a)



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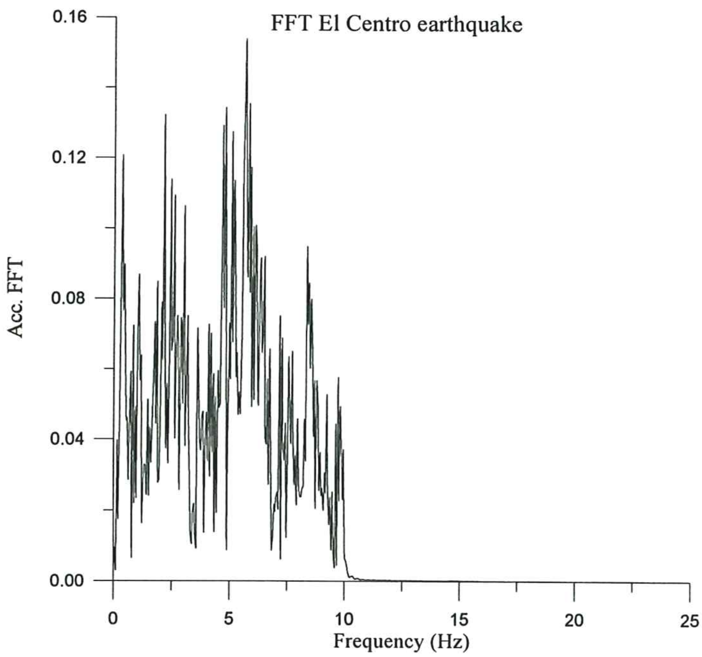


Figure 4.6. El Centro register. (a) Acceleration time history and (b) Fourier Response.

CHAPTER 5

Test Results

5.1 PUSH-OVER TEST RESULTS

A series of push-over tests have been performed on the bare and braced frames with an instrumented hammer in three different positions to perform a bending test, a torsion test and a lateral torsion test. (Figure 5.7).

The frames were instrumented with one accelerometer on the fifth floor.

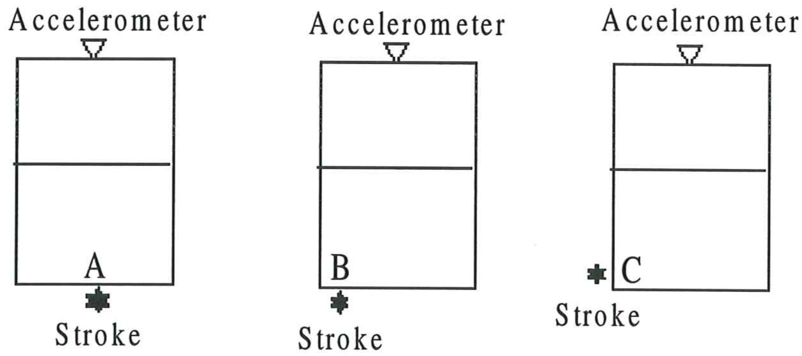


Figure 5.7. Position of the strokes and of the accelerometers. Plan view. (A) bending test; (B) torsion test; (C) lateral torsion test.

In case of bending test, with a stroke given in position A a comparison between bare and braced frames has been done. From the time-response diagrams it is possible to notice an amplification for the braced frame of 1.5 times bigger than in case of bare frame (Figures 5.8).

In Figures 5.9 the FFT diagrams are plotted for the two cases of bare and braced frames.

Figures 5.10÷5.13 show the time-response and FFT plots referred to the torsion tests (position B of the stroke) performed on the braced frame only and lateral torsion tests (position C of the stroke). From Figure 5.12 the amplification produced by the braced frame is even 4.78 times bigger than the amplification produced by the bare frame.

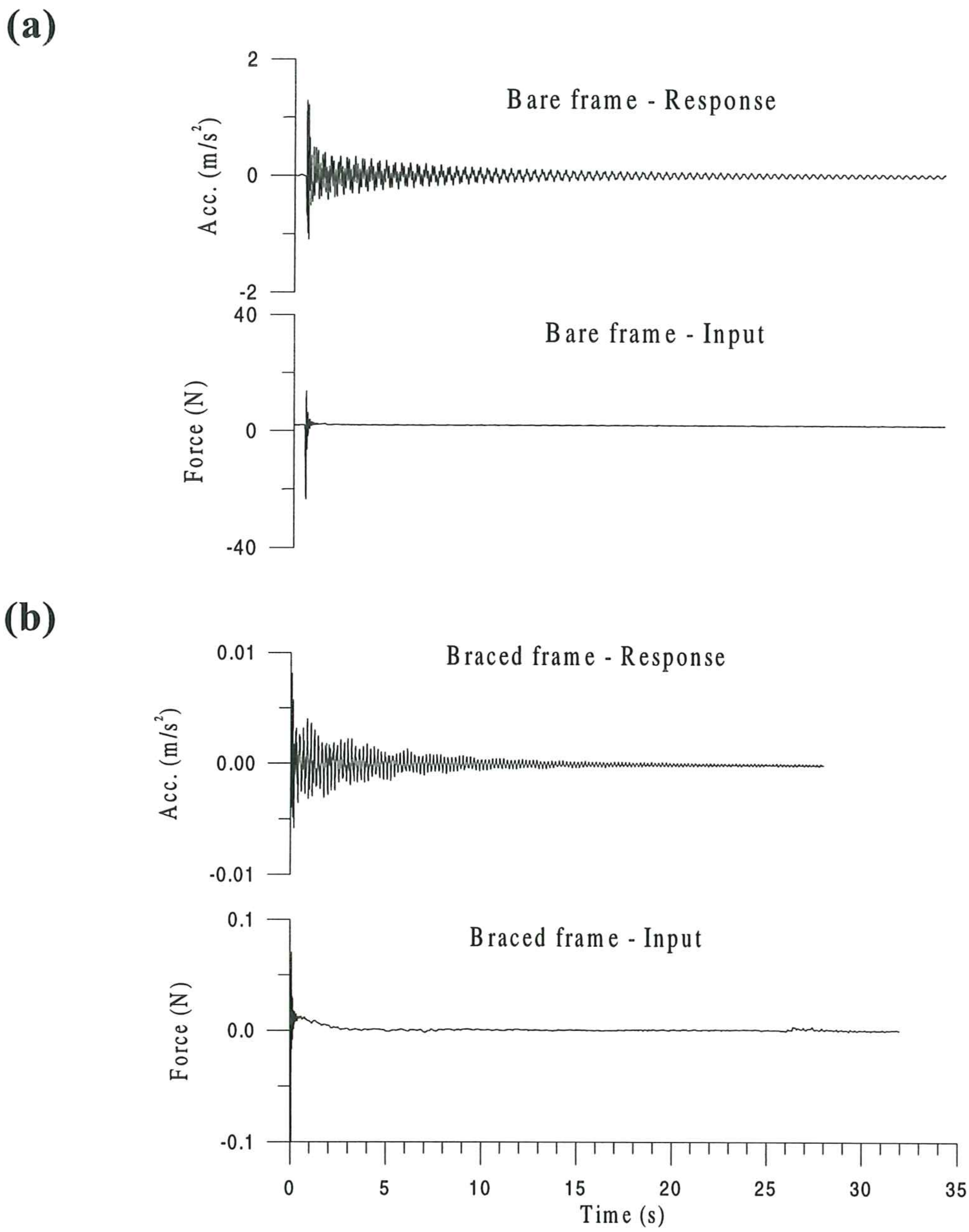


Figure 5.8. Time-response acceleration diagrams for (a) the bare frame and for (b) the braced frame. Bending test.

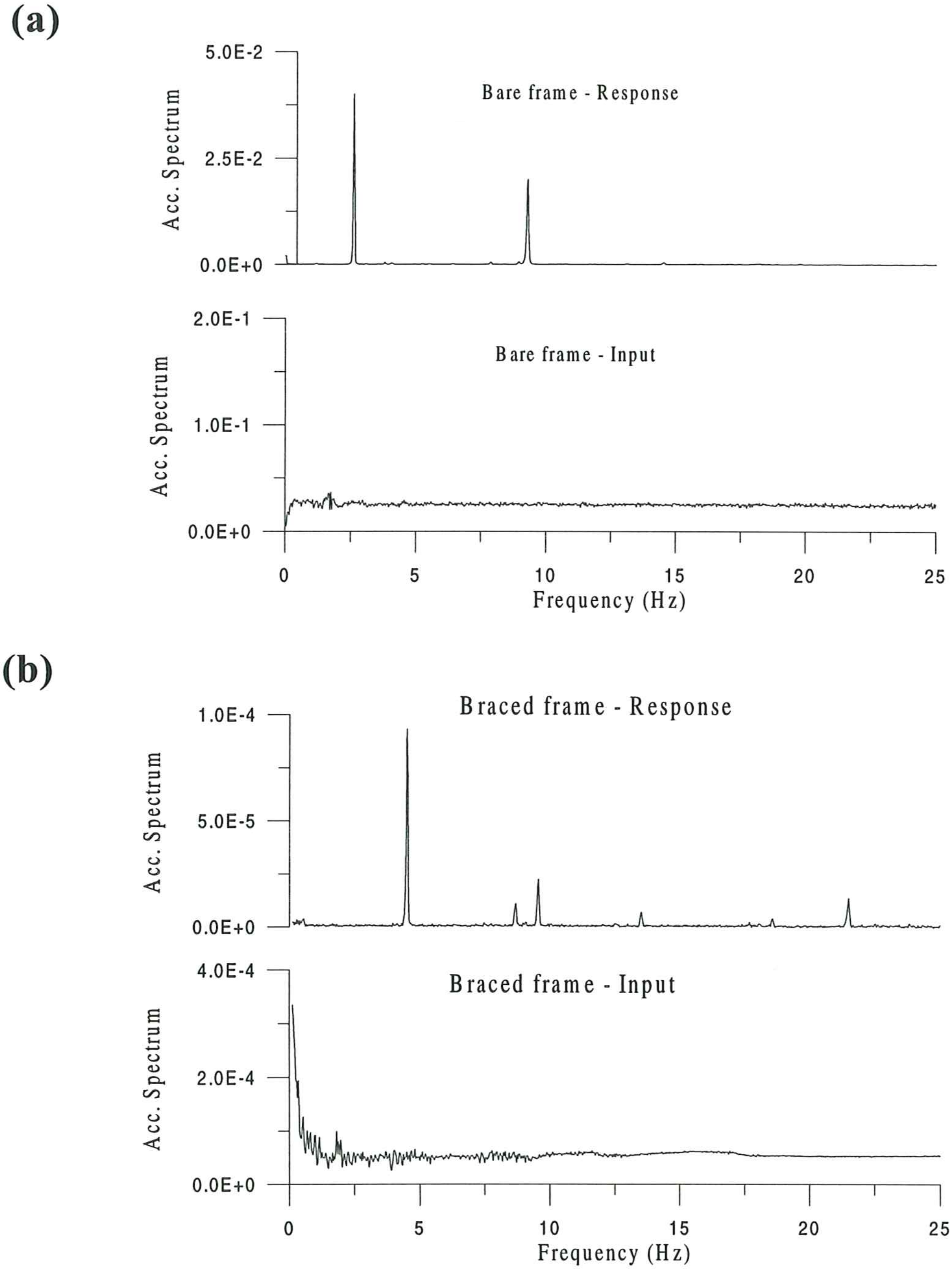


Figure 5.9. Frequency-response acceleration diagrams for (a) the bare frame and for (b) the braced frame. Bending test.

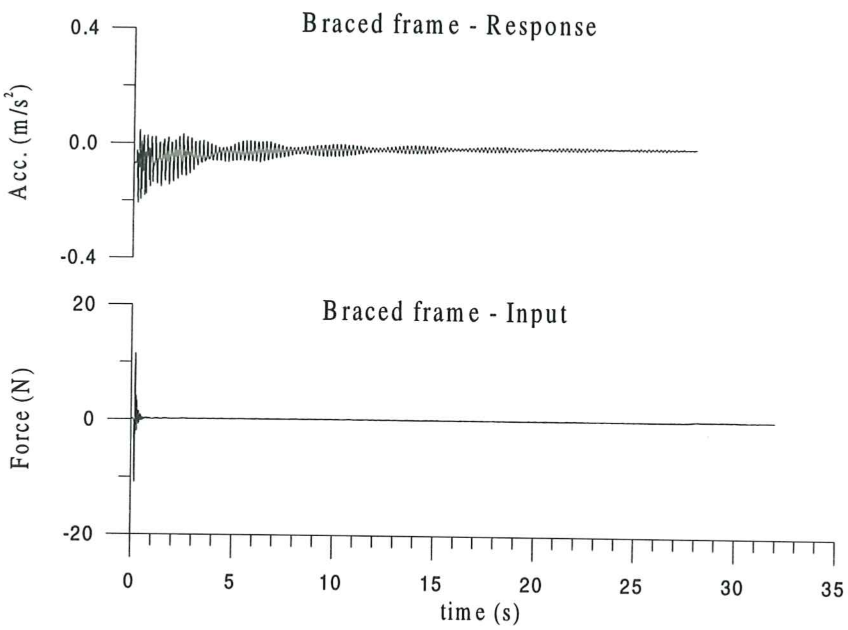


Figure 5.10. Time-response acceleration diagrams for the braced frame. Torsion test.

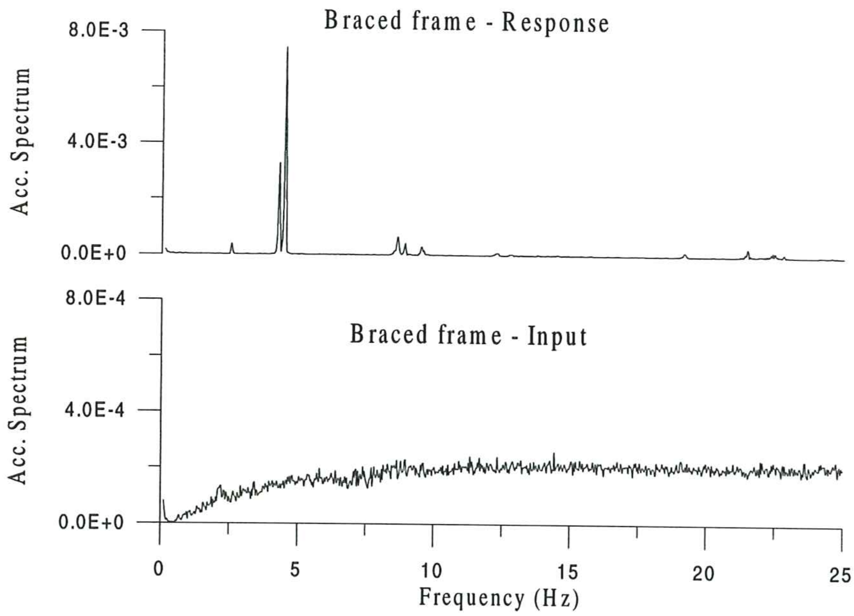


Figure 5.11. Frequency-response acceleration diagrams for the braced frame. Torsion test.

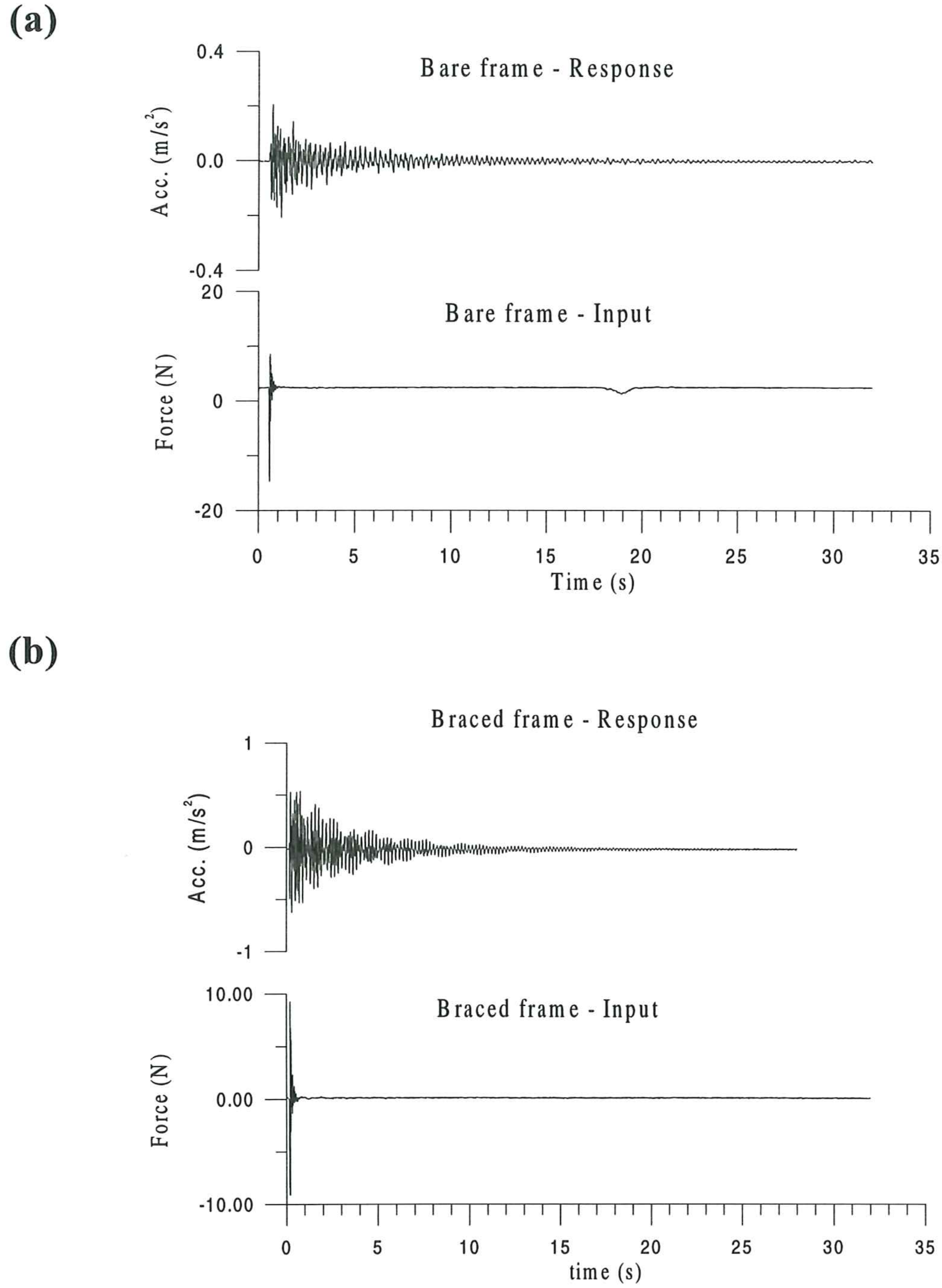
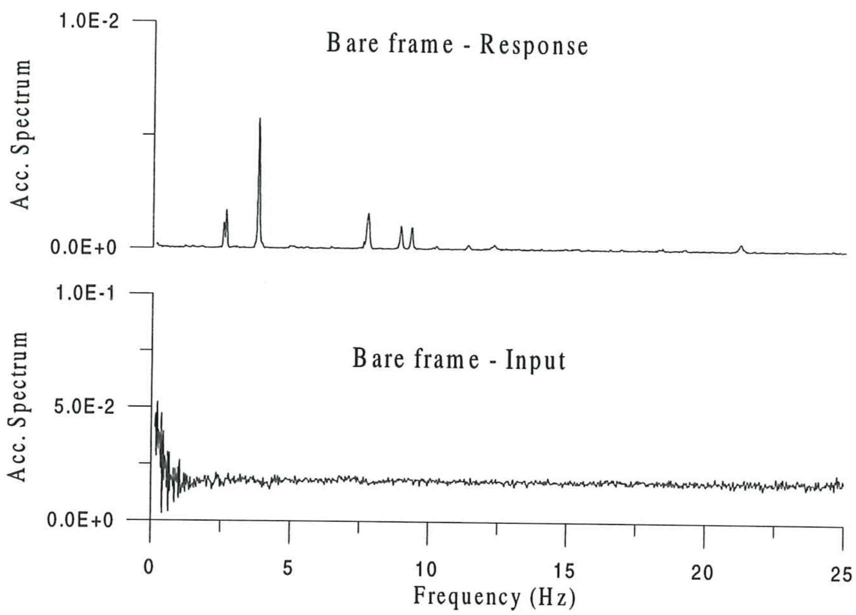


Figure 5.12. Time-response acceleration diagrams for (a) the bare frame and for (b) the braced frame. Lateral torsion test.

(a)



(b)

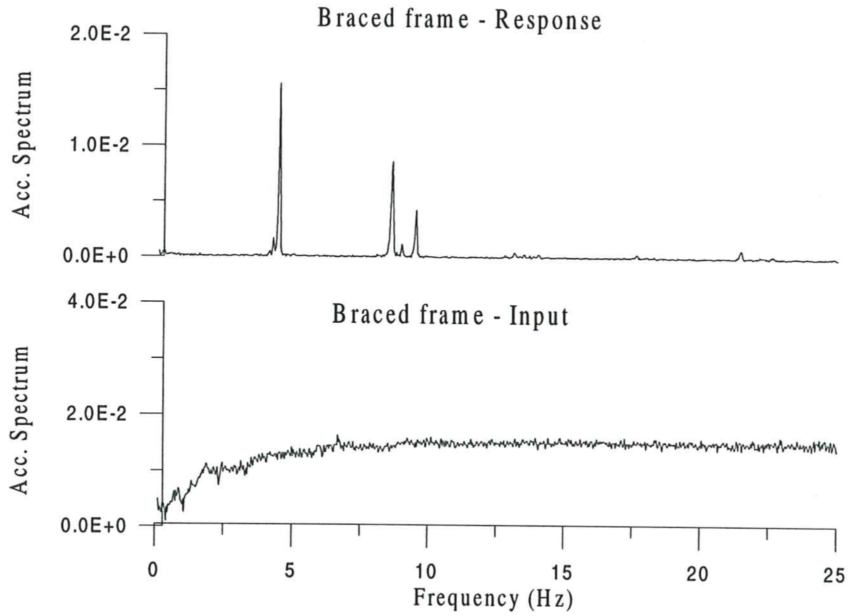


Figure 5.13. Frequency-response acceleration diagrams for (a) the bare frame and for (b) the braced frame. Lateral torsion test.

5.1.1 Natural frequencies

The experimental model considered as a shear-type frame has 5 degrees of freedom, one corresponding to each story. In reality other modes of vibration are present due to rotation effects caused by different reasons such as the lack of a plane rigid diaphragm at each floor level; different stiffnesses of the column-beam connections as a function of the bolting force, and so on.

The first two natural frequencies of the shear-type models were determined from the Fourier Spectra of the acceleration time-response diagrams of the push-over tests. This method was accurate and easy to conduct. The range of frequencies for the table was between 0 Hz and 12.5 Hz, therefore only the first two natural frequencies of the structure were determined:

	T1 [sec]	ω1 [Hz]	T2 [sec]	ω2 [Hz]
Bare frame	0.3095	2.625	0.10841	9.25
Braced frame	0.222	4.5	0.10458	9.5625

Table 5.1. Experimental natural frequencies and periods.

In Table 5.1 the first two experimental natural frequencies and periods for the bare and braced frames are shown.

5.2 RANDOM VIBRATION TEST RESULTS

The main objective of the experimental analysis was to study the response of a structure subjected to base motions. A series of random vibration tests have been carried out on the model to characterise it and to confirm the values of the natural frequencies and damping factors determined from the previous push-over tests.

The signal was a random noise with a broad-band Fourier spectrum. It was possible to determine the dynamic characteristics of the bare, braced and protected models and to evaluate the effectiveness of the analytical methods for predicting these quantities.

From the test sequence the random vibration tests correspond to test 2, 4, 5 and 7. Results will be shown only for test 2, 5 and 7.

The accelerometers were put on the table and on the 5th floor only, to compare the input-output results.

Directly from each test was possible to obtain a time-acceleration response, the autospectrum, the instantaneous spectrum and the transfer functions H1 and H2, (Figures 5.14÷5.22).

The instantaneous spectra are the FFT of the response at the 5th floor of only one acquisition. It is possible to determine the vibration frequencies of the models. From these plots it is not possible to evidence the frequencies of the shear-type models only because rotational and torsional modes are present too. For the protected frames an amplification of the response appears around 2.09 Hz, 4.16 Hz and 9.5 Hz.

The autospectrum is defined as the one-sided real-valued ensemble average of the squared magnitude of the one-sided instantaneous complex spectrum while the functions H1 and H2 are explained in the following.

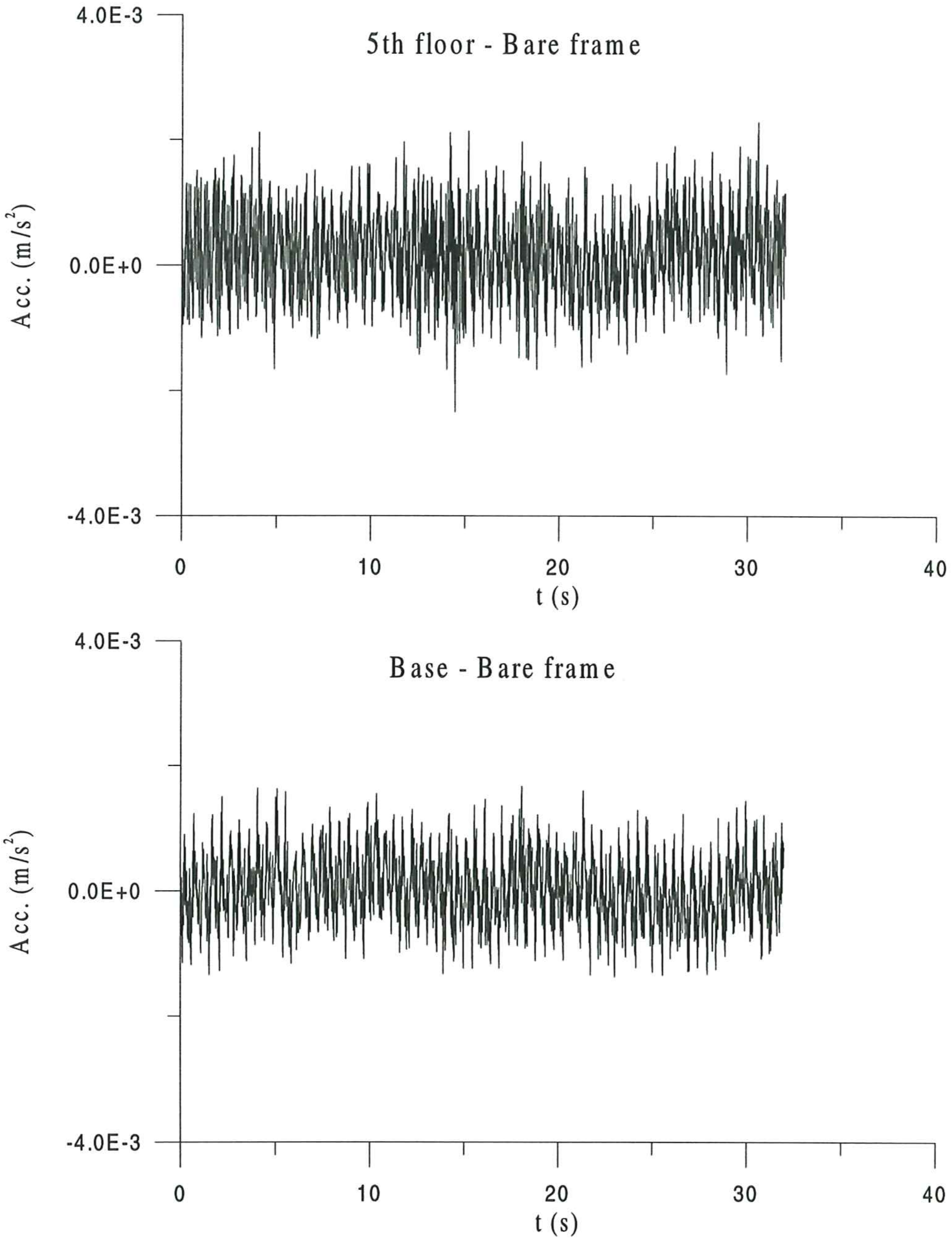


Figure 5.14. Time-response acceleration diagrams for the bare frame.

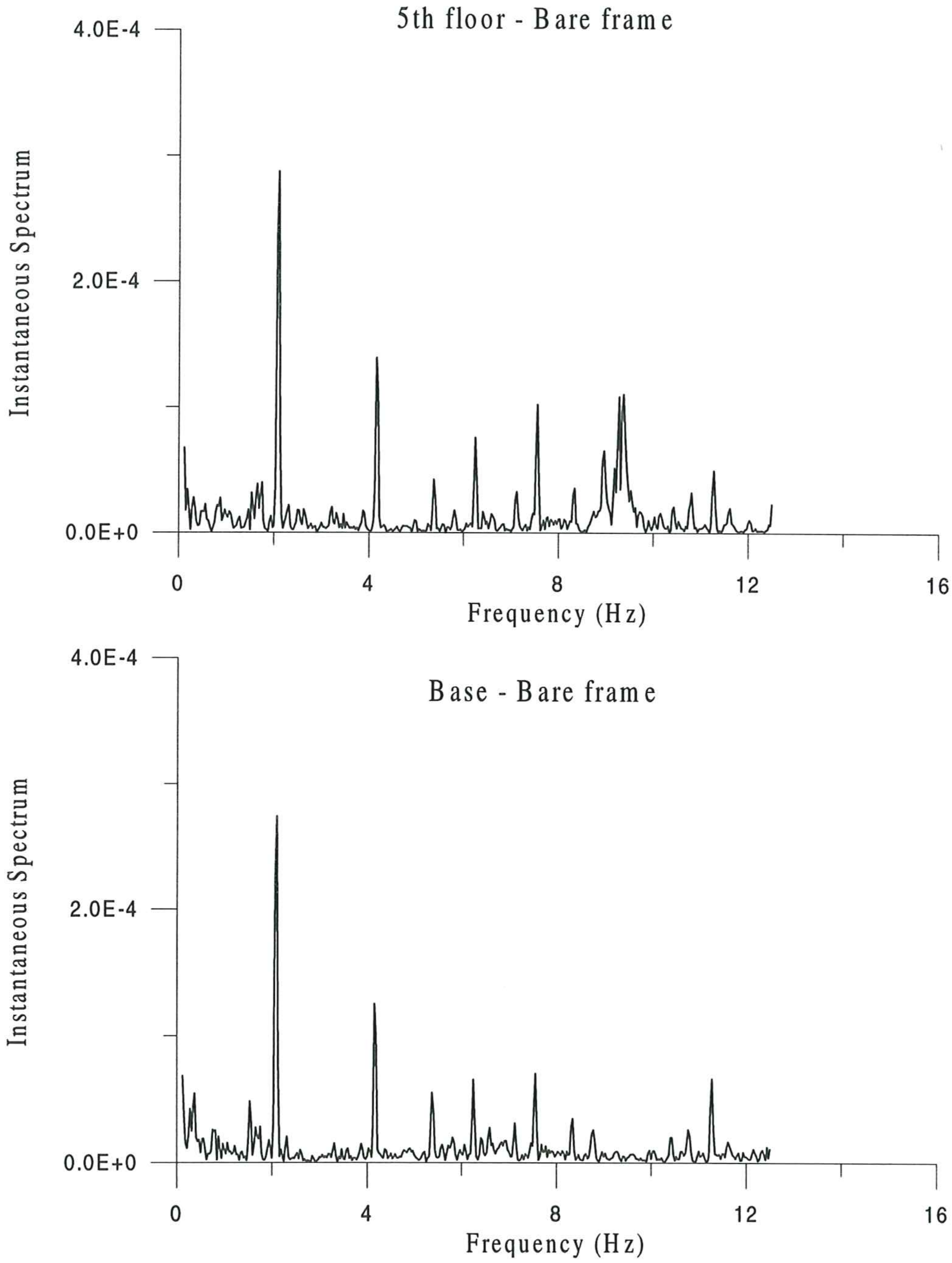


Figure 5.15. Frequency-response acceleration diagrams for the bare frame.

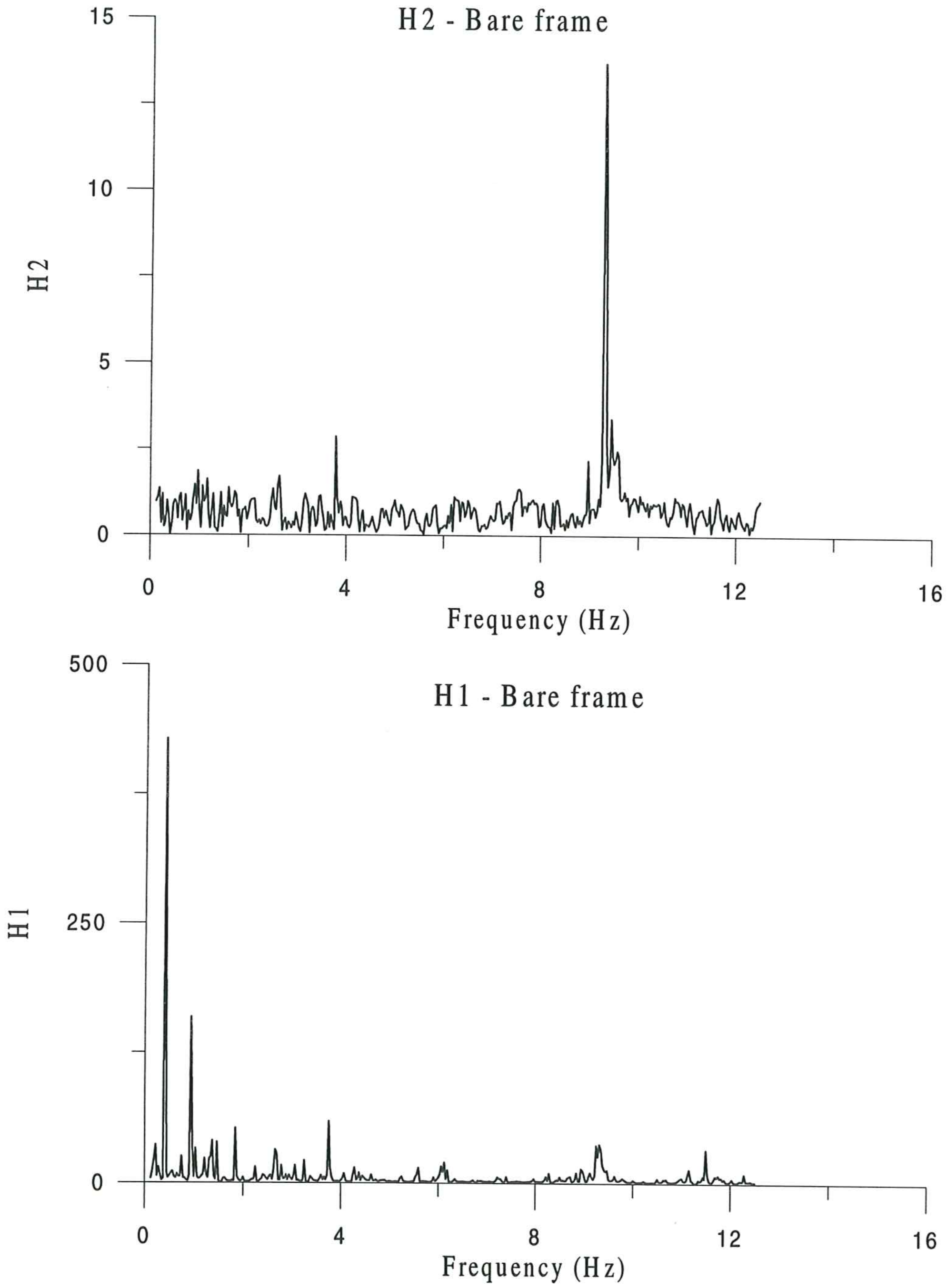


Figure 5.16. H1 and H2 diagrams for the bare frame.

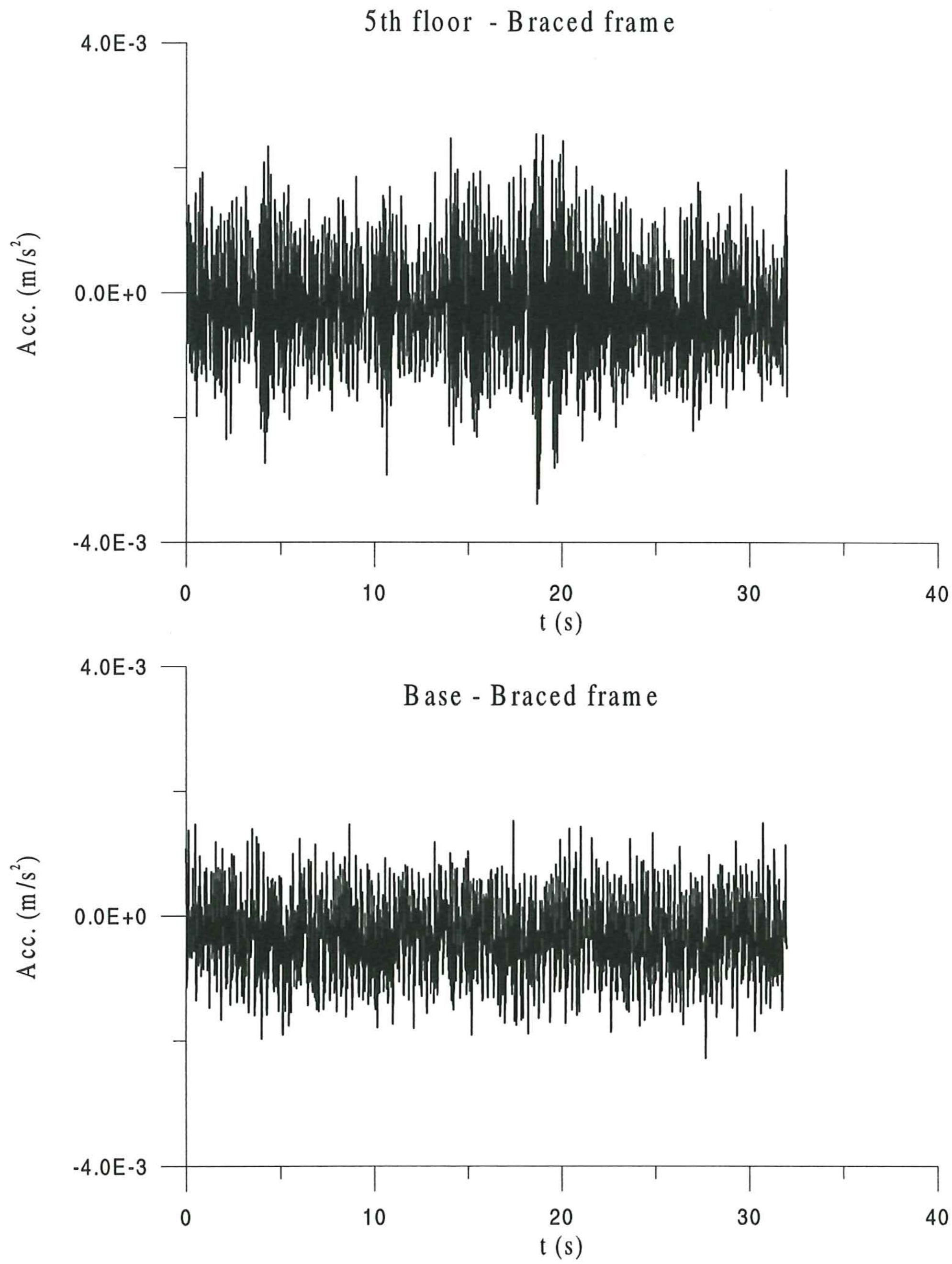


Figure 5.17. Time-response acceleration diagrams for the braced frame.

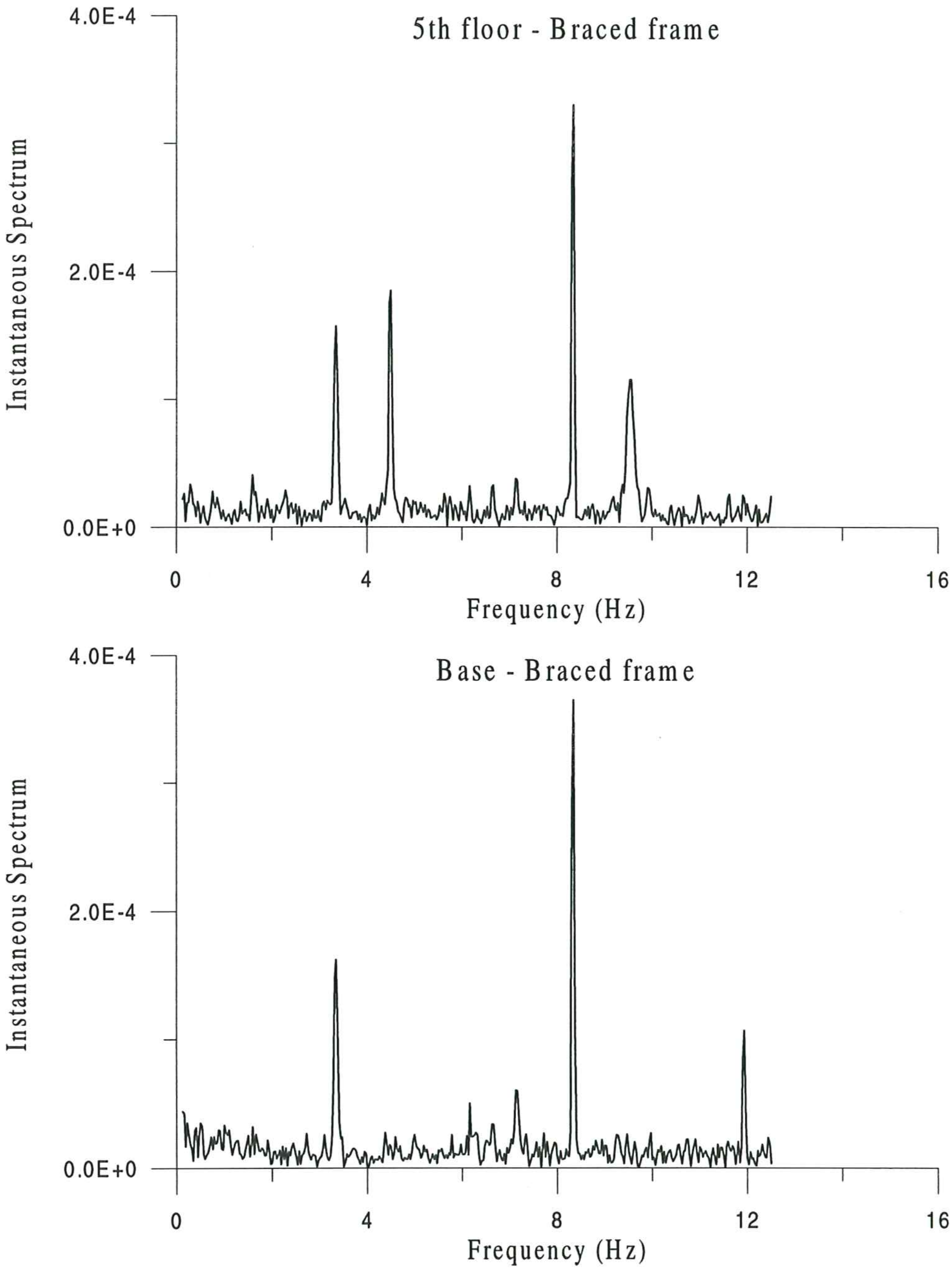


Figure 5.18. Frequency-response acceleration diagrams for the braced frame.

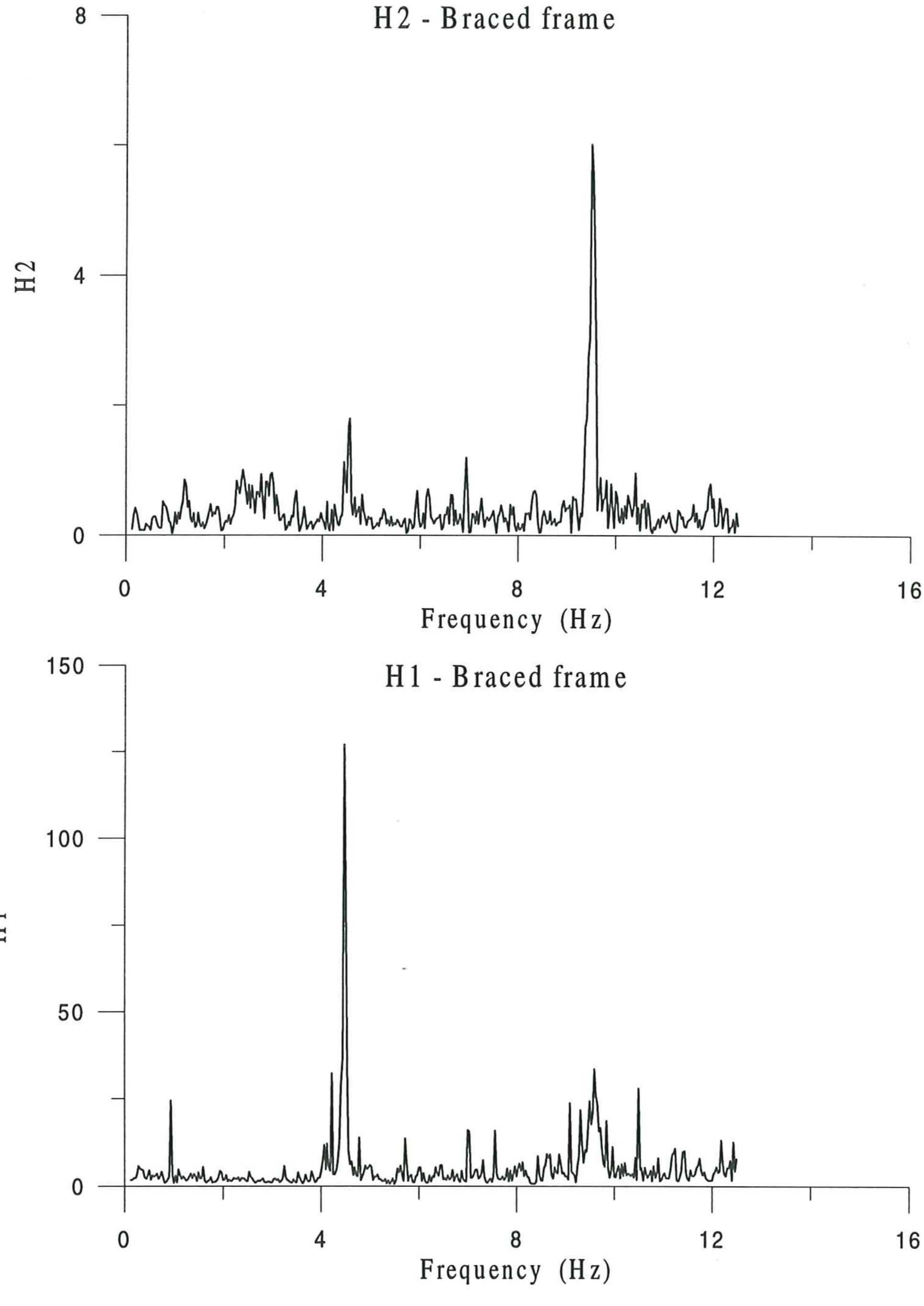


Figure 5.19. H1 and H2 diagrams for the braced frame.

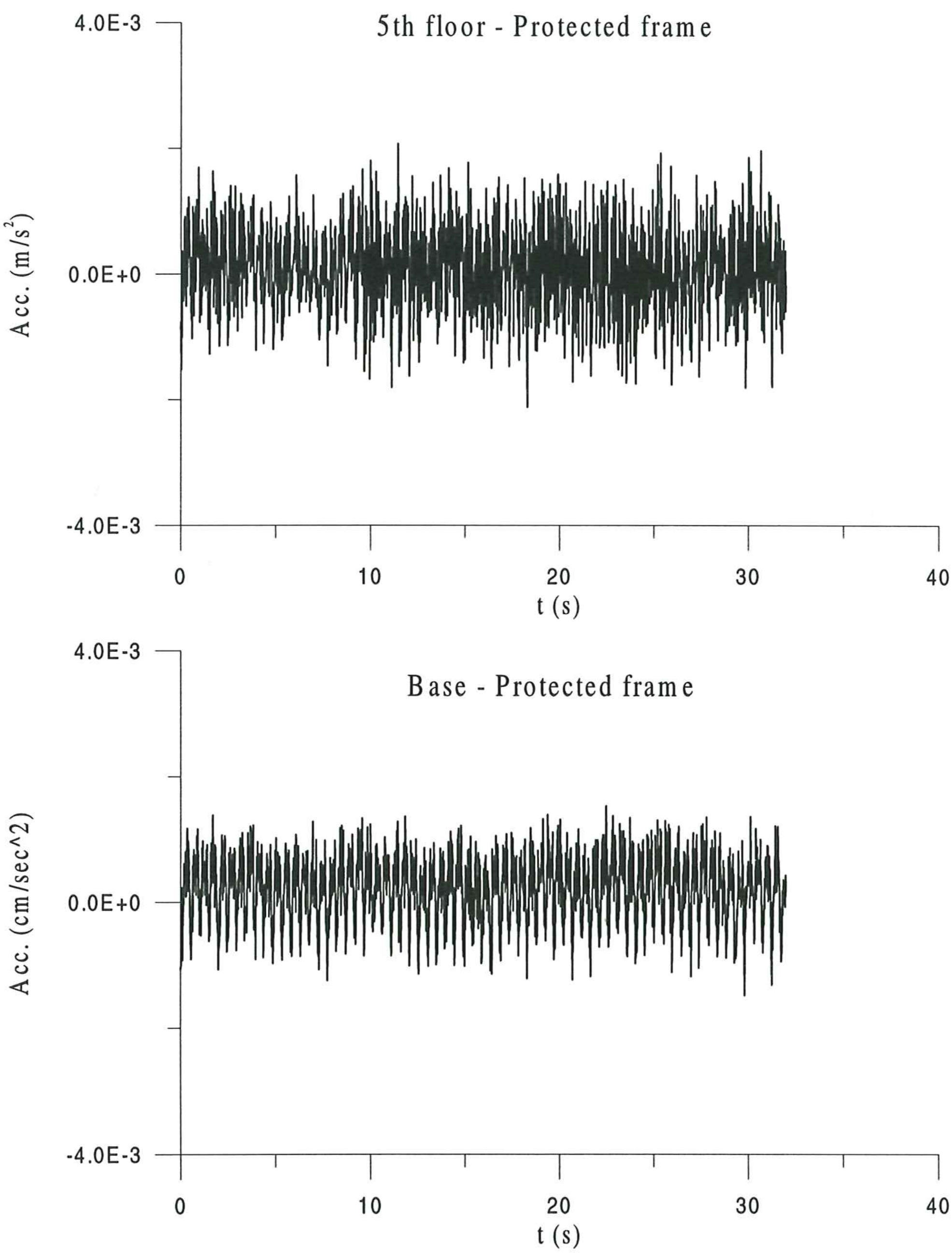


Figure 5.20. Time-response acceleration diagrams for the protected frame.

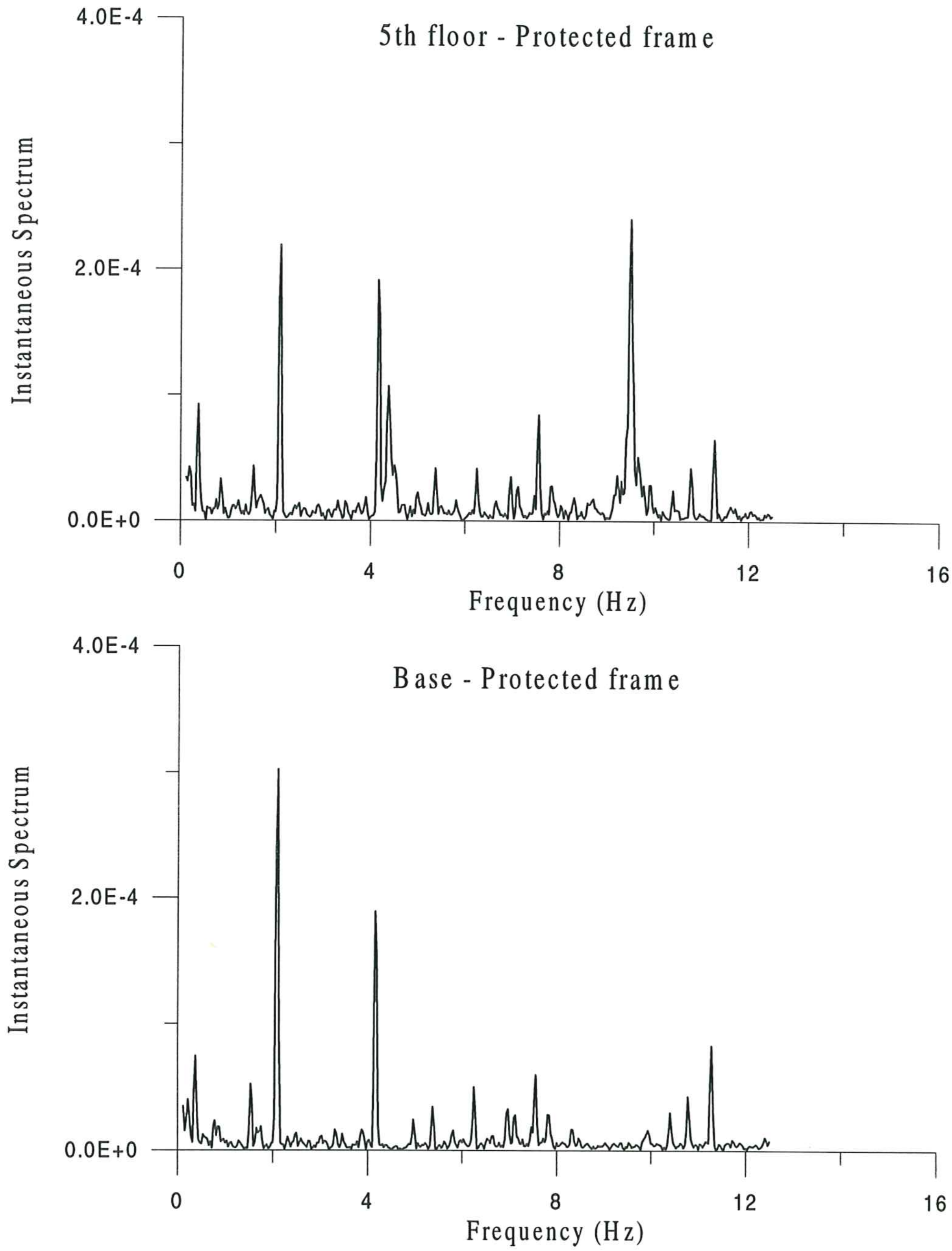


Figure 5.21. Frequency-response acceleration diagrams for the protected frame.

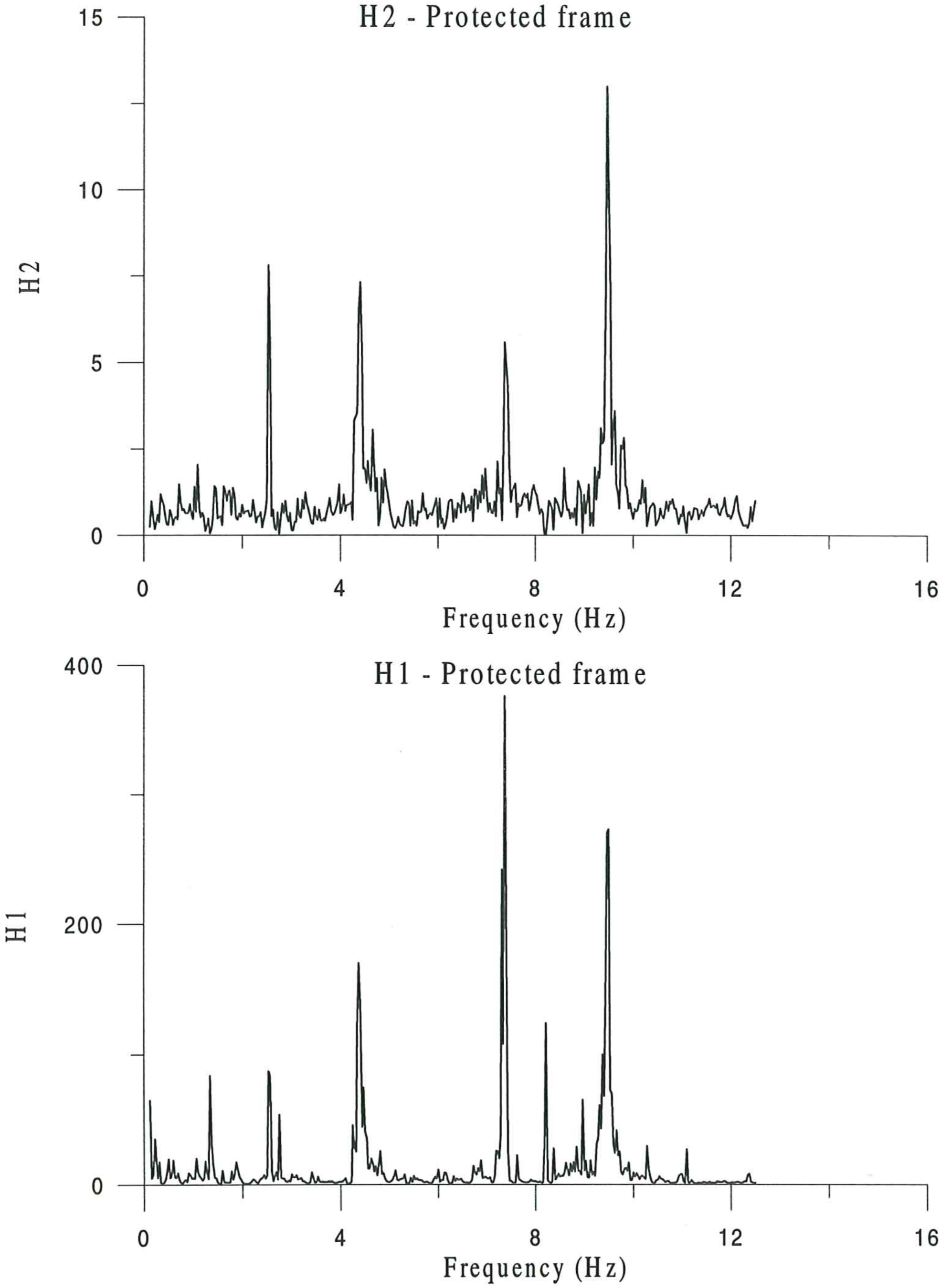


Figure 5.22. H1 and H2 diagrams for the protected frame.

5.2.1 Transfer function H(ω)

Usually a structure is modelled such as a multi-degree-of-freedom linear oscillator which receives an input signal x which is transformed in an output signal y in the last floor. Therefore the frame is considered like a filter characterised by its transfer function H(ω):

$$x \rightarrow H(\omega) \rightarrow y$$

being its mathematical expression in the frequency domain:

$$\mathbf{H}(\omega) = \frac{\mathbf{Y}(\omega)}{\mathbf{X}(\omega)}$$

X(ω) and Y(ω) are Fourier Transforms of the signals x(t) and y(t) respectively.

There are many different estimators for H(ω). Armini [1990] makes a comparison among some of them. For this research the functions H1 and H2 described by Ewins [1994] and Brüel & Kjær have been utilised:

$$H1 = Sxy/Sxx \qquad H2 = Syy/Syx$$

where: Sxx is the input autospectrum,
Syy is the output autospectrum,
Sxy is the cross spectrum.

According to this definitions and through an averaging process, H1 suppresses uncorrelated noise at the output and gives the most correct result at low levels (antiresonances) when the output signal is low, and H2 suppresses uncorrelated noise at the input and gives the most correct result at high levels (resonance peaks) when the input signal is low.

5.2.2 Transmissibility

From the random noise tests it was possible to determine the Fourier spectra of the response acceleration/input acceleration (Transmissibility or TR curves) of the fifth floor for the bare and braced frames (Figure 5.23) utilising the autospectra diagrams).

From the transmissibility plots it is clear that the amplification of the ground motion is very high in a structure when the frequency of the ground shaking coincided with any of the natural frequencies of the structure.

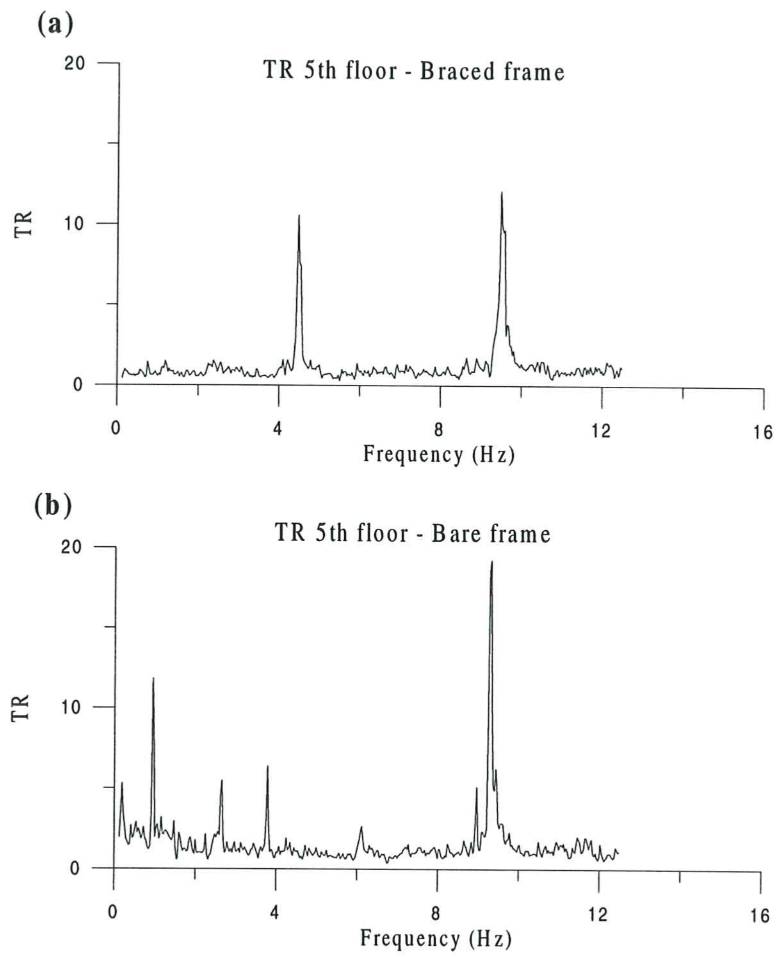


Figure 5.23. Transmissibility curve for the fifth floor for (a) the bare and for (b) the braced frames.

5.2.3 Modal damping

The modal damping ratios were obtained by using the Half-Power (Bandwidth) Method and the transmissibility curves for the fifth floor. The sharp and narrow shapes of this curve in the resonant frequency range made it difficult to calculate the modal damping ratios.

The damping factor (ξ %) for the bare and braced frames are shown in Table 5.2.

Damping factor [%]	Bare frame	Braced frame
ξ1	1.818 %	0.926 %
ξ2	0.354 %	0.690 %
ξ3	0.446 %	0.227 %

Table 5.2. Damping factors for the bare and braced frames.

Since the damping ratio for the bare frame is small in all the modes higher than the first, for practical considerations it can be assumed this ratio as constant and equal to 0.4%. For the braced frame the values of ξ for the three modes are too different to be unified to a single value.

5.3 EARTHQUAKE SIMULATOR TEST RESULTS

It is commonly accepted that the total energy transmitted from the ground to a structure is absorbed in two different ways. The first one is due to the elastic strain, the second one is due to the plastic strain. The elastic strain energy is temporarily absorbed by the structure, part of it is transmitted back to the soil through the foundations and the other part released as kinetic energy causing amplification in the response quantities. For this reason, conventional design has a tendency to dissipate the major amount of the total energy absorbed by means of inelastic behaviour of structural and non-structural elements. The need to increase the dissipated energy is satisfied by the use of friction dampers. It is shown here that the major portion of the total energy absorbed was dissipated by friction, minimising the amplification of the response.

In case of earthquake tests the damping ratios increase somewhat from the push-over tests; this is because the table is basically inactive in the pulse tests after an initial displacement, whereas the active table in the earthquake tests contributes to the measured damping via the hydraulic actuator system.

From the test sequence the earthquake simulator tests correspond to test 3, 6, 8 and 9 respectively for the braced frame, bare frame and protected frames (S=1 and S=2.1).

In the following the time-history and frequency response acceleration plots are shown for each floor and for the table. It is expected to find out a reduction of the acceleration transmitted by the table to the structure. That is the main purpose of a passive protection system. The peak acceleration to peak ground acceleration ratios are shown for each floor in Table 5.3.

	A^{1st} floor/Atable	A^{2nd} floor/Atable	A^{3rd} floor/Atable	A^{4th} floor/Atable	A^{5th} floor/Atable
El Centro S=1 Braced frame.	0.983	2.4118	2.7284	8.391	12.41
El Centro S=1 Bare frame.	1.122	2.883	3.810	2.883	4.968
El Centro S=1 Protected frame.	1.099	1.588	1.783	1.358	2.742
El Centro S=2.1 Protected frame.	0.930	1.366	1.742	1.192	2.030

Table 5.3. Acceleration amplification at different floors.

From Table 5.3 the maximum amplification ratio is found for the last floor in the braced frame, while the maximum value for the protected structure is determined at the fifth floor under a register with a scale factor S=1. When the input is stronger (S=2.1), the top amplification reduces; it is even more than six times reduced respect to the amplification obtained for the braced frame. That confirms that the friction dissipators are working dissipating a great amount of energy.

Figures 5.24÷5.31 show the acceleration time histories for the table and the five stories, for the braced, bare and protected structures. They illustrate the reduction in acceleration when the model is protected with friction dissipators. When the structure was tested as braced frame, the peak acceleration for El Centro register with a scale factor S=1 reached a value of 1.58 cm/sec^2 at the 5th floor. In case of bare frame the maximum acceleration is registered at the 5th floor with a value of 1.16 cm/sec^2. For the protected frame subjected to El Centro register with a scale factor S=1, the peak acceleration is reached at the 5th floor and is equal to 1.05 cm/sec^2. It is possible to notice a reduction of the time history accelerations at the fourth floor due to the energy dissipation produced by friction devices. Similar behaviour is displayed for the protected frame subjected to El Centro register with an intensity more than two times higher than the previous cases. The peak acceleration is 1.62 cm/sec^2 and it is still found at the top floor, while at the 4th story the acceleration values are reduced as consequence of the dissipation produced by the friction devices. Compared to the previous case, the increase of the peak accelerations is not proportional to the increase in the intensity of the register. Moreover the peak acceleration is lower than in the case of braced frame subjected to a register more than two times weaker. That means that the friction devices are dissipating a large amount of energy, that is they are working as expected, reducing the response amplification at the upper floors.

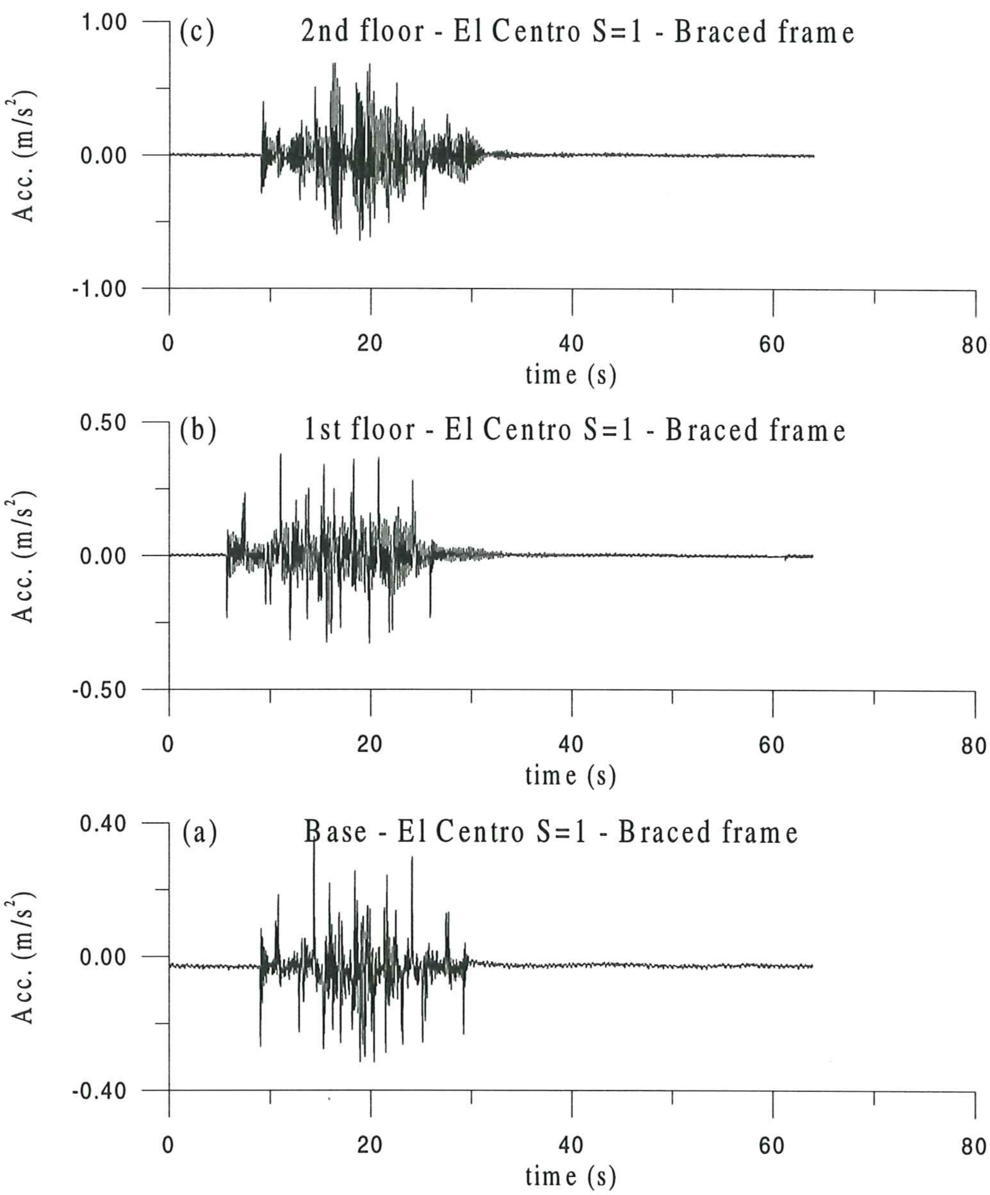


Figure 5.24. Acceleration time histories for the braced frame (a) in the table and (b) at the 1st and (c) 2nd floors.

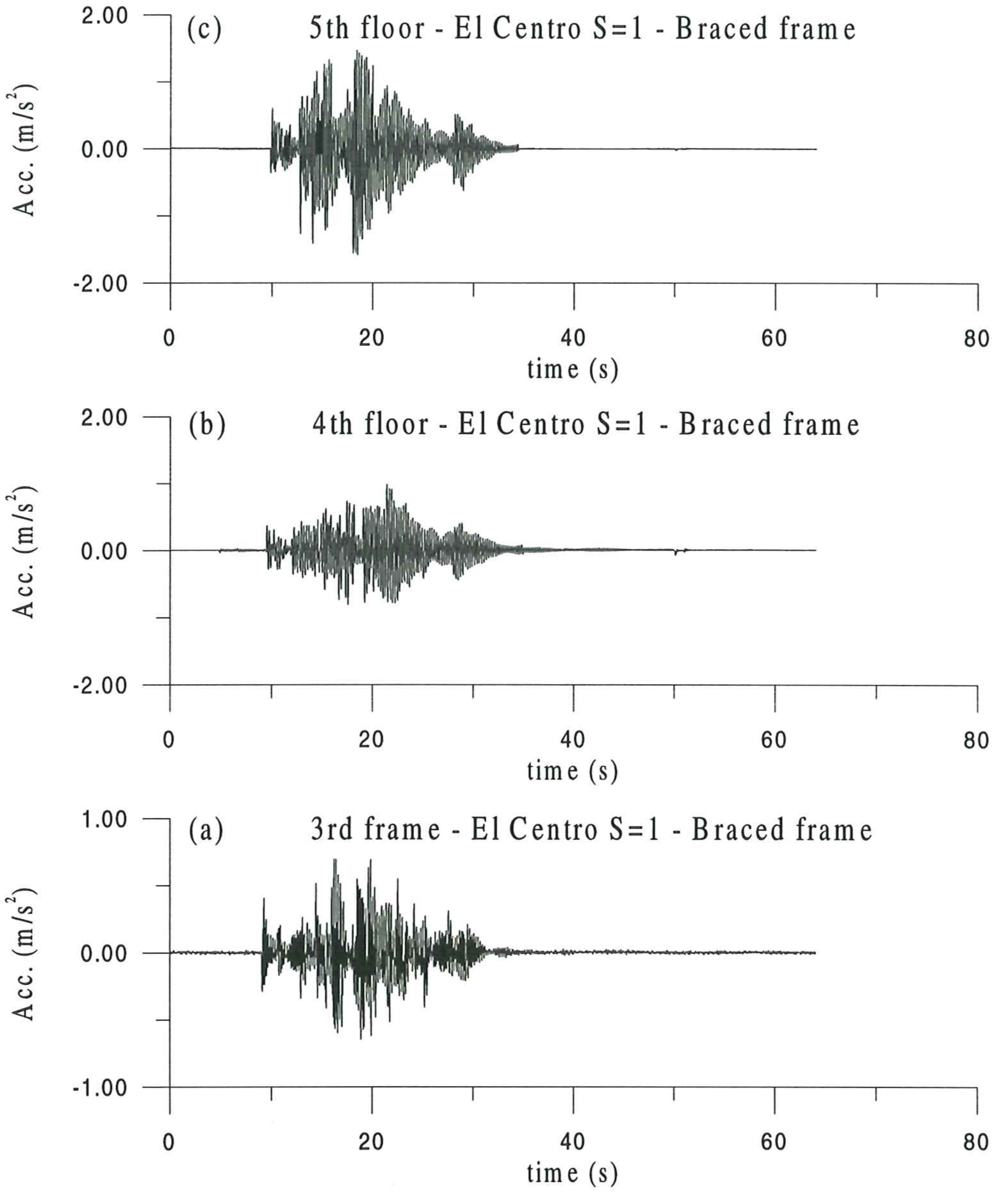


Figure 5.25. Acceleration time histories for the braced frame at the (a) 3rd, (b) 4th and (c) 5th floors.

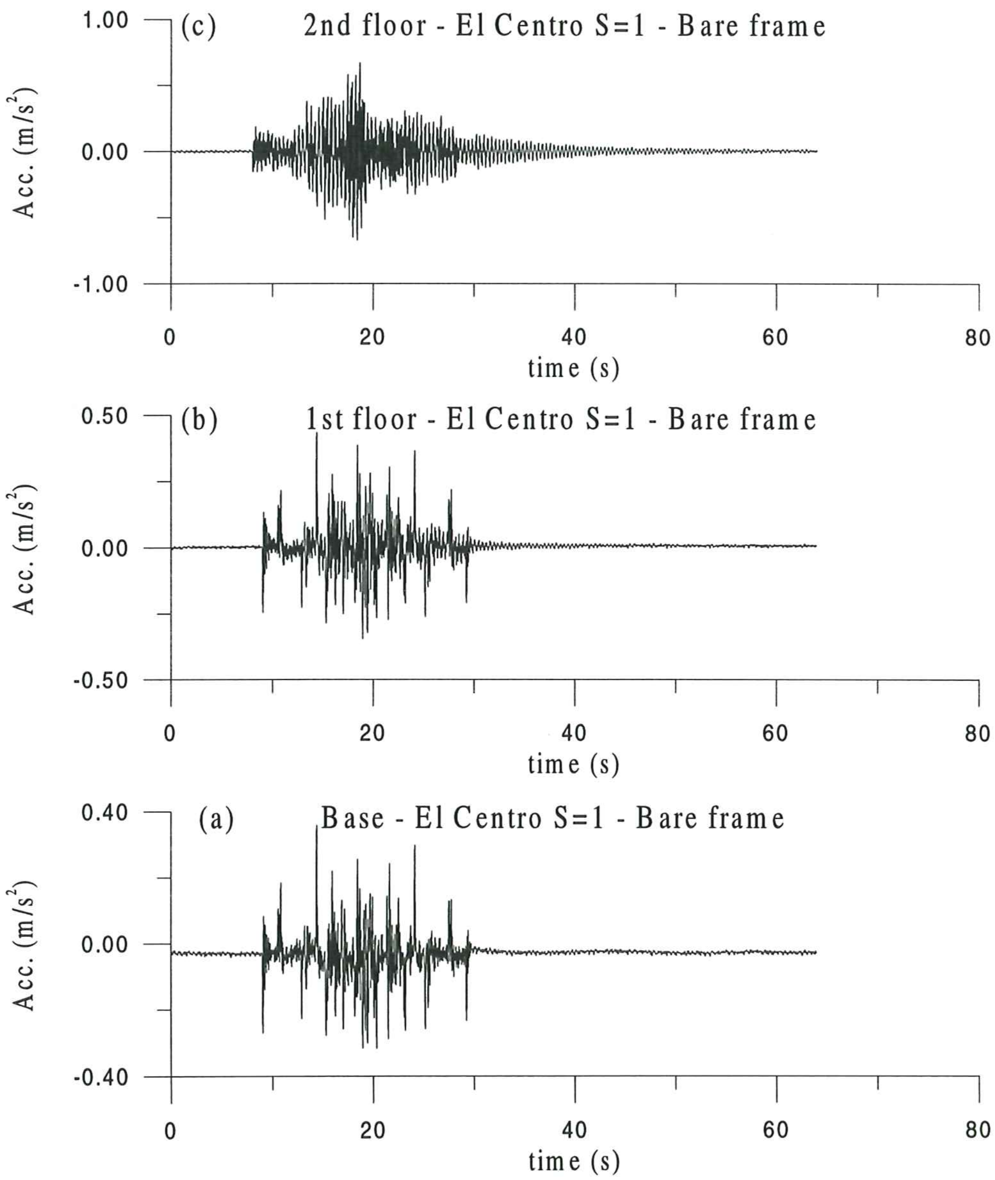


Figure 5.26. Acceleration time histories for the bare frame (a) in the table and (b) at the 1st and (c) 2nd floors.

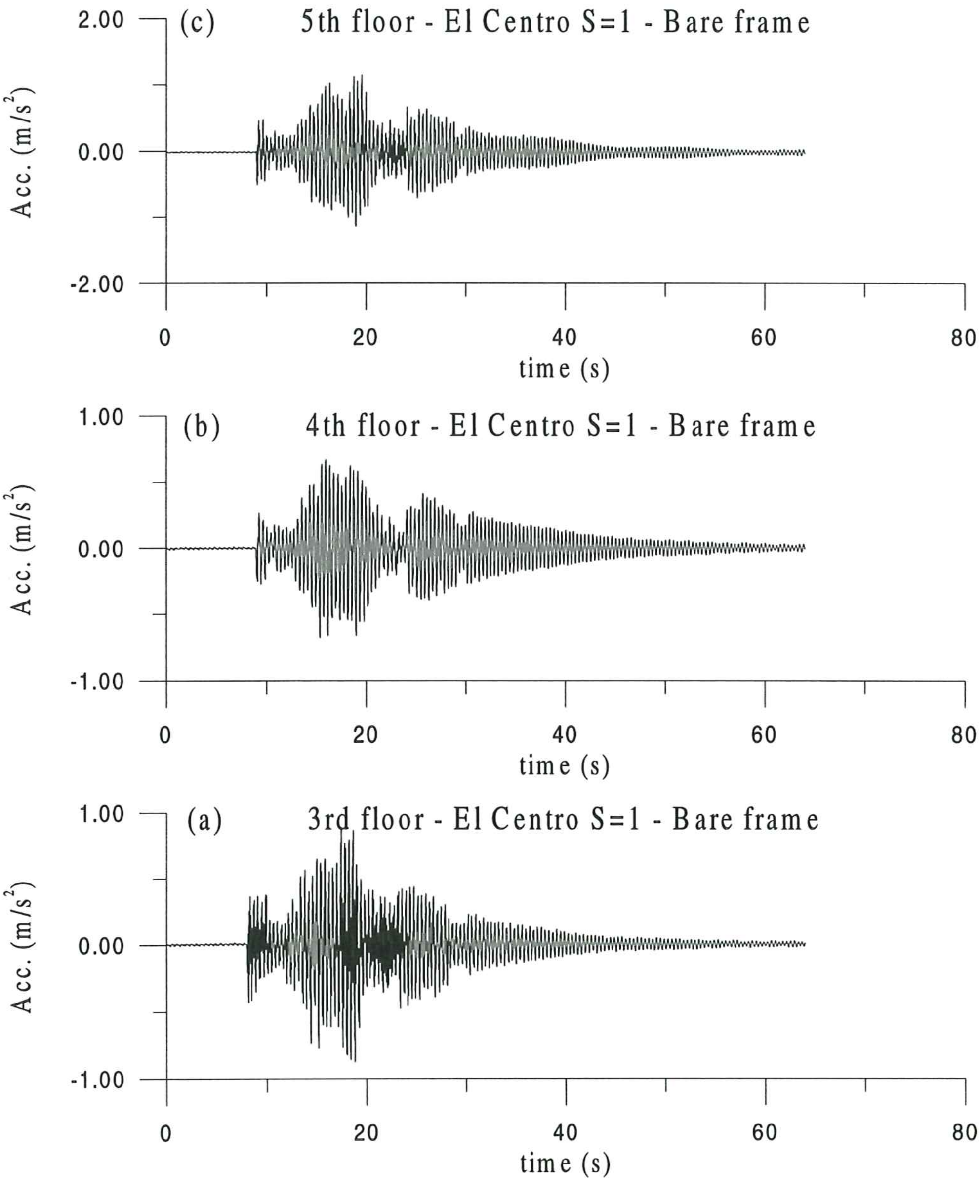


Figure 5.27. Acceleration time histories for the bare frame at the (a) 3rd, (b) 4th and (c) 5th floors.

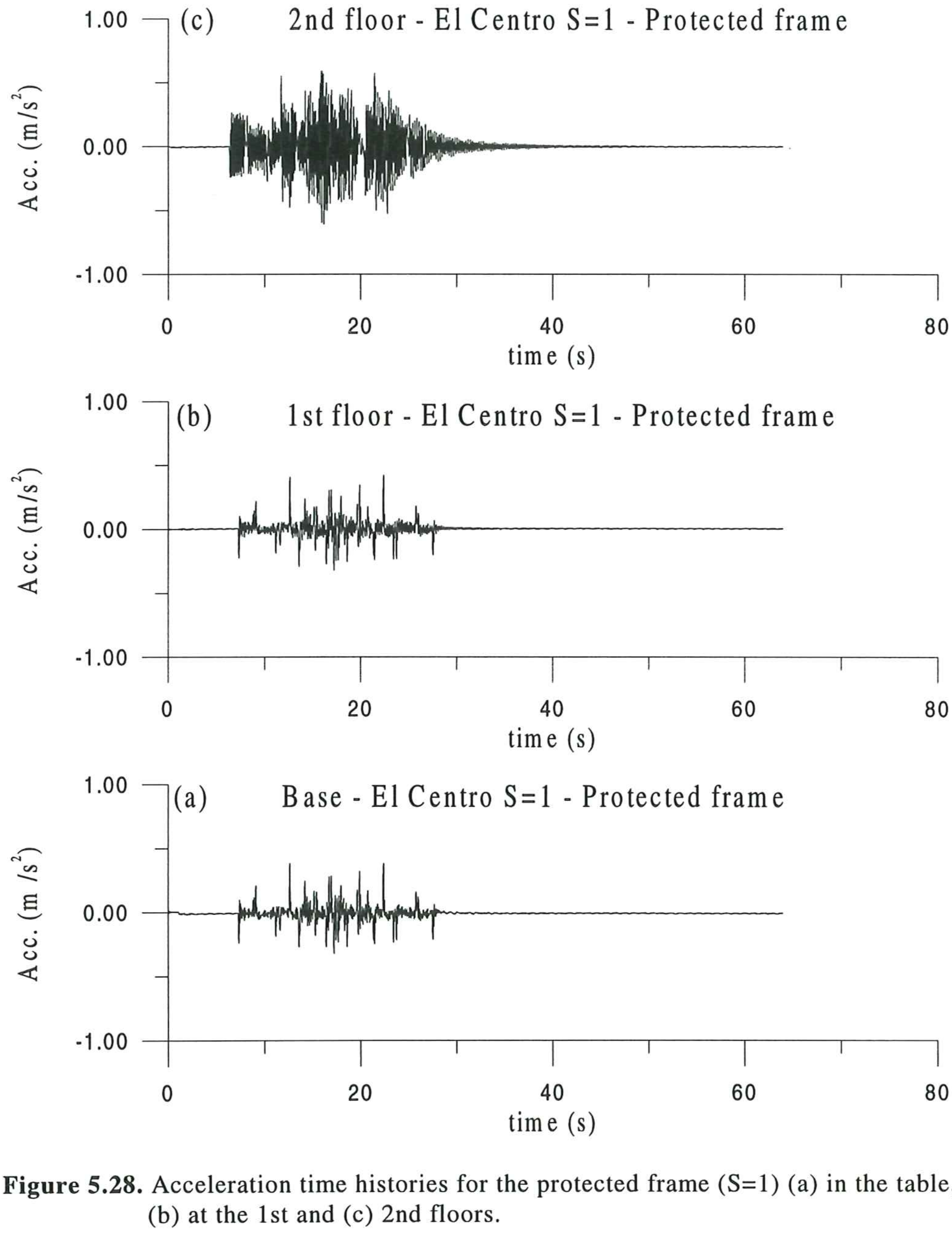


Figure 5.28. Acceleration time histories for the protected frame (S=1) (a) in the table, (b) at the 1st and (c) 2nd floors.

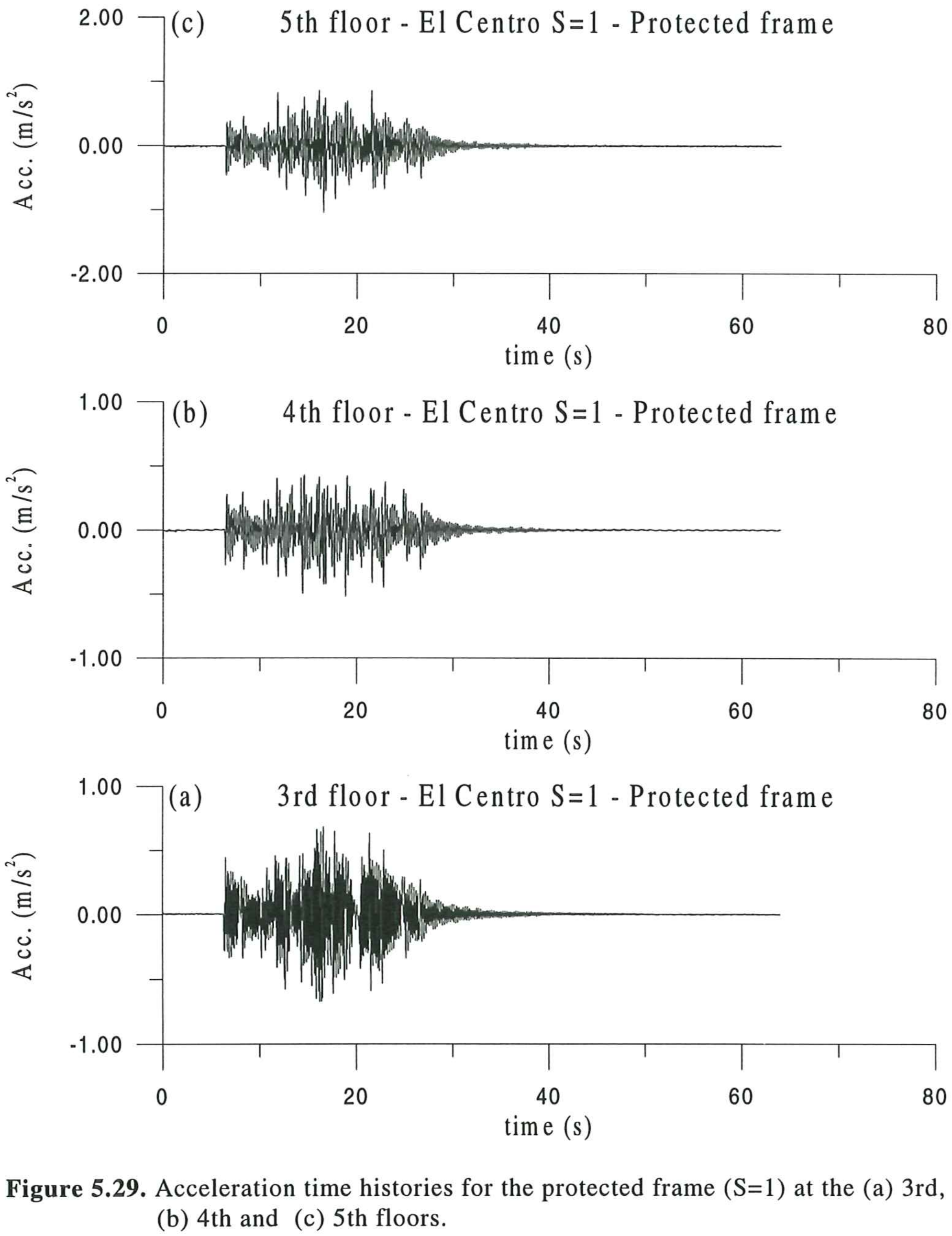


Figure 5.29. Acceleration time histories for the protected frame (S=1) at the (a) 3rd, (b) 4th and (c) 5th floors.

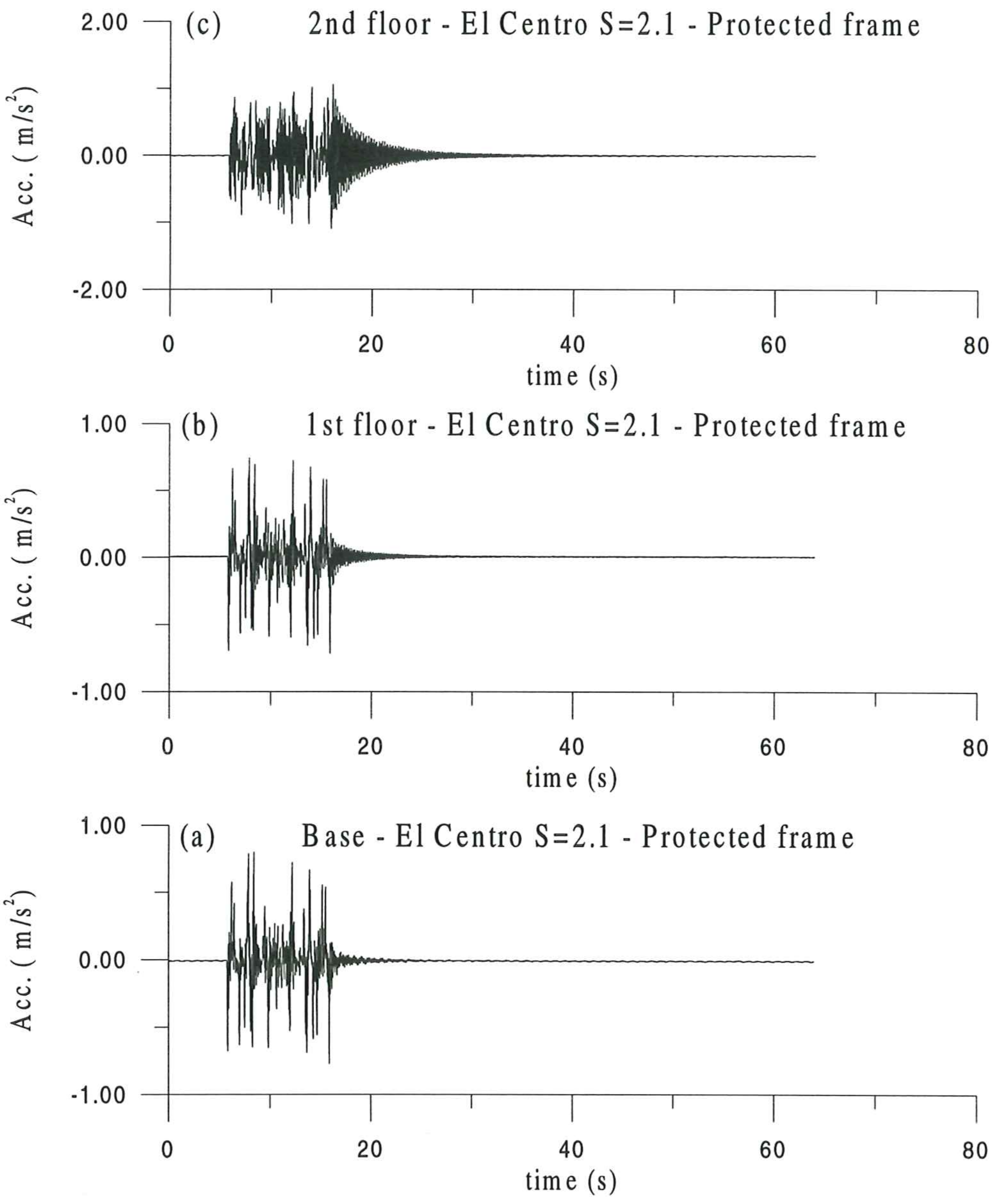


Figure 5.30. Acceleration time histories for the protected frame (S=2.1) (a) in the table, (b) at the 1st and (c) 2nd floors.

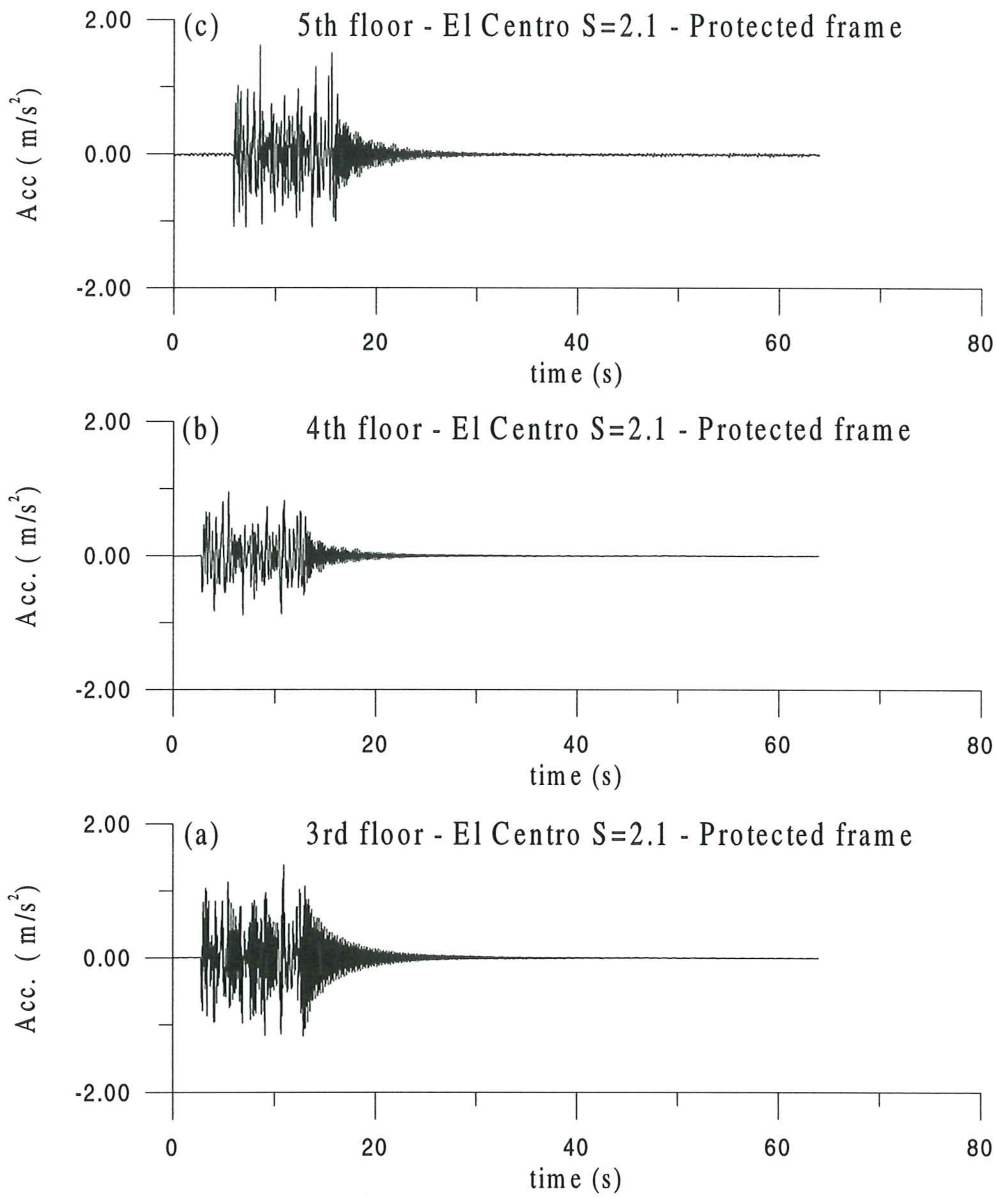


Figure 5.31. Acceleration time histories for the protected frame (S=3.5) at the (a) 3rd, (b) 4th and (c) 5th floors.

Plots 5.32÷5.39 show the Fast Fourier Transform of the accelerations in the table and at each floor for the braced, bare and protected frames subjected to El Centro register.

From Figure 5.6(b) it is possible to determine the frequency content of El Centro earthquake which is between 0 and 10 Hz, that is almost all important frequencies are excited by this signal.

From the FFT diagrams the first two natural frequencies for the braced frame are well established because the peaks are clearly plotted: f_1 = 4.281Hz, f_2 = 9.203Hz. The FFT acceleration value is higher for the first natural frequency in the last two floors.

In case of bare frame, the first natural frequency is shifted to lower values - f_1 = 2.578Hz - while the second one keeps the same value as in case of braced frame: f_2 = 9.156Hz. The acceleration in correspondence of the first natural frequency is maximum in the third, fourth and fifth floors.

For the protected frames, it is possible to determine only a range of frequencies with an amplification of the FFT values of the accelerations. That is especially true for low frequencies around 3Hz. That is due to the non-linear behaviour of the friction dissipators. The second natural frequency of the system is around 9Hz and presents a more evident peak of the FFT acceleration value, especially in the second, third and fifth floors. In case of protected frame with a scale factor of El Centro register S=2.1, the FFT peak values of the acceleration are more evident even for the first natural frequency around 3Hz.

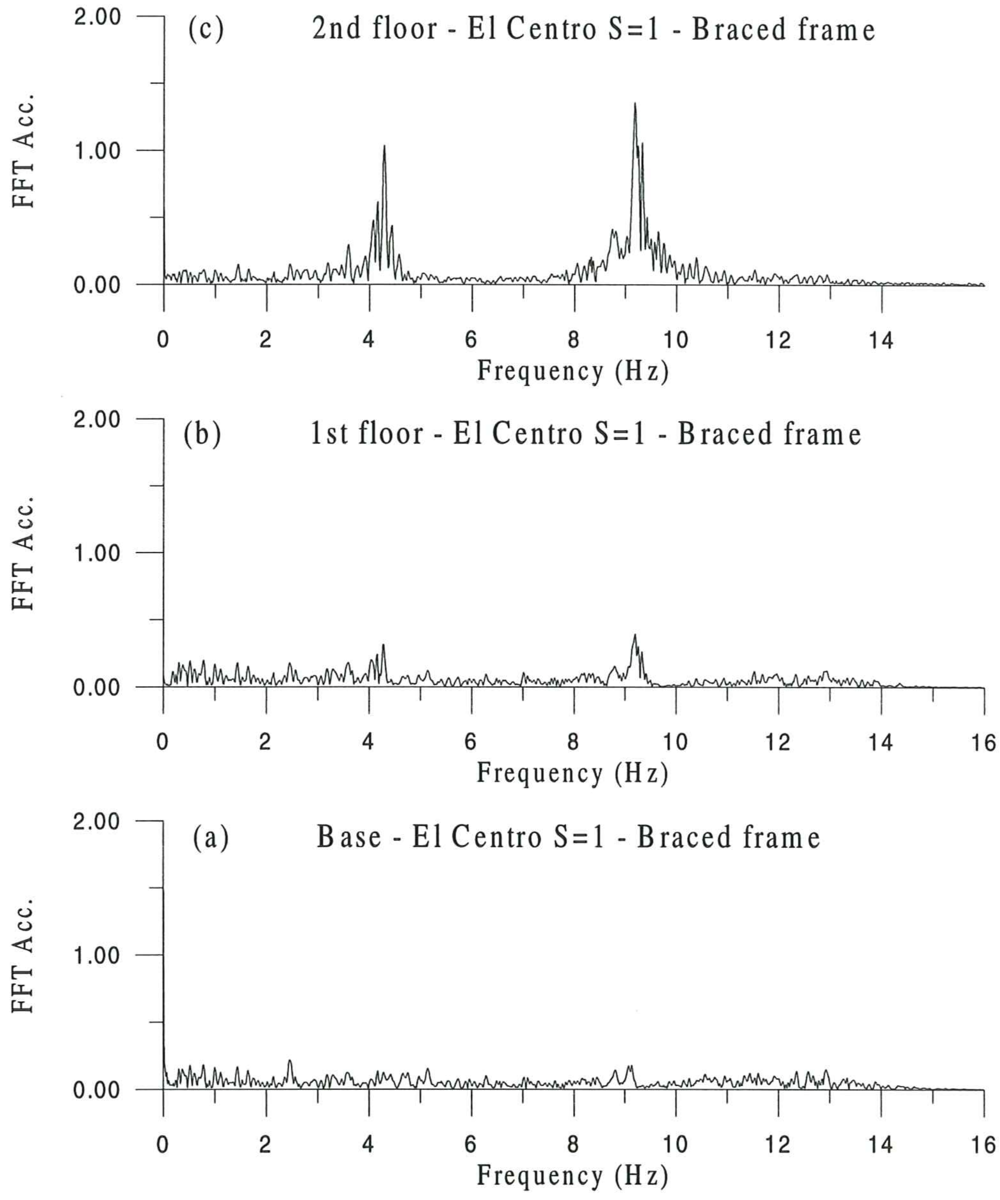


Figure 5.32. Acceleration FFT for the braced frame (a) in the table, (b) at the 1st and (c) 2nd floors.

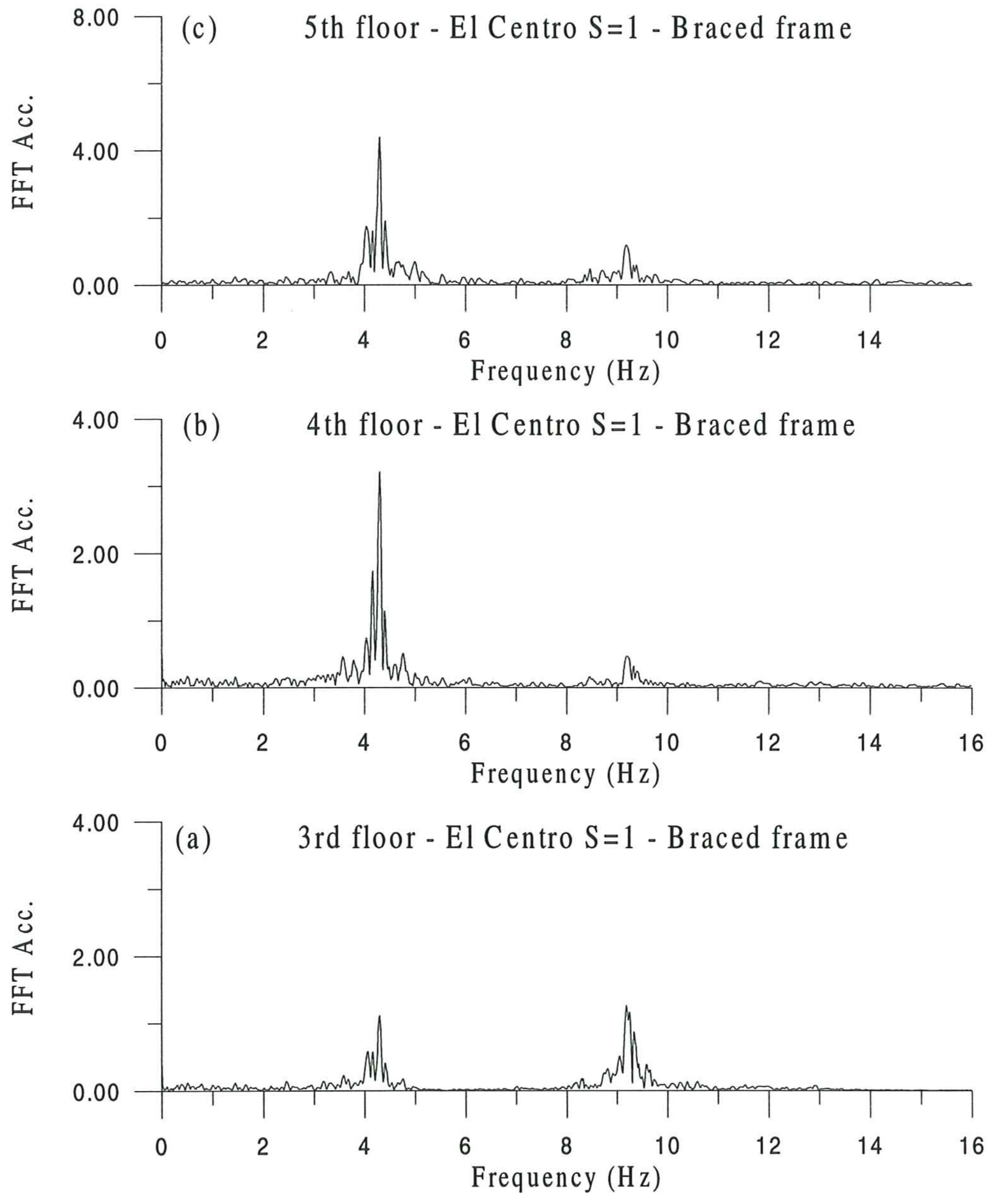


Figure 5.33. Acceleration FFT for the braced frame at the (a) 3rd, (b) 4th and (c) 5th floors.

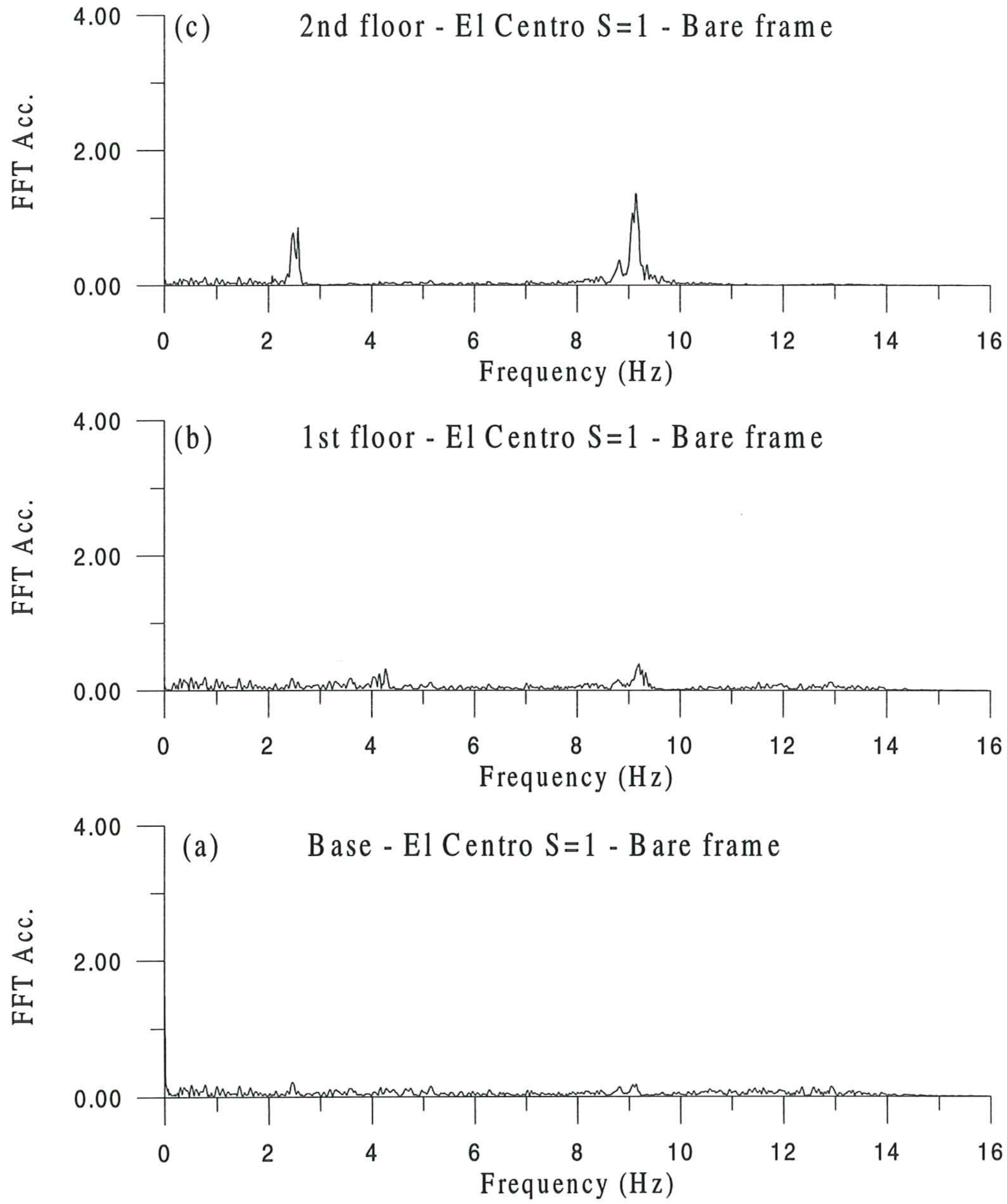


Figure 5.34. Acceleration FFT for the bare frame (a) in the table, (b) at the 1st and (c) 2nd floors.

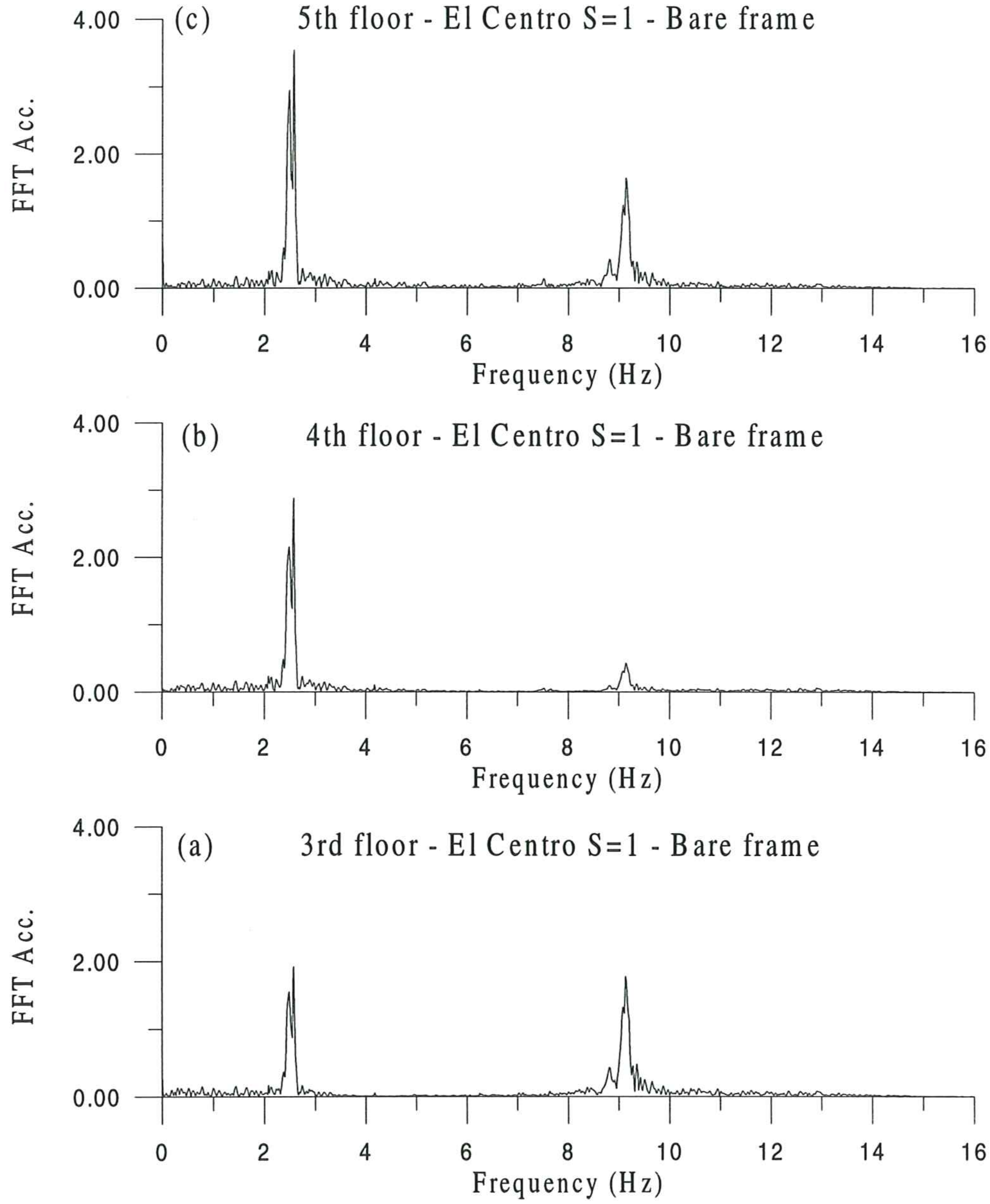


Figure 5.35. Acceleration FFT for the bare frame at the (a) 3rd, (b) 4th and (c) 5th floors.

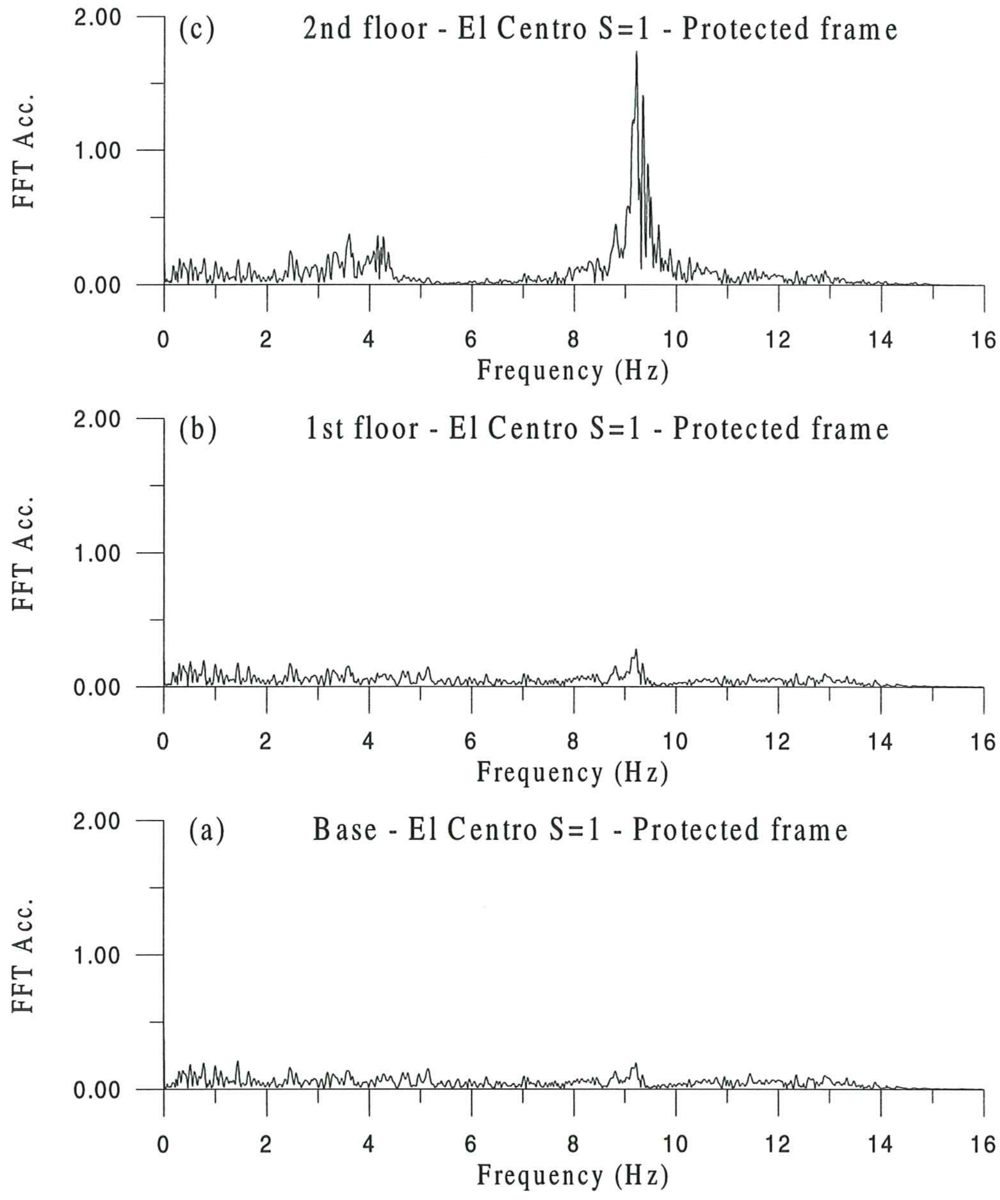


Figure 5.36. Acceleration FFT for the protected frame (S=1) (a) in the table, (b) at the 1st and (c) 2nd floors.

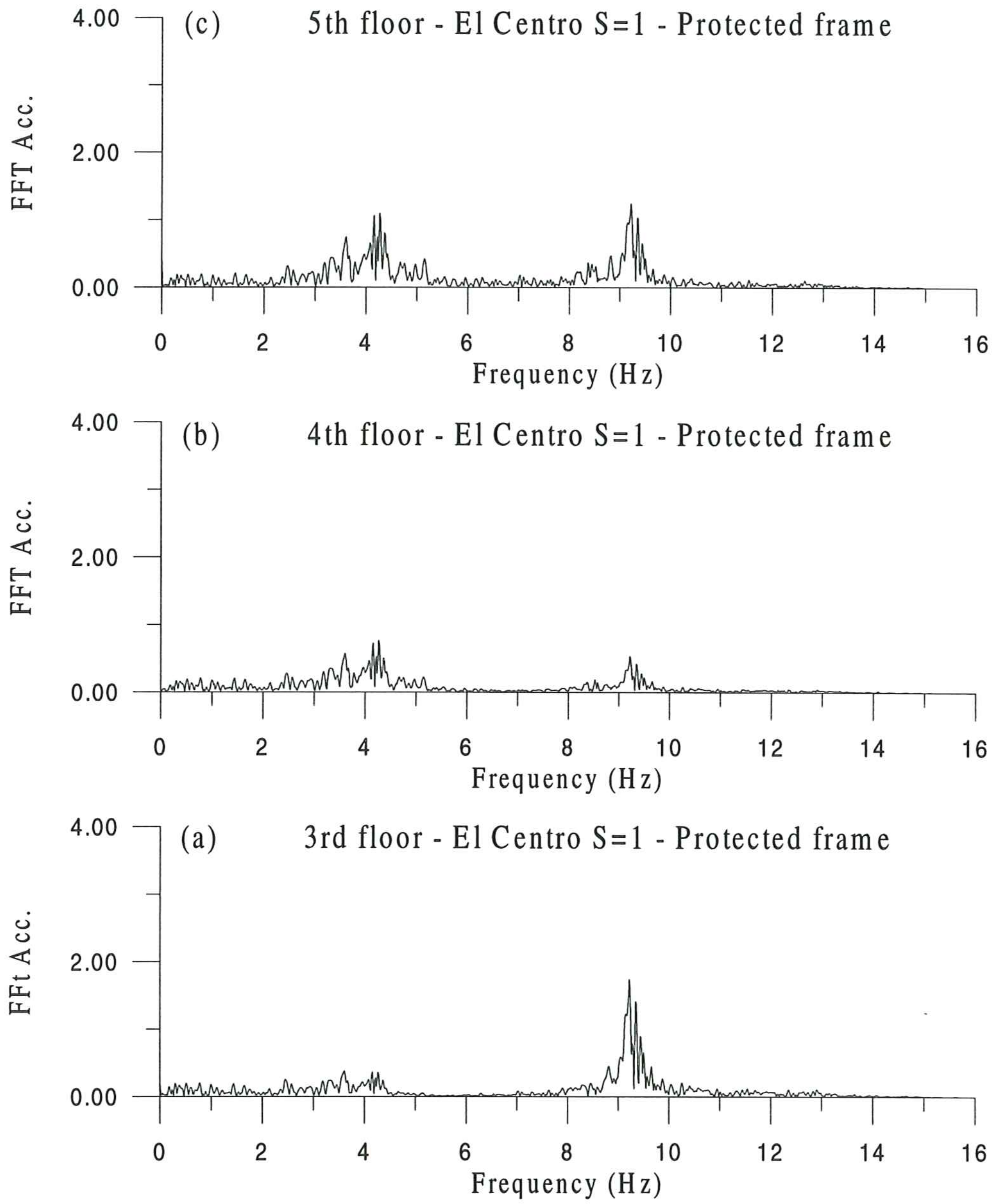


Figure 5.37. Acceleration FFT for the protected frame (S=1) at the (a) 3rd, (b) 4th and (c) 5th floors.

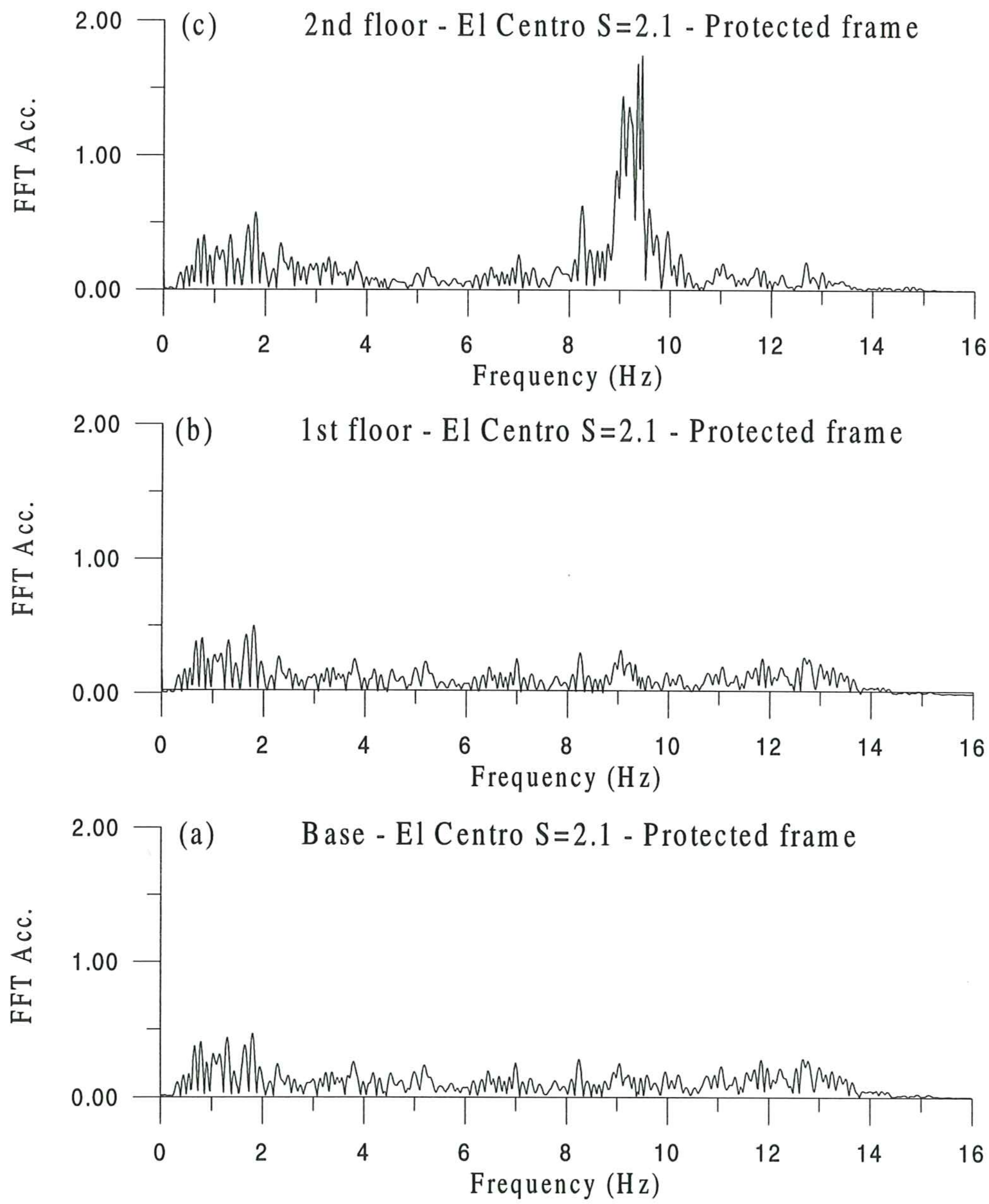


Figure 5.38. Acceleration FFT for the protected frame (S=2.1) (a) in the table, (b) at the 1st and (c) 2nd floors.

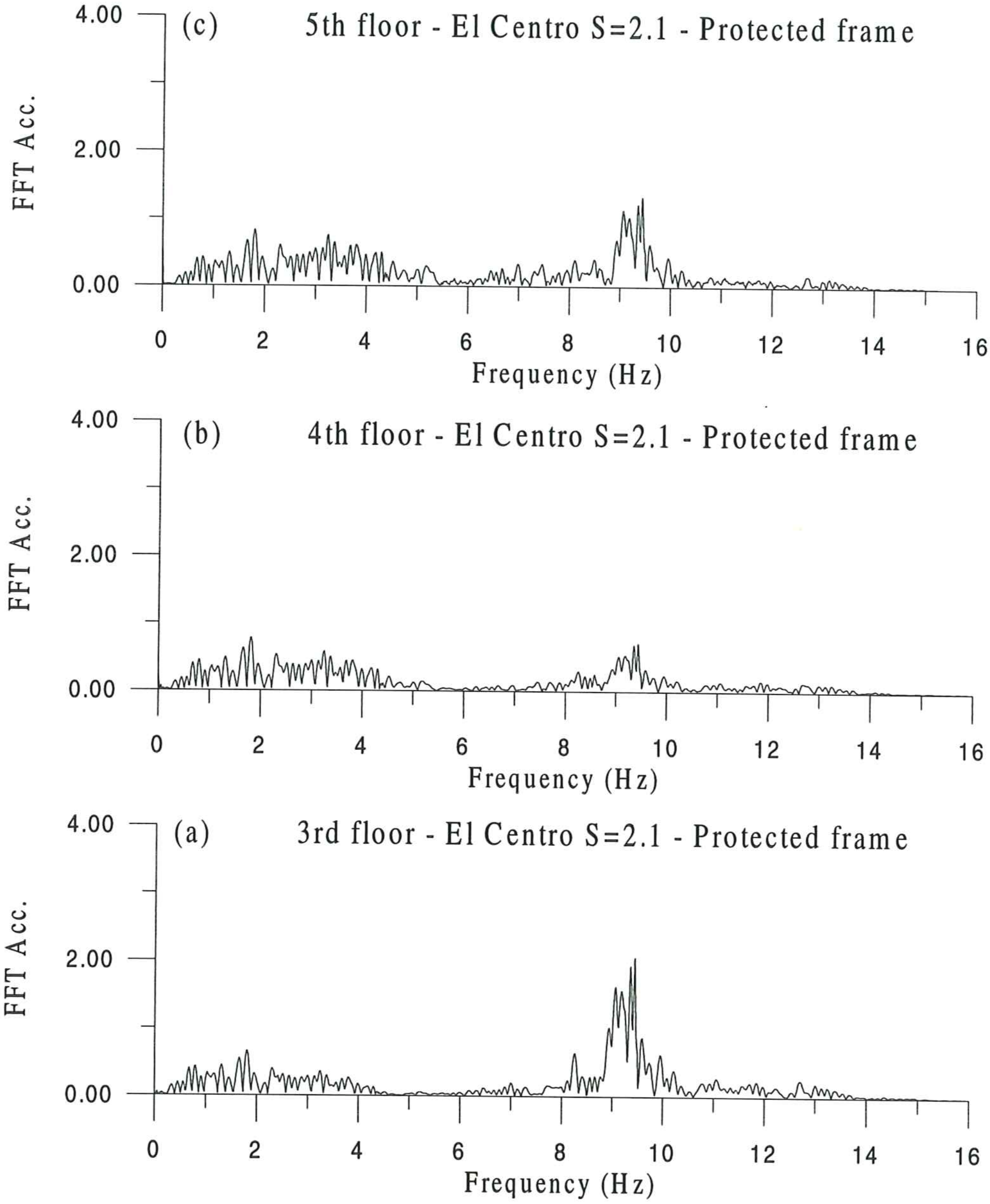


Figure 5.39. Acceleration FFT for the protected frame (S=2.1) at the (a) 3rd, (b) 4th and (c) 5th floors.

APPENDIX

In this Appendix the photos of the model and the friction devices utilised in the experimental study are shown.

Photo 1. The experimental model.

Photo 2. A transversal view of the experimental model.

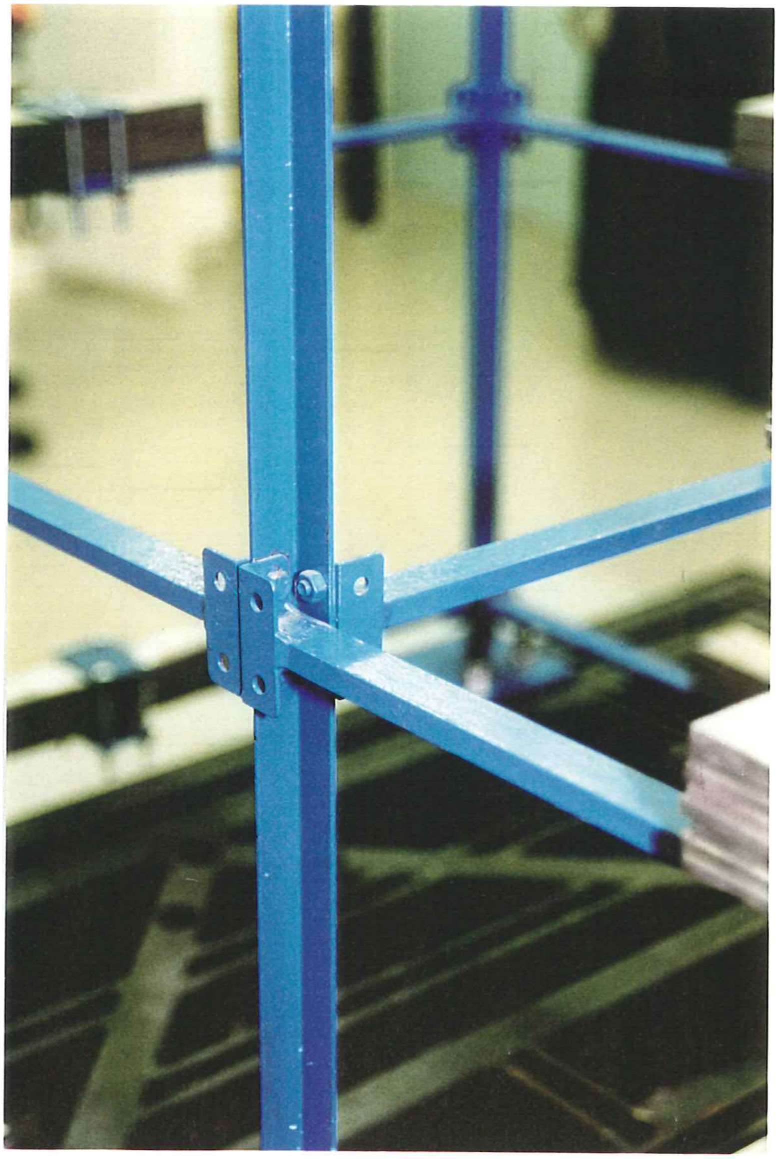


Photo 3. A detail of a triple-node connection.

Photo 4. A detail of the connection of the model to the shaking-table.

Photo 5. The instrumented model.

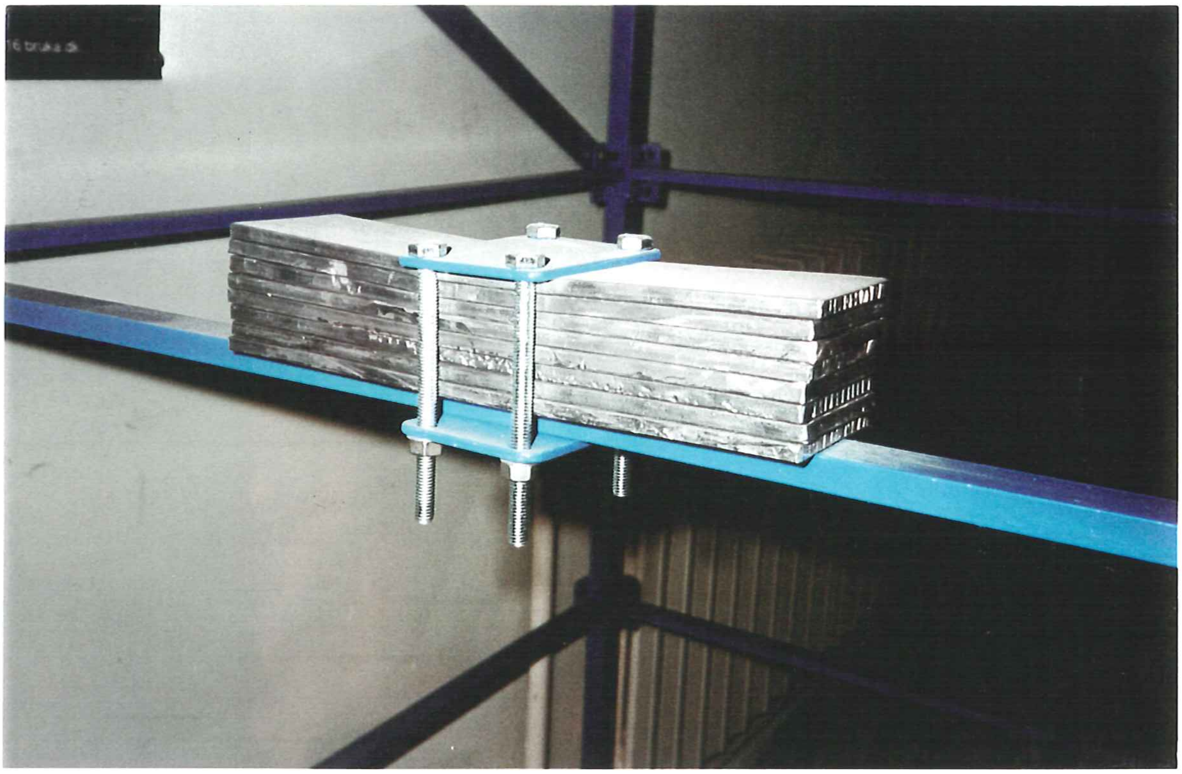


Photo 6. Detail of the added load.

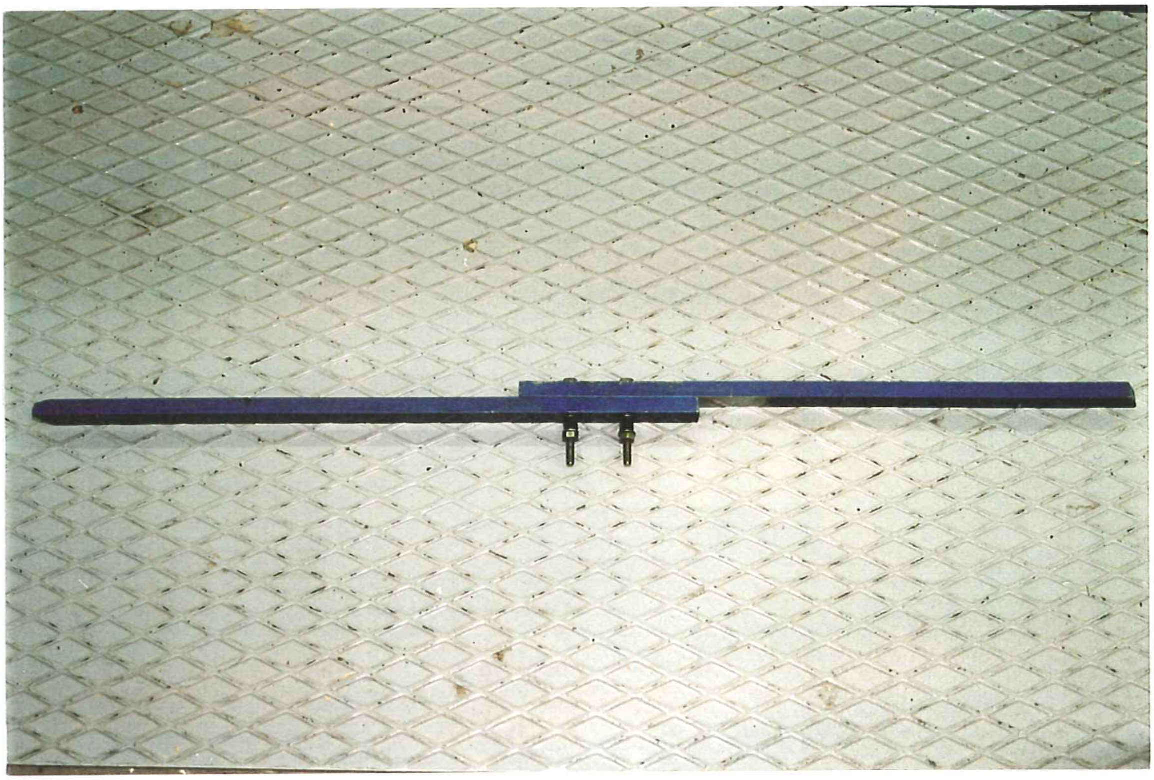
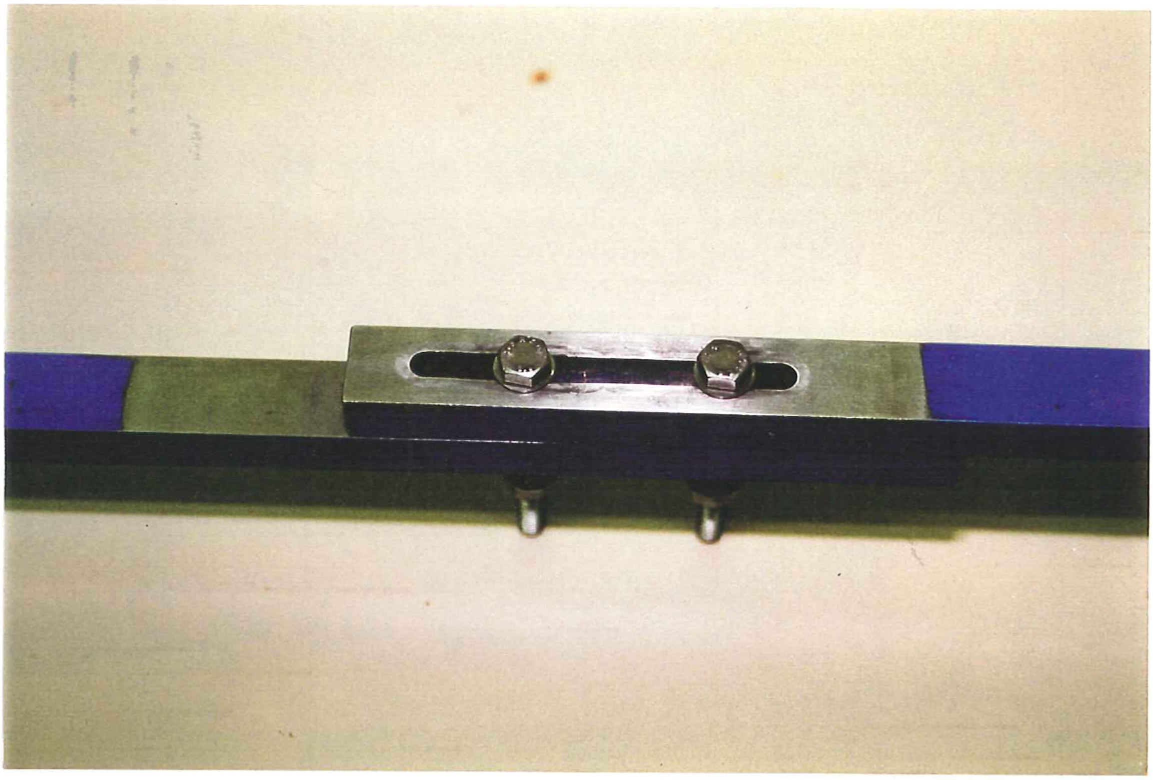


Photo 7. A friction dissipator utilised in the shaking-table tests.

(a)



(b)

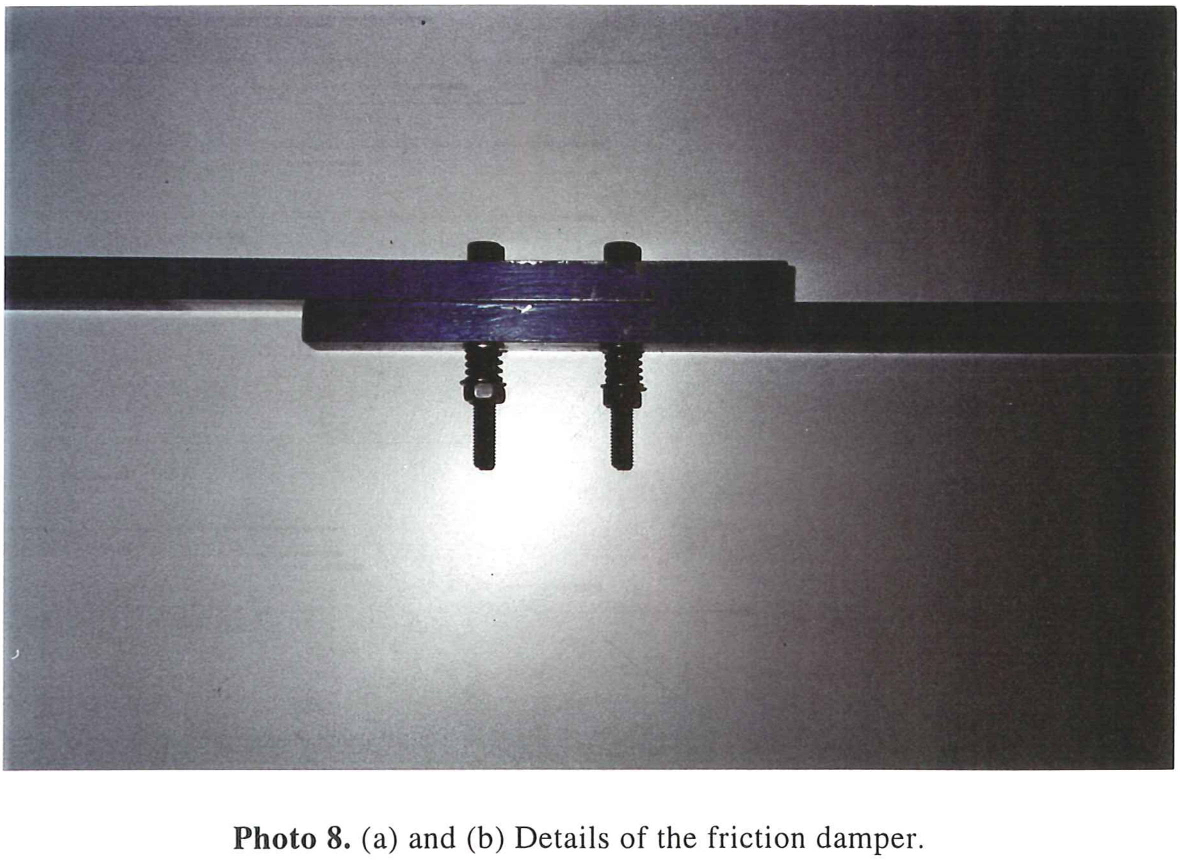


Photo 8. (a) and (b) Details of the friction damper.

Photo 9. The experimental model protected with friction dampers.

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