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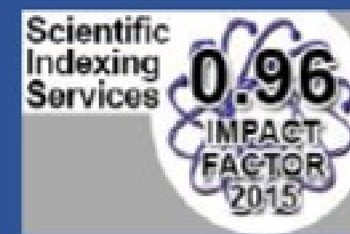


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# ON DYNAMIC CONTROL OF STRUCTURAL VIBRATIONS RESEARCH ACTIVITIES CONDUCTED WITHIN THE COVICOCEPAD PROJECT

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## ABSTRACT

*This paper provides an overview on the latest research and development (R&D) activities carried out under the project entitled **Co**mparison of **Vi**bration **Co**ntrol in **C**ivil **E**ngineering Using **P**assive and **A**ctive **D**ampers (COVICOCEPAD) which was executed within the framework of the Eurocores project, as a part of the sixth European program. The general interest of the paper relies upon the variety of presented highlights relevant to structural control research streams currently under development in a number of European universities, addressing the use of tuned liquid dampers (TLD), base isolation devices, magneto-rheological (MR) dampers and a hybrid technique using both devices together. The paper also provides details of a few new testing equipments which are in use of the relevant laboratories. Finally, research projects in the field of structural control at the involved research institutes are reviewed.*

**Keywords:** structural vibrations; smart structures; passive; hybrid control; control strategies; experimental tests.

## 1. INTRODUCTION

In the last two decades research and development (R&D) activities related to structural vibration control devices for buildings and bridges have been intensified to meet the construction market demand of effective systems in order to control damage caused by seismic and wind loading. This orientation is the result of a public necessity to guarantee the serviceability of construction lifelines during and after the occurrence of a moderate or severe seismic event [1, 2].

The objectives, advancements, activities and results of the project entitled **Co**mparison of **Vi**bration **Co**ntrol in **C**ivil **E**ngineering Using **P**assive and **A**ctive **D**ampers (COVICOCEPAD) are synthesized in the present paper. The theoretical problems, concerning the design of innovative control strategies and algorithms for the control of civil structures, and the experimental activities already developed or in course of development at the laboratories involved in the project are described.

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Manuscript received on 31th January 2012, reviewed and accepted on 24th July 2012 as per publication policies of *NED University Journal of Research*.

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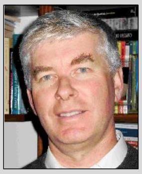

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Within this framework, the R&D activities and their results, and related scientific contributions of the project in terms of a number of features, such as passive, semi-active and hybrid control strategies (including tuned liquid dampers (TLD), base isolation systems, magneto-rheological dampers (MD), and their coupling), are reported in the paper [3-15].

Besides the experimental tests designed for the project at the University of Naples (by means of a previously acquired shake table) other research activities were carried out and experimental infrastructure was developed. This includes setting up a Quanser shake table II at FEUP (Porto), which was calibrated with the use of the supplied two degree-of-freedom (DOF) frame equipped with two passive/active TMD devices. Future collaborative work is planned, such as the analysis of obtained experimental results at FEUP and their analytical modelling, theoretical developments at the University of Naples (Federico II) and IST (Lisbon), and larger scale tests using vibration damper devices on a steel frame at LNEC (Lisbon).

This paper presents a synthetic collection of results from theoretical, numerical and experimental activities carried out within the project related to the dynamic control of structural vibrations. These range from testing on classical base isolators to their novel design conceived (at the University of Naples) in such a way so as to take into account the mechanical parameters of the subsoil at the site in order to get a higher performance. Results are reported in regard to the modelling and static/dynamic testing of semi-active control devices exploiting the special characters of magneto-rheological fluids, executed in Porto.

The problem of controlling the rigid structures behaving according to a unilateral scheme due to simple pure rocking under dynamic solicitations [13-15] is addressed (activities developed at the University of Naples). The experiments executed on the shake table on rigid structures prototypes equipped with TLD confirm the potentials. The existing research on this type of control device in regard to the reduction of the dynamic effects on the structural oscillations is limited.

Results relevant to the development of new control algorithms of linear-derivative type are reported. This work was carried out at the University of Porto and aims at controlling bridges decks. The method adopts control criteria which are able to cause a considerable improvement of the system response.

Finally, planning of tests to be executed and/or in course of development in Lisbon on a large shake table facility are described along with the description of facility which provides the details in advance of the future research directions.

## 2. DEVELOPMENTS OF PASSIVE CONTROL DEVICES USING BASE ISOLATION (BI)

### 2.1 Design and Numerical Testing of a BI Device

In the last two decades, R&D of structural vibration control devices for buildings and bridges has been intensified to meet to the construction market needs that demand more effective systems to decrease the damage caused by seismic and wind loading. This orientation is the result of a public necessity to guarantee the serviceability of construction lifelines during and after the occurrence of a moderate/severe seismic event.

In this context, the strategies based on the passive control, namely the base isolation (BI) systems, shock absorbers (SA) and tuned mass dampers (TMD) are well-known and accepted methodologies due to their effectiveness in mitigating the effects of dynamic loading. However, the limitations that these devices/methodologies have to allow variations of the dynamic loading or structural parameters encouraged the study and development of more advanced control systems based on active, semi-active or hybrid control devices (**Figure 1**).

The numerical models used to simulate this new type of control in civil engineering structures allows to conclude that is possible to apply them successfully provided experimental research is conducted in order to validate these models so that they are accepted as a possible structural vibration control solution. Since active devices are not a practical and reliable option in the near future, as a structural building or bridge control systems, due to energy demand and possible failure in case of power loss, semi-active control devices gain significant attention by the civil engineering community in the last few years. They have the benefits of acting both as passive and active devices without requiring large amount of energy.

The easiest and cheapest way to protect a structure from undesired vibration is to add a passive isolation system to reduce the response in some sensitive region. **Figure 2** shows a single DOF (SDOF) isolator, modelled by the transmissibility transfer function

$$T = \frac{x_p}{x_b} = \frac{\dot{x}_p}{\dot{x}_b} = \frac{\ddot{x}_p}{\ddot{x}_b} = \frac{1 + 2\xi_p \dfrac{s}{\omega_n}}{1 + 2\xi_p \dfrac{s}{\omega_n} + \dfrac{s^2}{\omega_n^2}} \tag{1}$$

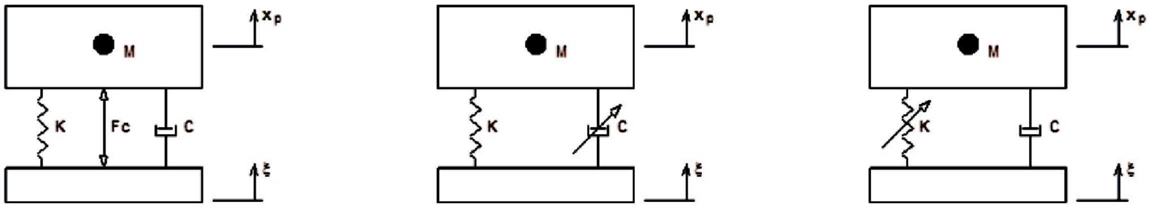



**Figure 1. Active and semi-active vibration control strategies.**

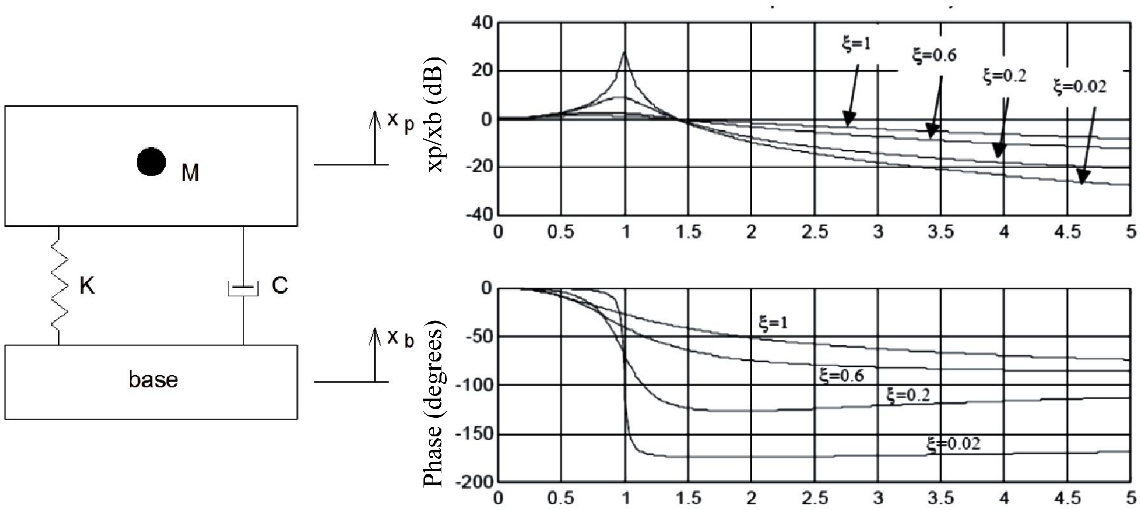



**Figure 2. Passive 1-DOF isolator and transmissibility transfer function.**

where $S$ is the Laplace variable, $\omega_n$ is the natural frequency, $\xi$ is the passive damping ratio, $x_p$ and $x_b$ are the payload and base displacements. The transmissibility modulus and phase for several damping ratios are also shown in **Figure 2**. In **Figure 2**, it is visible that at low passive damping ratios, the resonant transmissibility is relatively large while at frequencies above resonant peak is quite low and the reverse is true for relatively high damping ratios.

The base isolation system is the first approach in the analysis of structures with passive energy dissipation devices and is based on the use of a model with two DOF (translational) represented in **Figure 3**. The background to this work was based on Naeim and Kelly [16] and Soong and Dargush [17] that resumes the theory of seismic isolation and the design of seismic isolated structures, within the framework of existing codes. In this context such model was used earlier [18, 19]. Also Figueiredo and Barros [20] have addressed the importance of the influence of increasing damping in seismic isolation.

The classical design approach for the base isolation devices accounting horizontal seismic component is also shown in **Figure 3**. In this case, the equation of motion (Eq. (2)) has the matrix identifications (mass, damping and stiffness matrices) as given in Eq. (3).

$$M\ddot{X}(t) + C\dot{X}(t) + KX(t) = - M\,\{I\}a(t) \tag{2}$$

$$M = \begin{bmatrix} m_b & 0 \\ 0 & m_1 \end{bmatrix},\; C = \begin{bmatrix} c_b + c_s & -c_s \\ -c_s & c_s \end{bmatrix},\; K = \begin{bmatrix} k_b + k_s & -k_s \\ -k_s & k_s \end{bmatrix} \tag{3}$$

where $c_b$ is the damping coefficient of the base isolation system; $c_s$ is the damping coefficient of the structure; and $k_s$ is the stiffness of the structure.

Base isolation devices are made of natural rubber that guarantees damping ratios in the order of 10-20% of critical damping, considerably larger than structural damping ratios for steel frames (in the order of 2%). In the study, a vibration absorber mounting, similar to those used for machine isolation, is used to simulate the base isolation system. As this simplified passive system has an insignificant energy dissipation (an important characteristic for real building applications) the expected structural

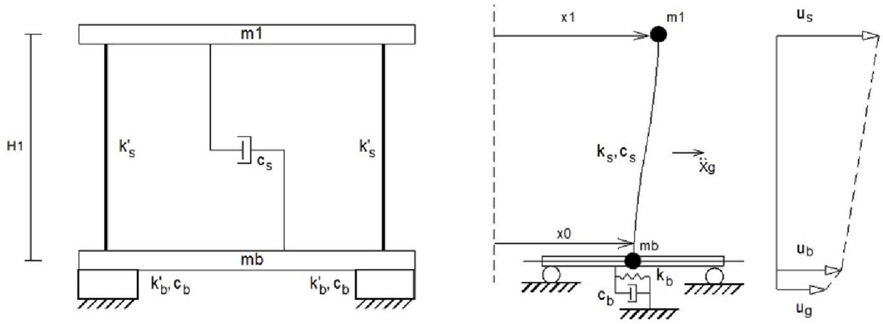



**Figure 3. Passive energy dissipation at the base of the structure (base isolation).**

behaviour remains the same as for a real base system. Note that the main purpose of a base isolation system is to increase the structural flexibility, and consequently the fundamental period to a more secure range so that the resonance effects are significantly lowered (and often irrelevant) making it a type of pseudo-filter.

## 2.2 Improved BI Devices Accounting for Soil Properties

The capacity of base-isolation devices for structural applications in mitigating inertia forces, due to intense earthquakes, strongly depends on the proper calibration of the isolator own frequency. This should be carefully dimensioned taking into account both the dynamical characteristics of the superstructure and the frequency content of the expected disturbance [21, 22]. Surface layers of the ground on which the building is founded have the capacity of filtering the incoming dynamic excitation making its signal dependent on the characteristics of the soil. It is usually very important to make some possible forecast about the distribution on the territory of the seismic wave [23-25] as well as to develop models for studying the effects of the soil on the structure and of the structure on the soil [26, 27].

Therefore, in designing a base isolator system, one should not neglect the interaction effects between the structure and the soil at the site. The soil behaves like a filter for the incoming seismic excitation, mainly affecting its frequency composition and its overall dynamic character.

In the Department of Structural Engineering at the University of Naples Federico II, a strategy for designing an effective isolation device on the basis of the knowledge of the structure mechanical characteristics and of the soil properties has been developed. This is based on the idea that the isolator behaviour should be such as to minimize the energy introduced in the structure by the dynamic excitation, while allowing a bounded energetic absorption in the isolator itself, lower than a prefixed threshold.

The adoption of this approach requires the evaluation of the energies characterizing the structure $E_{str}(\omega \mid \eta, \xi)$ and the base isolation device $E_{is}(\omega \mid \eta, \xi)$ during the seismic motion, as functions of the varying isolator characteristics, such as $m_{is}$, $c_{is}$, $k_{is}$, and of the soil properties $\eta$ and $\xi$. These energies can be obtained by referring to the integrals of the relevant auto-spectral density terms.

The search of the isolator parameters can be obtained by considering that the energy absorbed by the structure is minimum while the energy introduced in the isolator is kept bounded, by setting up the problem

$$\text{Find} \quad \min_{m_{is}, c_{is}, k_{is}} \mathcal{E}_{str}(\omega \mid \eta, \xi)$$
$$\text{Sub} \quad \begin{cases} \mathcal{E}_{is}(\omega \mid \eta, \xi) \leq \overline{\mathcal{E}}_{is} \\ m_{is} \geq \overline{m}_{is} \end{cases} \tag{4}$$

In Eq. (4), one can observe the presence of two constraints, the aforementioned bound on the isolator energy absorption $\overline{E}_{is}$ and a practical bound on the minimum value of the isolator mass $\overline{m}_{is}$ . Numerical investigation developed at the University of Naples on base isolated shear-type structures, subject to accelerations compatible with the Kanai-Tajimi spectra with assigned parameters $\eta$ and $\xi$, allow to observe that in the cases when the soil is very soft (mean square ratio higher than one) or poorly stiff (mean square ratio approximately one) with comparison to the structure, the isolator is not useful.

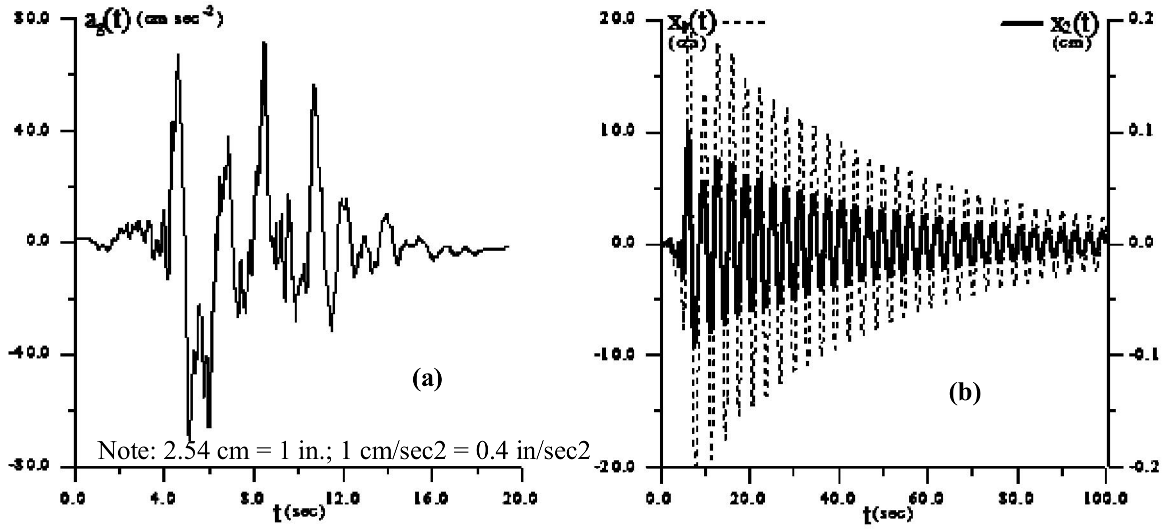



**Figure 4. (a) Base acceleration compatible with the Kanai-Tajimi spectrum with $\eta$ = 5 sec$^{-1}$ and $\xi$ = 3; (b) time response of the isolated structural system.**

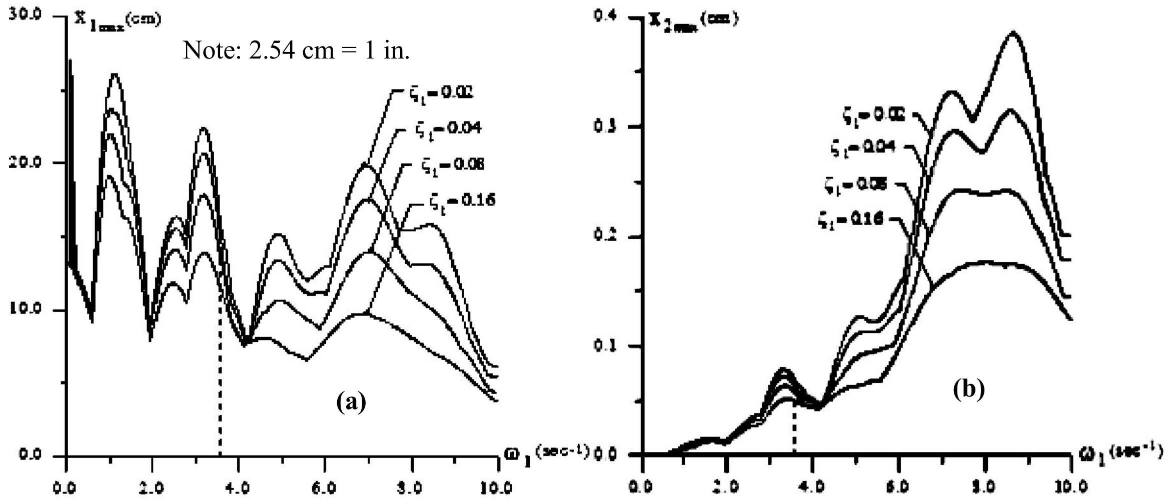



**Figure 5. Curves of the maximum drifts attained during the motion by varying isolator parameters $\omega_1$ and $\xi_1$: (a) isolator; (b) superstructure.**

In **Figure 4**, one reports the time response of the optimal isolator $x_1(t)$ and of the optimally isolated superstructure $x_2(t)$ are reported, for a base isolated SDOF shear frame subject to a base acceleration compatible with a Kanai-Tajimi spectrum with parameters $\eta$ = 5 sec$^{-1}$ and $\xi$=3.

The results on the same structure while varying the isolator mechanical characteristics in terms of $\omega_1$ and $\xi_1$ are plotted in **Figure 5**, where the maximum response values attained by the isolation device and by the superstructure are reported.

## 3. DEVELOPMENTS ON SEMI-ACTIVE CONTROL DEVICES USING MAGNETORHEOLOGICAL FLUIDS

### 3.1 Set up of the Model

The adoption of semi-active devices to control the vibration of a base excited structure has shown some interesting potentials. Among the possible semi-active technologies, the magneto-rheological fluid based devices are seen as a promising solution for structural control.

Two types of rheological fluids can be used to create a structural control system: Magneto-rheological (MR) and Electro-rheological (ER) fluids. MR fluids are materials that exhibit a change in rheological

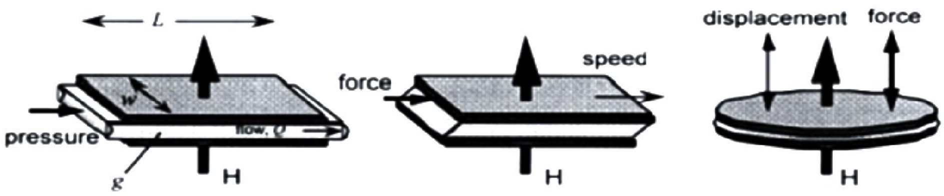



**Figure 6. Basic operation modes of MR fluid.**

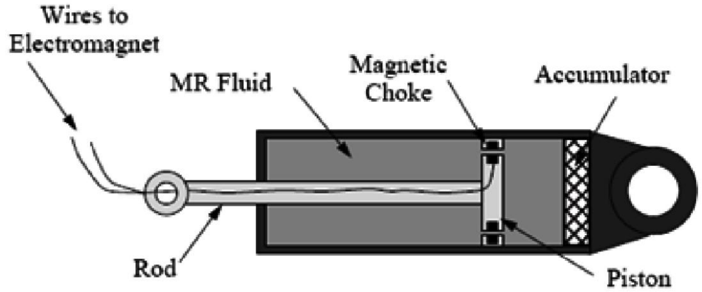



**Figure 7. Schematic representation of a magneto-rheological damper.**

properties with the application of a magnetic field while ER fluids exhibit rheological changes when an electric field is applied to the fluid.

Although power requirements are approximately the same MR fluids only require small voltages and currents while ER fluids require very large voltages and very small currents. However, ER fluids have many disadvantages including relatively small rheological changes and significant property changes with temperature. Therefore, MR fluids have become an extensively studied as 'smart' fluids and some experimental research has been conducted in previously to produce a 'smart' control device with this fluid.

Usually, the MR fluid devices are built to operate in the following modes (**Figure 6**): valve mode or flow mode; direct shear mode or clutch mode; squeeze film compression mode and the combination of some of the previous modes.

For vibration control purposes the 'smart' MR fluid effect is interesting since it is possible to apply this phenomenon to create a variable damping device or a 'smart' hydraulic damper. The current applied to a MR fluid essentially allows controlling the damping force without the need of mechanical valves that are commonly used in adjustable dampers. This offers the possibility to create a reliable damper since a failure in the control system reverts the MR damper to a passive damper (**Figure 7**).

The MR damper performance is often characterized by using the force versus velocity relationship. Regular viscous damper has an ideal linear constitutive behaviour and the slope of the line is known as the damper coefficient. In the case of MR dampers the possibility to change the damping characteristics leads to a force versus velocity envelope that can be described as an area rather than a line in the force-velocity plane. This behaviour is the fundamental condition to build a 'smart' damper since it is possible to design a controller to follow any force-velocity relationship within the envelope to create a control strategy.

According to the available literature, two common methods can be used to model MR devices as MR dampers: the parametric modelling technique that characterizes the device as a collection of springs, dampers, and other physical elements and the non-parametric modelling that employ analytical expressions to describe the characteristics of the modelled devices. Many authors have developed modelling techniques for the MR dampers based on both these methods. In recent studies, Dyke et al [28] presented a simple parametric model based on the extension of the Bouc-Wen model [29] that allows a good approximation to the real MR damper behaviour as shown in **Figure 8**.

The Bouc-Wen model [29] is based on the Markov-vector formulation to model nonlinear hysteretic systems using the modified model shown in **Figure 8**, the MR force can be computed by

$$F_{MR} = c_1 \dot{y} + k_1 \left( x - x_0 \right) \tag{5}$$

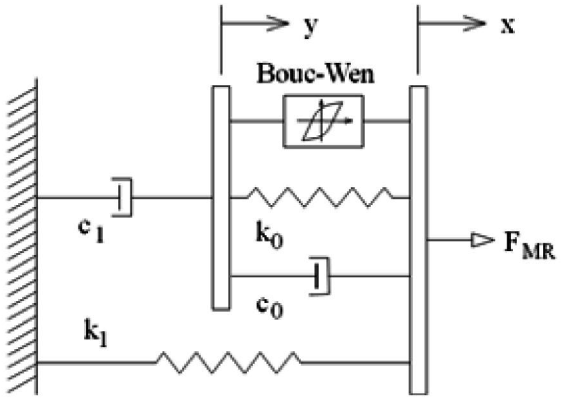



**Figure 8. Modified Bouc-Wen model for a MR damper.**

$$\begin{cases} \dot{y} = \dfrac{1}{(c_0 + c_1)} \left\{ \alpha z + c_0 \dot{x} + k_0 (x - y) \right\} \\ \dot{z} = -\gamma \left| \dot{x} - \dot{y} \right| z \left| z \right|^{n-1} - \beta (\dot{x} - \dot{y}) \left| z \right|^{n} + A (\dot{x} - \dot{y}) \end{cases} \tag{6}$$

where $z$ is the revolutionary variable related to the velocity $\gamma$ by the parameter $\alpha$; $F_{MR}$ is the predicted damping force; $k_1$ is the accumulator stiffness; $c_0$ is the viscous damping observed at larger velocities; and the parameters ß, $\gamma$ and A allow controlling the linearity in the unloading and the smoothness of the transition from the pre-yield to the post-yield region. The dashpot $c_1$ is included to produce the roll-off at low velocities; $k_0$ is used to control the stiffness at larger velocities; and $x_0$ is the initial displacement of spring $k_1$ associated with the nominal damper due to the accumulator.

All of the parameters in Eq. (6) may be expressed as Eq. (7).

$$\begin{cases} c_1 = c_{1a} + c_{1b} u \\ c_0 = c_{0a} + c_{0b} u \\ \alpha = \alpha_a + \alpha_b u \\ u = -\eta\ (u - v) \end{cases} \tag{7}$$

In Eq. (7), the damping constants $c_0$ and $c_1$ depend on the electrical current applied to the MR damper. The variable $u$ is the current applied to the damper through a voltage-to-current converter with a time constant $\eta$ and the variable $v$ is the voltage applied to the converter.

A simple MATLAB/SIMULINK block diagram can be used to simulate the Bouc-Wen model [29] of a MR damper as shown in **Figure 9**.

### 3.2 Set Up of the Device

To study the behaviour of a MR damper, some experiments were carried out on a MTS universal testing machine at the Mechanical Engineering Laboratory at FEUP, with the MR damper device RD-1005-3 supplied by LORD Corporation (**Figure 10**).

According to the device specifications, it has a capacity to provide a peak to peak force of 2224 N (500 lb) at a velocity of 51 mm/sec (2 in./sec) with a continuous current supply of 1 A. The MR damper was tested using the computer-controlled servo hydraulic MTS universal testing machine shown in **Figure 11**. The MR damper was attached to the MTS machine (operating under displacement control mode) and a 5 kN (1.12 kips) load cell was incorporated at the upper head to measure the force applied to the damper. The results were automatically collected by the computer-controlled MTS equipment and stored in a desktop computer.

After assemblage, the MR damper was forced with a sinusoidal signal at a fixed frequency, amplitude and current supply. To obtain the response of the MR damper under several combinations of frequencies, amplitudes and current supplies, a series of tests were carried out.

A set of frequencies (0.5, 1.0, 1.5 and 2.0 Hz), amplitudes (2, 4, 6, 8 and 10 mm (0.08, 0.16, 0.24, 0.31 and 0.39 in.)) and current supplies (0.0, 0.1, 0.2, 0.25, 0.5, 0.75, 1.0 and 1.5 A) were used in the testing program. In order to control and avoid temperature failure, especially at higher frequencies, a thermocouple was used to measure the external temperature of the MR damper. Typical results of this experimental research are shown in **Figures 12** and **13**.

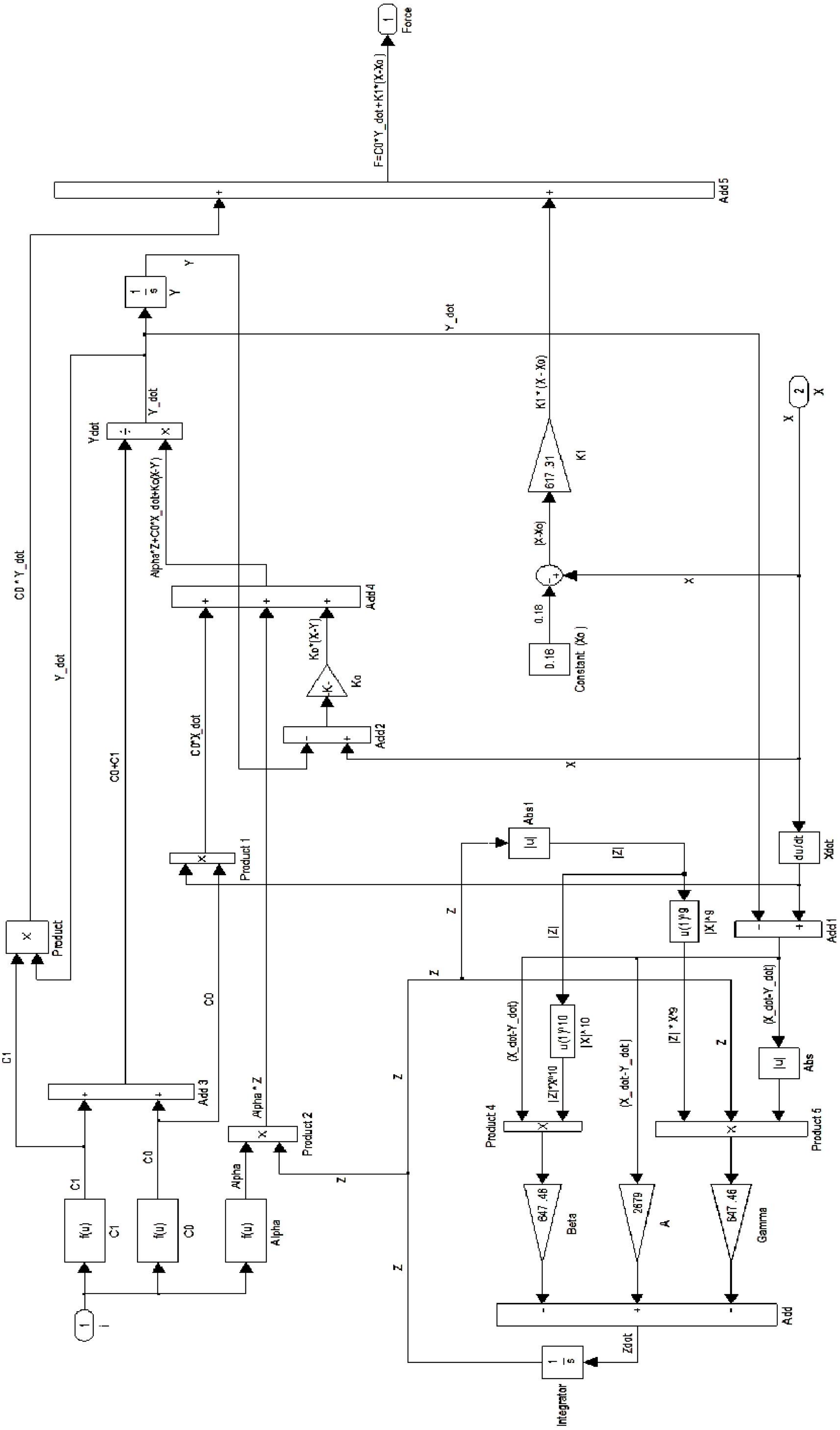
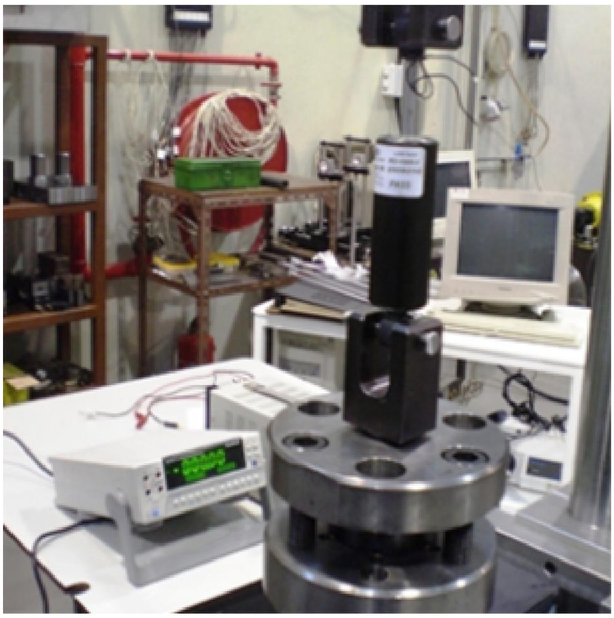



**Figure 9. Block diagram of a Magneto-Rheological damper using a Bouc-Wen model.**



| Parameter | Value |
|---|---|
| Extended length | 208 mm (8 in.) |
| Device stroke | ±25 mm (1 in.) |
| Max. Tensile force | 4448 N (1 kips) |
| Max. Temperature | 71ºC (160oF) |
| Compressed length | 155 mm (6 in.) |
| Response time | <10 ms |
| Max. Current supply | 2 A |

**Figure 10. Magneto-rheological damper RD-1005-03 test setup.**

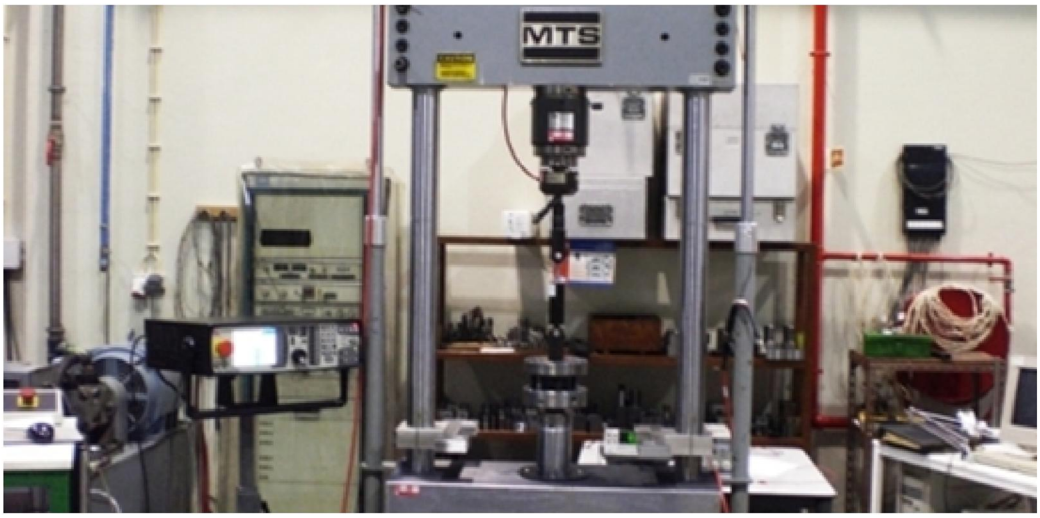
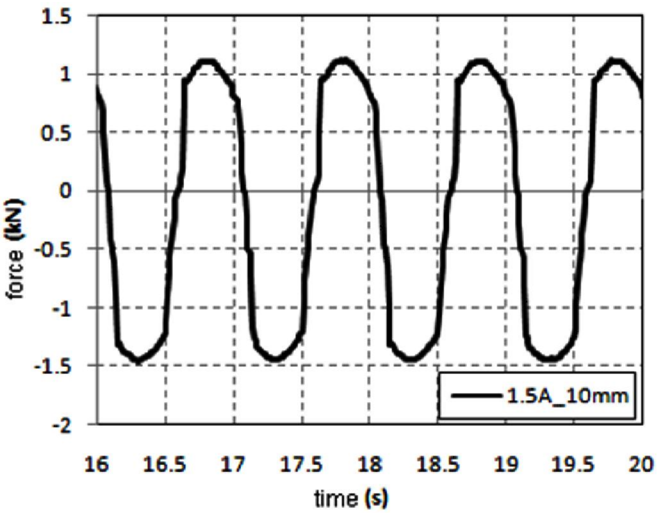
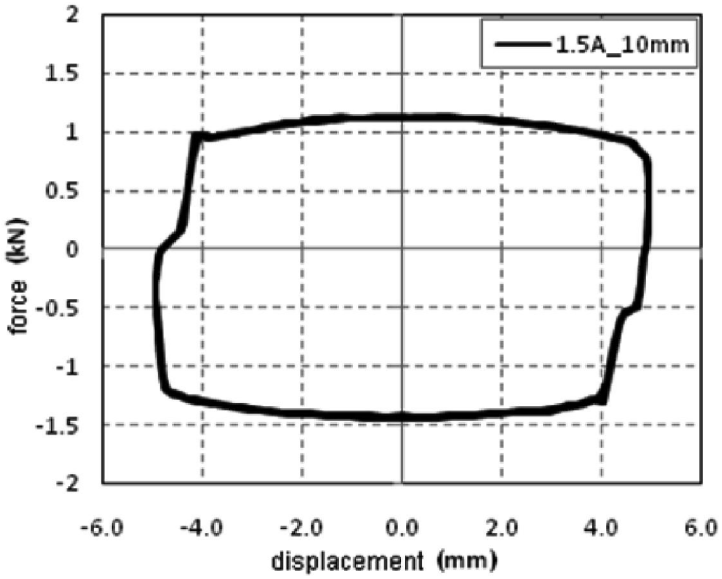



**Figure 11. Magneto-rheological damper with current supply device connected to MTS universal setup.**




Note: 25.4 mm = 1 in.; 4.448 kN = 1 kips

**Figure12. MR damper RD-1005-03 force-time history (1.5 A and 10 mm (0.39 in.)).**




Note: 25.4 mm = 1 in.; 4.448 kN = 1 kips

**Figure 13. MR damper RD-1005-03 force-displacement curves (1.5 A and 10mm (0.39 in.)).**

The variable current tests demonstrate that increasing the input current implies an increase in the force required to yield the MR fluid and a plastic-like behaviour is observed in the hysteretic loop. The frequency dependent test showed that the maximum damping force increases with the frequency due to large plastic viscous force at higher velocity. The frequency dependent test showed that the maximum damping force increases with the frequency due to large plastic viscous force at higher velocity.

## 4. SET UP AND PLAN OF EXPERIMENTAL DYNAMIC CAMPAIGNS ON SCALED STRUCTURAL FRAMES EQUIPPED WITH PASSIVE, SEMI-ACTIVE AND HYBRID DEVICES

### 4.1 Set Up of the Experimental Campaign

The study of the experimental dynamic behaviour of a scaled metallic load frame with passive and semi-active devices, namely BI devices and MR dampers (**Figure 14**), is under development by performing tests at the FEUP-COVICOCEPAD Lab, with the help of Quanser shake table II (**Figure 15**), as the dynamic loading actuator.

The experimental research employs three control strategies: (1) a passive control based on base isolation devices; (2) a semi-active control based on a MR damper assembled to the structure; (3) a hybrid control technique through the association of the base isolation devices with the MR damper. To study the semi-active control strategy, a MR damper is placed at the first floor level attached to the frame and rigidly attached to the shake table. The structure is excited using the N-S component of the 1940 El Centro earthquake as the seismic ground acceleration, time-scaled with Froude similarity according to the shake table model, in order to obtain the frequency characteristics of the structure and the physical model parameters.

The equation of motion that describes the behaviour of a controlled building under an earthquake load is given by

$$M\ddot{x} + C\dot{x} + Kx = -\Gamma f - M\lambda\ddot{x}_g \tag{8}$$

where $M$ is the mass matrix; $C$ is the damping matrix; $K$ is the stiffness matrix; $x$ is the vector of floors displacements; $\dot{x}$ and $\ddot{x}$ are the floor velocity and the acceleration vectors, respectively; $f$ is the measured control force; λ is a unit vector; and Γ is a vector that accounts for the position of the MR damper in the structure.

This equation can be rewritten in the state-space form as

$$\dot{z} = Az + Bf + E\ddot{x}_g \tag{9}$$

$$y = Cz + Df + v \tag{10}$$

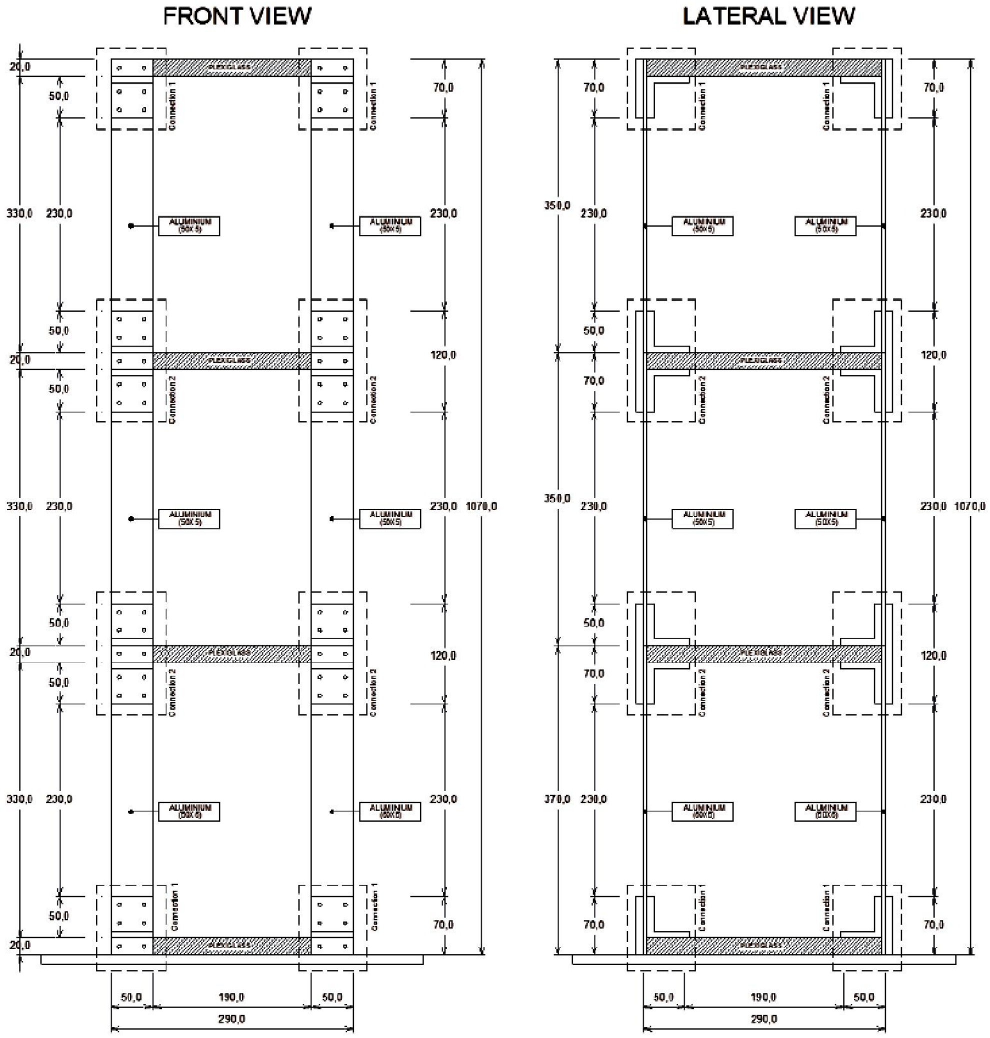



**Figure 14. 3-DOF small metallic frame at FEUP – COVICOCEPAD Lab.**

**Figure 15. Quanser shaking table II (and its controllers) at FEUP – COVICOCEPAD Lab.**

where $z$ is the state vector, $y$ is the vector of measured outputs; and $v$ is the measurement noise vector. The other matrices and vectors are defined by

$$A = \begin{bmatrix} 0 & I \\ -M^{-1}K & -M^{-1}C \end{bmatrix} \qquad B = \begin{bmatrix} 0 \\ M^{-1}\Gamma \end{bmatrix} \quad E = -\begin{bmatrix} 0 \\ \lambda \end{bmatrix}$$
$$C = \begin{bmatrix} -M^{-1}K & -M^{-1}C \\ I & 0 \end{bmatrix} \qquad D = \begin{bmatrix} M^{-1}\Gamma \\ 0 \end{bmatrix} \tag{11}$$

To control this semi-active structure based on a control force determination, the absolute acceleration of some relevant selected points in the structure, the displacement of the control device, and the control force are measured.

## 4.2 Selected Algorithms for the Tested MR Damper

After calibrating the MR damper numerical model, it is necessary to select a proper control algorithm to efficiently use this device in reducing the dynamic response of structural systems. The control strategy depends on the MR damper model selected to simulate the nonlinear hysteretic behaviour of this device [30, 31].

The fundamental condition to operate the MR damper is based on a generated damping force that is related with the input voltage; the control strategy is selected so that the damping force can track a desired command damping force. The available models can be categorized in static and dynamic models. The basic difference between them is that the static models do not include dynamic relation between input and output.

As mentioned in the above, this work is based on the Bouc-Wen model [29] that can represent the hysteresis dynamics explicitly. Therefore, an efficient control algorithm must be developed or chosen from the available literature to correctly characterize the intrinsic MR fluid behaviour, maximizing the MR damper characteristics as a semi-active control device.

In the last few years, several approaches have been proposed and intensively studied for better selection of the input voltage that must be applied to the MR damper to achieve the maximum performance. Among the proposed strategies, the following are the most studied: Lyapunov based control; decentralized bang-bang control; LQG (linear quadratic Gaussian) or clipped-optimal control; $H_2$/LQG control; fuzzy control and also artificial neural network (ANN) control strategies. Some of these control strategies will be applied in this research program in order to understand the pros and cons of each strategy. After performing an exhaustive study on the numerical model and small mock-up experimental model, the adoption of some reliable vibration control algorithms to medium-scale experimental models (whose basic strategy is resumed in the following section) is in course of development.

### 4.4.1 Semi-active control strategy based on Clipped-Optimal Control

A semi-active clipped-optimal control based on acceleration feedback was proposed by Dyke et al. [28]. The control diagram of this control strategy is shown in **Figure 16**.

As shown in Figure 16, this strategy has two controllers and is based on a linear optimal controller designed to set the command signal to zero or to the maximum level. The signal is selected according with the desired command force $F_d$, and the comparison between this force and the effective MR damper force. In this case, the command signal can be computed as

$$F_{MR} = \begin{cases} F_d, & F_d \cdot \dot{x}_{dev} < 0 \\ 0, & otherwise \end{cases} \tag{12}$$

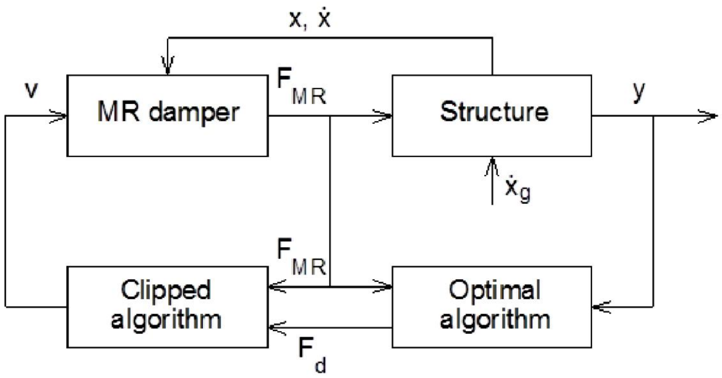



**Figure 16. Semi-active clipped-optimal control diagram.**

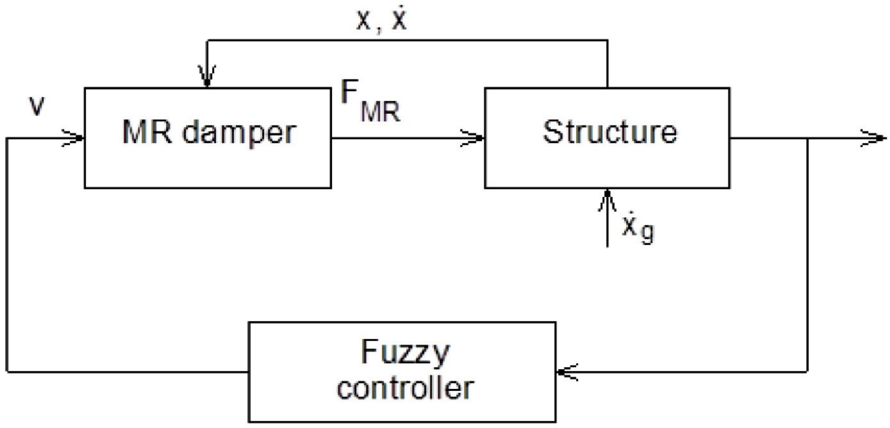



**Figure17. Semi-active clipped-optimal control diagram.**

where $F_{MR}$ is the control force of the MR damper and $\dot{x}_{dev}$ denotes the velocity of the damper device.

### 4.4.2 Semi-active control strategy based on Fuzzy control

One of the recent semi-active control strategies is based on the fuzzy control inference. As shown in **Figure 17**, this strategy has only one controller.

The fuzzy algorithm is used to compute the command signal and the voltage is selected using a fuzzy rule inference. The model can much easier accommodate uncertainties of input data and structural response sensors. This is a more reliable strategy since it guarantees the bounded-input/ bounded-output stability of the controlled structure [31].

## 5. EXPERIMENTAL DYNAMIC CAMPAIGNS ON SCALED RIGID STRUCTURES PROTOTYPES EQUIPPED WITH LIQUID DAMPERS

### 5.1 Passive Control Using TLD

Among the many vibration control devices available to increase the damping characteristics of the structures, TLD offer several advantages, such as low cost, easiness to install in existing structures and effectiveness even for small-vibrations [4-7, 11-13, 32-34].

The performance of TLD relies mainly on the sloshing of liquid at resonance to absorb and dissipate the vibration energy of the structure. The liquid is contained in partially filled tanks mounted on the structure. The shear force $F_{TLD}$ caused by the inertia of the liquid mass reduces the structural response due to the excitation action $F_g$ (**Figure 18**).

Tuning the natural frequency of liquid sloshing with the natural frequency of structure, results in optimization of the effectiveness of damper.

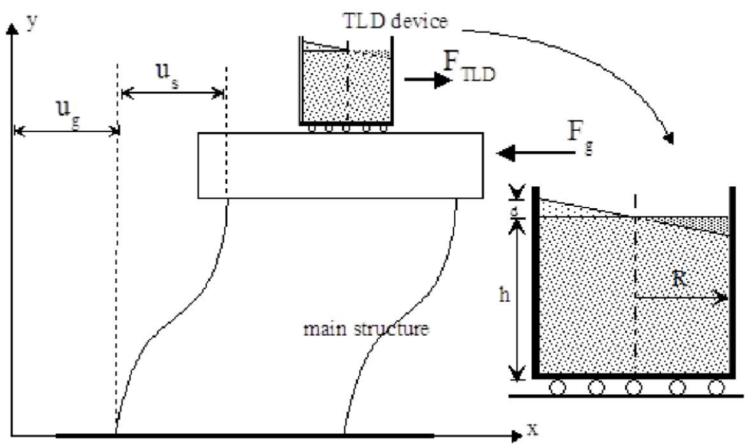



**Figure 18. SDOF shear frame equipped with TLD device.**

Theoretical models of liquid sloshing in TLD can be obtained either from a mechanical analogue, or from more exact analytical models of the structural and liquid domain. A reliable prediction of dynamic response of control devices based on fluid motion plays a central role for better understanding of real perspectives offered by TLD for applications in the field of intelligent structures, as regards to mitigation of earthquake and vibration hazards through vibration control.

The need of predicting and preventing failures associated to rocking and overturning of rigid structures undergoing strong ground shaking have motivated a number of studies on rocking response of rigid blocks and the possibility of coupling sloshing devices for attenuating their rocking response to dynamic excitations appears interesting.

Laboratory tests developed in the Department of Structural Engineering at the University of Naples on metallic blocks (**Figure 19**) equipped with sloshing dampers prototypes and located on a shake table show the potential benefits produced by these devices in attenuating the primary model response [32]. Experimental data in **Figures 20-22** show that significant beneficial effects can be produced in the attenuation of the rocking motion; this is apparent, for example, from the diagram of the curves depicting the empty/full ratio for the model equipped with a trapezoidal tank (**Figure 19**): the ratio between the model responses for the empty tank and for the tank filled with 4 cm (1.57 in.) or 8 cm (3.15 in.) of water shows the performance of the device, whose effectiveness is mainly lumped on a given frequency range. As it appears, the level of the liquid in the tank is able to deeply change the characteristics of the device and, therefore, its overall response.

The effectiveness of the device can be appreciated by comparison with the unit-ratio line. Time histories also confirm the advantage of adopting such devices in the considered frequency range, showing a good mitigation of the dynamical response of the primary model.

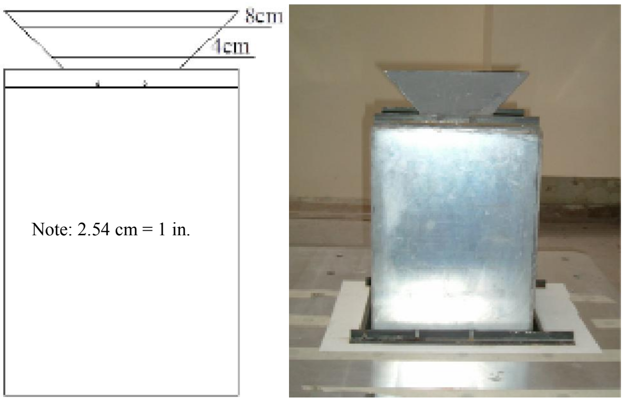



**Figure 19. Block equipped with 45° tank marked at two liquid levels (4 cm and 8 cm (1.6 in. and 3.1 in.)).**

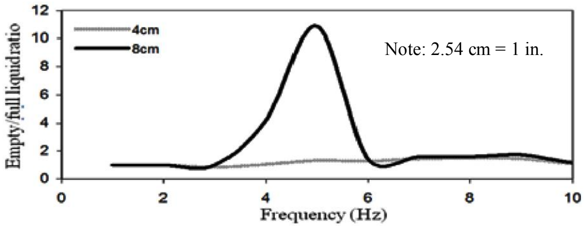



**Figure 20. Empty/full liquid ratio for the trapeizoidal tank placed on the 30×30×40 cm (12x12x16 in.) block tank.**

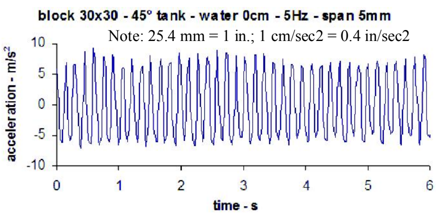



**Figure 21. Block 30×30×40 cm (12x12x16 in.) equipped with a trapezoidal (45°) empty tank.**

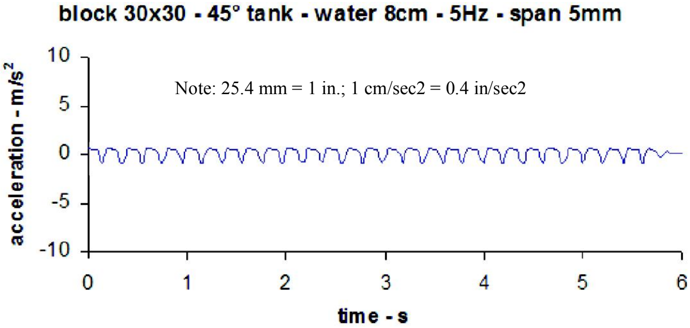



**Figure 22. Block 30×30×40 cm (12x12x16 in.) equipped with a trapezoidal (45°) tank filled with 8 cm (3 in.) of water.**

## 6. DEVELOPMENT OF ALGORITHMS FOR SEMI-ACTIVE CONTROL OF BRIDGES

Seismic protection of bridges involves different types of control with distinct characteristics. Semi-active control is one of the new fields of research in the area, using controllable semi-active devices that have the potential to achieve a performance close to active devices using less energy. Following an algorithm, the semi-active device is capable of modifying its dynamic characteristics in order to improve the global system response. The key factor that determines the full application of that potential is the choice of the right control algorithm [35, 36].

In the scope of COVICOCEPAD, a research program was performed in order to develop a set of control algorithms. In those algorithms, the input force on the device depends both on the velocity and the displacement of the bridge deck.

The force calculation is based on an algorithm that uses a special combination of displacement, velocity and control criteria causing a considerable improvement of the system response. The control algorithms were tested in a system subjected to two different actions: a simple harmonic function in resonance with the model and a generated accelerogram according to the Portuguese code.

The results obtained were compared with the no protection case and using a passive device, demonstrating the significant advantages of this semi-active control algorithm.

In order to evaluate the behaviour of a seismic protection in a structural system, a parameter reflecting its influence has to be included in the dynamic equation of motion. Therefore, in Eq. (13), $\Gamma$ corresponds to the force generated in the seismic protection device. While $\Gamma = 0$ is in the situation of no protection. For different kinds of seismic protection, $\Gamma$ will take different functional configurations

$$m\ddot{x}(t) + c\dot{x}(t) + kx(t) + \Gamma = F(t) \tag{13}$$

The semi-active control function developed follows Eq. (14), with the functions $f$ and $g$ defined by Eqs. (15) and (16). Function $f$ depends on the velocity (in the time instant in analysis) the maximum velocity response of the last three iterations and the parameter F, which is discussed latter. In function $g$, the displacement in the time instant in analysis, the maximum displacement response of the last three iterations and the parameter G are joined together with the coefficient $C_1$. This coefficient ($C_1$) varies between -1 and 1, depending if the structural displacement and velocity have the same sign, as stated in Eq. (17). As the algorithm was created with the purpose to be implemented in variable damping devices, such as magneto-rheological devices or variable-orifice devices, in each iteration, $\Gamma$ has the same sign as the term $c\dot{x}$ of Eq. (13)

$$\Gamma = \frac{|f(\dot{x}) + g(x)|}{|\dot{x}|} \times \dot{x} \tag{14}$$

$$f(\dot{x}) = |\dot{x}_{\max 3}| F\dot{x} \tag{15}$$

$$g(x) = |x_{\max 3}| C_1 G x \tag{16}$$

where $\dot{x}_{max3}$ is the maximum velocity response for the last three iterations; $x_{max3}$ is the maximum displacement response for the last three iterations; and

$$C_1 = \begin{cases} 1 & if\ x\dot{x} > 0 \\ -1 & if\ x\dot{x} \le 0 \end{cases} \tag{17}$$

The parameters F and G should be studied for each structure model in analysis and for each seismic action to be considered. These parameters allow the adjustment of the algorithm in order to improve the structural response. To the purpose of comparison with an equivalent passive situation, F and G should be such that Eq. (18) is verified for every time instant.

$$\Gamma \le F_{\max Passive} \tag{18}$$

Where $F_{max\ Passive}$ is the maximum force of the passive device.

The velocity-dependent term (function *f*) of the algorithm provides a regular seismic protection similar to a damper ($F = cx^{\alpha}$), having the maximum force at the time of maximum velocity (or when displacement equals zero) and forming an elliptical shape in the force versus displacement chart (**Figure 23**). The displacement-dependent term (function *g*) runs in the opposite way: the force is maxima for maximum displacement (or when velocity equals zero), as seen in **Figure 24**. It implies that this term completes the elliptical shape referred, turning the sum into a rectangle (**Figure 25**), which results in maximizing the dissipation cycle and consequently reducing the structural response.

In **Figure 26**, the comparison is made between the seismic response of a viaduct located in the southern region of Portugal, with and without control. In the semi-active control device, four different sets of characteristic values were tested for the studied control algorithms.

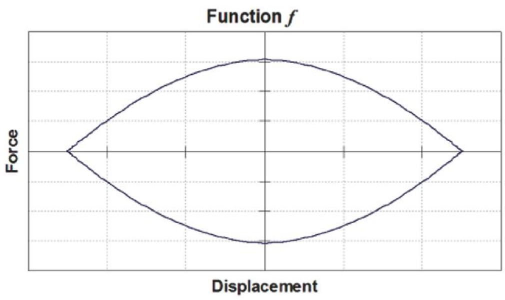



**Figure 23. Force vs. displacement of function** $|f(\dot{x})| \mathrm{x} \frac{\dot{x}}{|\dot{x}|}$

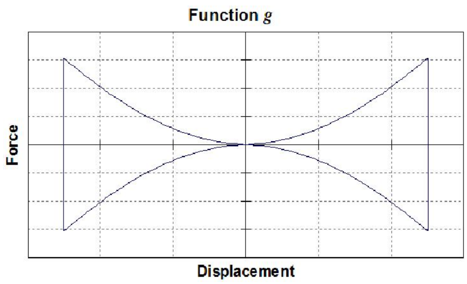



**Figure 24. Force vs. displacement of function** $|g(x)| \mathrm{x} \frac{\dot{x}}{|\dot{x}|}$

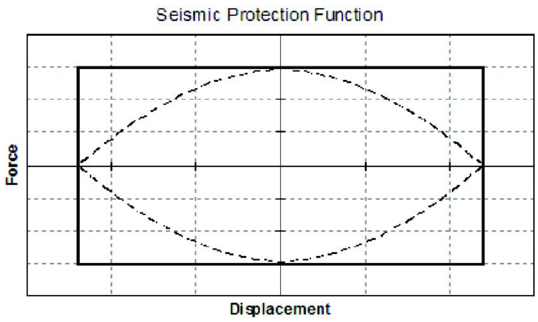
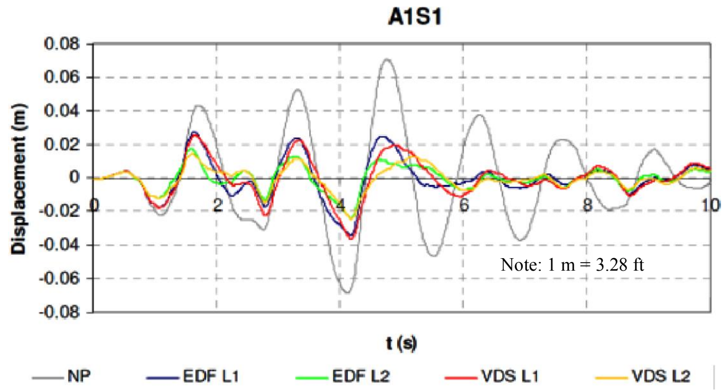



**Figure 25. Force vs. displacement of function** $\Gamma$.




NP – No Protection (without control)
EDF L1, EDF L2, VDS L1, VDS L2 – Different solutions of control algorithms

**Figure 26. Comparison between the semi-active control and no protection seismic response.**

## 7. STEEL FRAME FOR SHAKE TABLE TESTS AT LNEC

As mentioned earlier, the scaled frame at FEUP is under testing as a small mock-up structure at the QST of FEUP to assess passive and semi-active control of vibration methodologies and algorithms using base isolation and MR devices.

With the experience gained during the study, further structural vibration tests for the COVICOCEPAD project are planned and/or under development at a larger scale at the LNEC shake table. These include the use of TLD and MR devices placed on a higher scale steel frame whose dynamic characteristics are already known as it has been analyzed during previous applications while studying in detail the characteristics of the LNEC shake table [37, 38].

The structural elements of this test steel frame were designed through an elastic analysis with the finite element program SAP-2000NL for two earthquake time history accelerations, both with peak ground acceleration of 0.5g: one was a simulated earthquake according to type 1 seismic action of the Eurocode-8 [39]; and the other was the 1940 El Centro earthquake.

The El Centro time history is a low frequency shake with most of its energy in the frequency range below 8 Hz (periods above 0.125 s), hence it is suitable to test the specimen at its natural frequency. Moreover, this earthquake, even if is not ideally suited to the role of the standard time history, is useful to test any shake table up to its performance limits.

In **Figures 27** and **28**, the 1940 El Centro time history of acceleration and the corresponding response spectrum versus natural period are plotted.

On the other hand, the choice of the simulated earthquake according to type 1 seismic action of Eurocode-8 (EC-8) was justified by exciting the frame with a maximum force at its natural frequency (in this case, about 8 Hz). As a result, for this frame, the simulated earthquake according to type 1 seismic action is more suitable compared to the simulated earthquake according to type 2 seismic action.

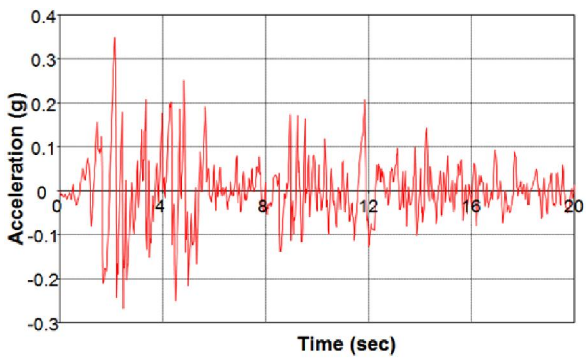



**Figure 27. Acceleration time history of 1940 El Centro earthquake.**

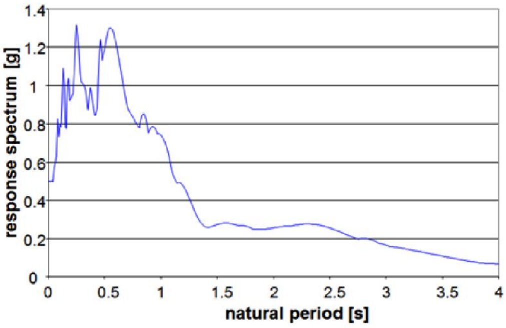


**Figure 28. Response spectrum of El Centro earthquake.**

On the other hand, the choice of the simulated earthquake according to type 1 seismic action of Eurocode-8 (EC-8) was justified by exciting the frame with a maximum force at its natural frequency (in this case, about 8 Hz). As a result, for this frame, the simulated earthquake according to type 1 seismic action is more suitable compared to the simulated earthquake according to type 2 seismic action.

**Figures 29** and **30** represent the acceleration time history for a simulated earthquake (according to type 1 seismic action of Eurocode-8) and the corresponding response spectrum.

According to design requirements the main dimensions of the frame are given in **Table 1**.

The longitudinal and transverse dimensions were chosen according to the platform limits for shake table tests as well as the height of the column and the inter-storey distance. The slab floors can be 3 m (9.8 ft.) spaced, allowing the possibility of adding another slab floor and consequently another degree of freedom simply using structural bar elements with the same characteristics and details of the basic structure. The main dimensions of the structural elements are presented in **Figure 31**.

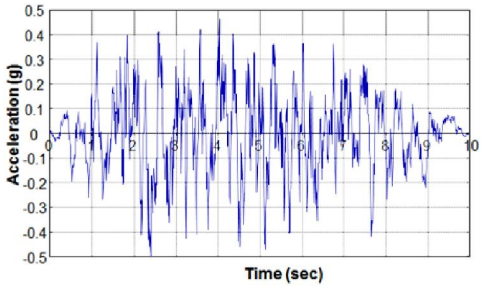


**Figure 29. Time history acceleration of the simulated earthquake according to type 1 seismic action of EC-8.**

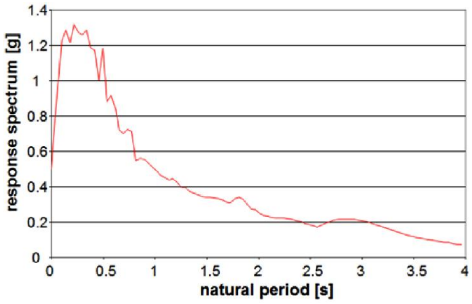
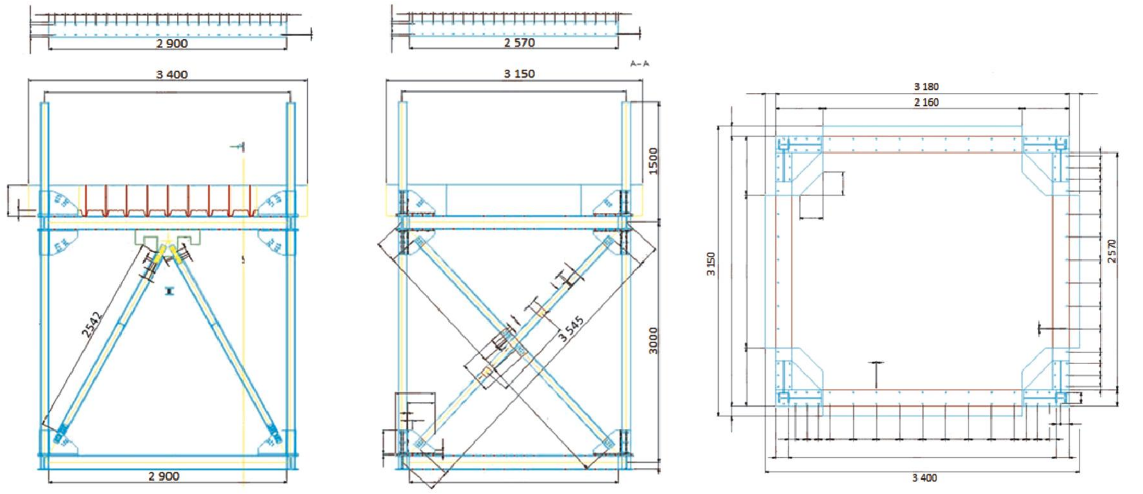


**Figure 30. Response spectrum for the simulated earthquake according to type 1 seismic action of EC-8.**

**Table 1. Dimensions of steel frame**

| | Length m (ft) |
|---|---|
| Inter-storey distance | 3.00 (9.8) |
| Column height | 4.50 (14.8) |
| Longitudinal length | 3.00 (9.8) |
| Transverse length | 2.75 (9.0) |




Note: All dimensions are in mm; 25.4 mm = 1 in.

**Figure 31. General overview of the specimen, plan, longitudinal and transverse overviews.**

The choice of the steel sections was based on a compromise between flexibility and maximum admissible displacements during the tests, because it is necessary to have a structure with natural frequency in a useful range for experimental tests (between 6 and 20 Hz) but with bounded displacements not exceeding the imposed limit by the actuator capacity.

Therefore, columns and beams were respectively chosen as HEB 100 and HEB 180 steel sections, while the K and X braces were chosen as HEB 100 and UPN 100 sections. For the latter case the choice was imposed to allow the cross of the braces. The steel selected is of grade S355.

After this first choice of the main structural elements steel sections, the frame was verified through an elastic analysis with the SAP-2000NL programme, assuming the passive device with high yield strength and hence unable to dissipate energy. The maximum internal forces on the main elements through this analysis are summarized in **Table 2**.

All the previously selected steel sections satisfy the demands of resistance for stresses and buckling phenomena. Further details of the elements are given in **Figure 32**.

The connections between the braces and the devices were verified for the maximum forces that the passive-control device bears and were ensured with bolts of grade 10.9. The connections between the braces and the base structural elements were designed with the same construction details, while the joints between the brace and column or beam plates were accomplished by welding. Some details of these connections are shown in **Figure 33**.

With this configuration, the frame weight was calculated and the choice of the thickness of the slab was made in order to reach approximately a value of 10 tons. The slab is constituted by sheet metal and concrete properly connected, but during the concrete casting the slab should be shored up in order to reduce the lowering of the slab in the setting phase.

The slab is shown in **Figure 34** and the details of the casting are shown in **Figure 35**.

Shake table tests on the frame in **Figure 36**, already tested at the National Technical University of Athens (NTUA), and equipped with different type of control devices are planned and/or in course of development.

**Table 2. Maximum internal forces in main structural elements**

| Element | Axial load kN (kips) | Bending moment kN.m (k.ft) | Shear load kN (kips) |
|---|---|---|---|
| Column | 114.0 (25.6) | 6.1 (4.5) | 9.7 (2.2) |
| Beam | 37.7 (8.48) | 6.6 (4.9) | 24.1 (5.4) |
| Longitudinal brace | 100.1 (22.5) | 0.6 (0.4) | 0.4 (0.09) |
| Transverse brace | 60.1 (13.5) | 0.3 (0.2) | 0.4 (0.09) |

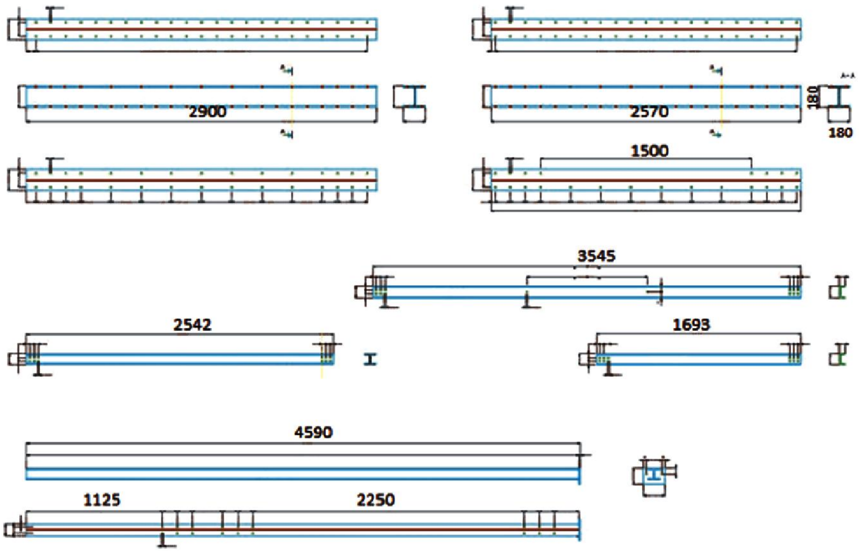



Figure 32. Details of steel structural elements: beams, columns and braces.

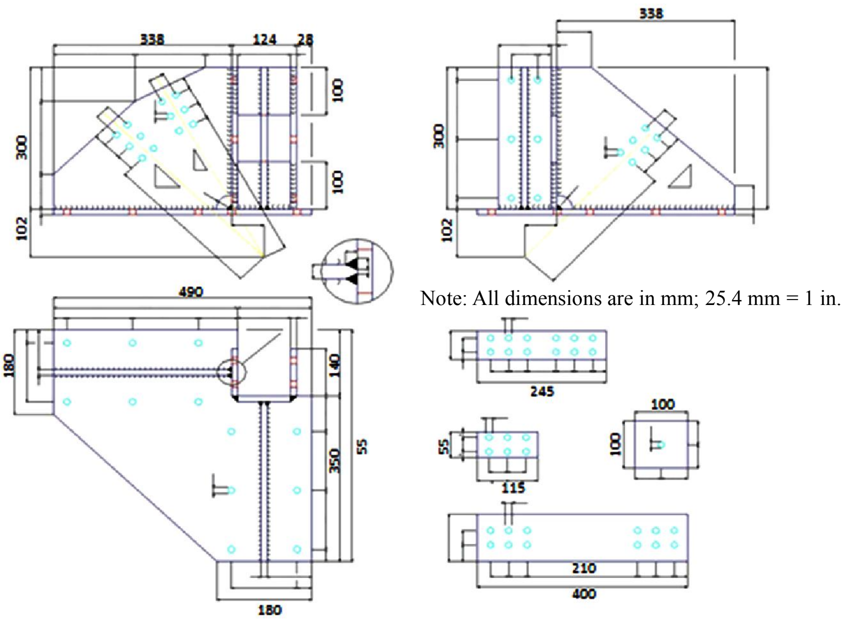



Figure 33. Details of the connections and the plates.

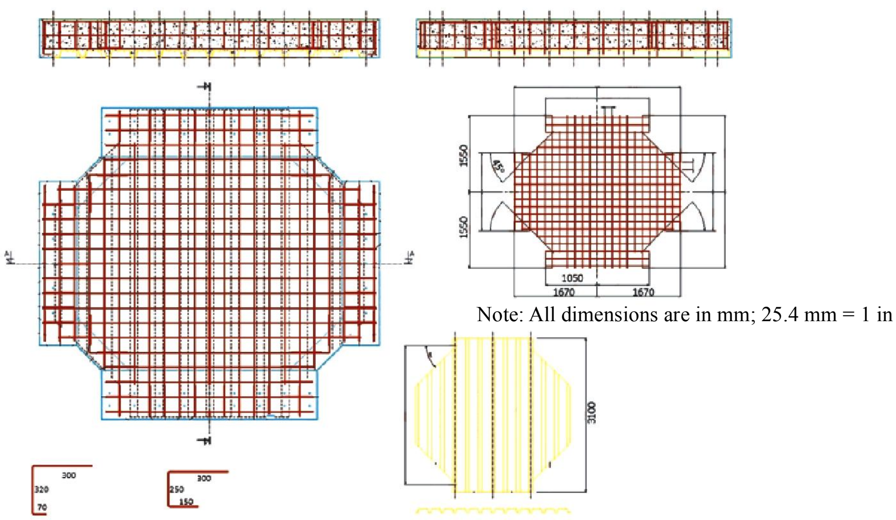



Figure 34. Plan and section views of the floor slab.

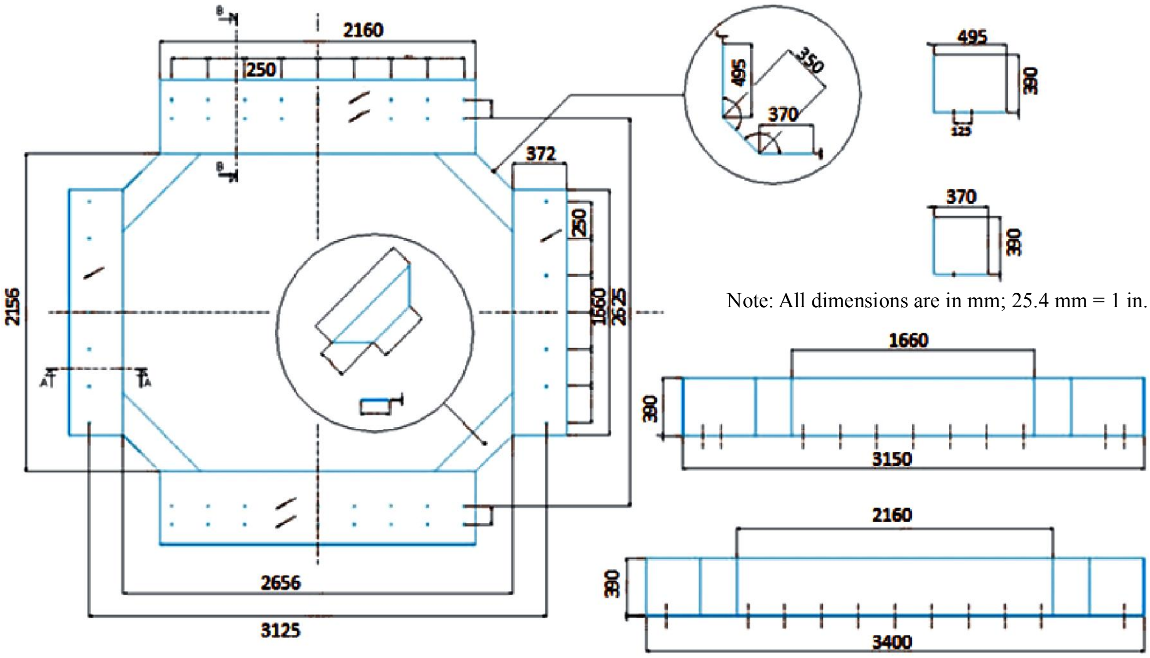



**Figure 35. Casting details of the floor slab.**

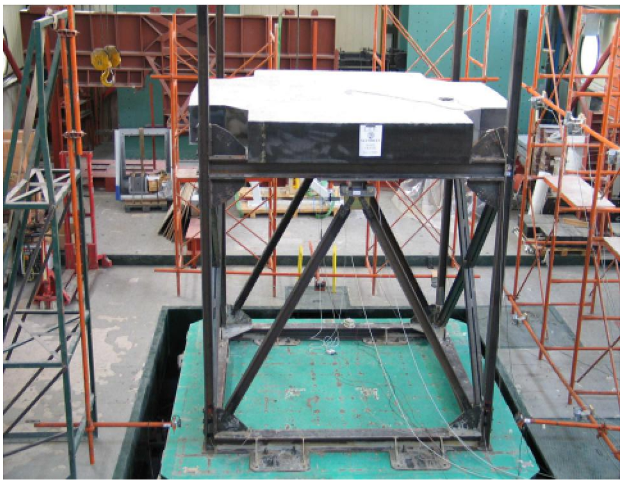


**Figure 36. The steel frame during performance tests at NTUA.**

## 8. CONCLUSIONS

The paper represents a report on the ongoing experiences and activities carried out by an international team of experts in the field of structural control, consisting of a number of European research units and laboratories.

The results are summarized concerning the design and numerical testing of base isolation devices which are able (for example, the device designed at the University of Naples), to account for the characteristics of the site treated as a filter, allowing obtaining a significant improvement in the performance of the classical isolation system of the structure.

The modelling and experiments executed on an MR device in laboratory at FEUP are reported which describes the set up of tests in course of development on the related semi-active actuation system, and on other passive and hybrid control devices applied on a steel frame prototype placed on a shake table facility.

The results are reported deriving from laboratory experiments performed on structures exhibiting a non-linear behaviour during the shake due to a purely rocking motion, whose oscillations are controlled by introducing some devices exploiting the dissipative skill of liquid tanks. The application shows that TLD appear promising even in the case when the structure basically reacts to the dynamic solicitation as a rigid block.

Moreover, some algorithms designed for the control of vibrations in bridge decks are synthesized, showing a significant improvement in terms of reduction of the structural dynamic response that can be achieved by adopting such control strategies.

Finally, experimental programmes for different types of devices, to be mounted on a scaled steel frame placed on a shake table, are mentioned. These specify the design characteristics of the frame and the shake table facility.

In conclusion, the summarized results provide detailed insight in the continued research activities that appear of interest in the field of structural control.

**ACKNOWLEDGEMENTS**

This work reports research developed under the R&D Eurocores Project COVICOCEPAD within the S3T Program, approved independently by European Science Foundation (ESF, Strasbourg), financially supported by portuguese "FCT – Fundação para a Ciência e a Tecnologia" (Lisbon – Portugal) under Programa Operacional Ciência e Inovação 2010 (POCI 2010) of the III Quadro Comunitário de Apoio funded by FEDER, and also by Italian "CNR – Centro Nazionale della Ricerca" (Rome-Italy), under the EC Sixth Framework Program.

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