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Faculty of Engineering, University of Porto

INTEGRITY RELIABILITY AND FAILURE CHALLENGES AND OPPORTUNITIES

Editors:

J.F. Silva Gomes

Shaker A. Meguid

IRF'2009

INTEGRITY, RELIABILITY AND FAILURE (CHALLENGES AND OPPORTUNITIES)

Editors

J.F. Silva Gomes and Shaker A. Meguid

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CHAPTER XXIII

SYMPOSIUM

DYNAMICS AND STABILITY

Coordinated by

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IRF'2009

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Editors

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Introduction to Symposium on Dynamics and Stability

Throughout the years there have been numerous studies on Dynamics and on Stability of elastic structures, a topic of concern to civil engineering as well as to mechanical and electrical engineering and applied science in general. The thematic of this seminar pretends to encompass such general topics as those related to static buckling of structures or of their individual members or of complex mechanical systems, to actual areas of concern and development in structural dynamics and control of vibrations in engineering. Additionally this seminar also addresses the stability of structures under general dynamic loads, which are sensitive to initial imperfections and hence prone to catastrophic buckling failures; the analysis of these latter structural systems are based upon definitions of dynamic buckling and on estimates and assessment of dynamic loads, as well as on single-mode and generalized criteria for dynamic buckling.

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REF: S2601_A0236

AN ORIGINAL APPROACH TO BUCKLING OF BEAM/COLUMNS UNDER CONSERVATIVE LOADING

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SYNOPSIS

The paper presents an original approach to stability of prismatic beam/columns subjected to conservative loading, *eg.* Euler column. The slender beam/column, namely column for which the critical state is defined by critical force, is modelled as elastic structure with possible compressing deformation. The load is applied statically and the stability analysis is applied by means of static approach. The main points of the presented model are: the column preserve its stable bent shape at Euler critical load, existence of buckling is explained on the base of energy considerations. The mathematical description of the model follows the equations of energy balance. The results are verified experimentally.

DESCRIPTION OF THE MODEL

Considering a column subjected to compressive axial load one observe that for relatively small loads the deformation is shortening and for relatively large loads the deformation is bending. It follows from the Euler solution that the transition from straight shape to the bent one appears at the critical Euler load. In the proposed herein model, this transition appears at load much lower than Euler force and appears for load which is defined by energy balance. At this load column finds it easier to keep bent shape than straight one because the energy of bending is lower than energy of compressing. The detailed measurement [2] shows that the first buckling displacement, appears at load $P_0 \approx P_E \sqrt{3}$, where P_E is the critical Euler load. So at P_0 the load reaches its real buckling value and we call it the real bifurcation point.

Moreover, in the discussed model the shortening of the column axis remain at the same level as at P_0 , even for $P > P_0$. This value is called critical shortening of the column axis and is equal to $\varepsilon_k = \pi^2 / \lambda^2$, where λ is the slenderness ratio and. Notice that, in the considered herein elastic range of deformation, the value of the critical shortening is independent on the material properties as Young modulus, depends only on the slenderness ratio λ .

Let us consider the loading with the force $P > P_0$. Until the critical Euler load compressing proceeds with the stable bent shape. What is important, exactly at Euler load the shape of column remains the stable bent too. There is no bifurcation of equilibrium at the Euler load, herein this state is determined by the energy of elastic deformation which reaches the its maximum value equal to U_H . This energy is defined as a function of material features and properties of the shape of cross-section.

$$U_H = \frac{1}{2} \sqrt{R_H^3 / E \pi A i}, \quad (1)$$

where R_H is the proportional limit, E - Young modulus, A - cross-section and i - radius of inertia. Moreover, the state at Euler load is defined by energy balance. The relation between energy of bending and energy of compressing can be written as

$$2U_{zg} + U_{sc} = U_H \quad (2)$$

where energy of bending $U_{zg} = \frac{M_{gk}^2 l}{2EJ}$ and energy of compressing $U_{sc} = \frac{P_E^2 l}{2EA}$. The relation (2) is verified experimentally, see [2] and references there.

Due to the fact that in the presented model, do not exists a critical point which is defined as a bifurcation of equilibrium, as in stability analysis of column, the structure is called beam/column.

The crucial point of the actual model is finding, that for the loads being in the range $P_0 > P > P_E$ the column remains in the stable bent shape, the values of deflection can measured and calculate. The load-deflection curve, in this range, is shown in Fig.1 where the values coordinates of points are taken from experiment, see [2] and references there. First deformation occurs at $P_0 \approx P_E \sqrt{3}$, then deflections arises up to point 10, which relates to Euler load P_E .

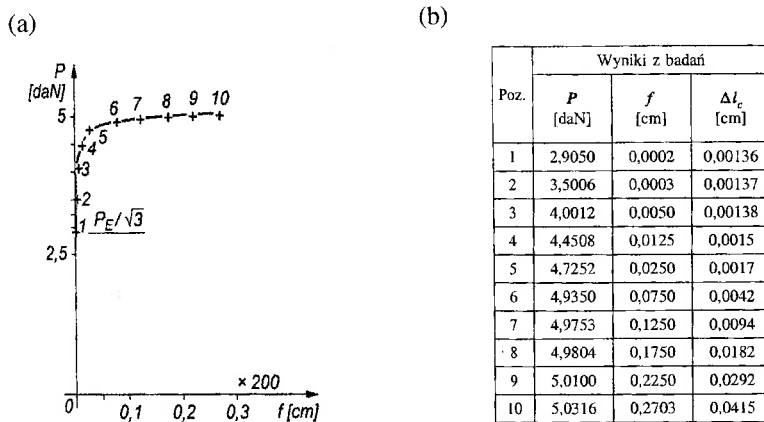


Fig.1. a) Load-deflection curve b) Experimental data: P – compressing force, f – deflection, Δl_c – shortening of column axis

For loads higher than P_E the column suddenly bows out laterally. Plastic strains and the material strengthening occurs. The value of Euler load divide the ranges of elastic and elasto-plastic deformation. For load lower than P_E deformation are elastic whereas for load $P > P_E$ plastic deformation occurs the along the bar axis.

Acknowledgment:

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REF: S2602_A0568

THE GUSSET PLATE EFFECT IN STEEL PLATE SHEAR WALL SYSTEMS

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SYNOPSIS

A steel plate shear wall dual system consists of a moment resisting frame, plus thin infill shear panels. In the literature, it is stated that the ultimate lateral load bearing capacity of such a dual system can be evaluated by a simple addition of the discrete frame, brought about by the formation of plastic hinges and that of detached infill panel evaluated when yield patterns form along their diagonal tension folds. However, more recent studies show that there is an interaction effect between the wall and the frame and that the combined system can resist additional loads. In other words, the principle of superposition does not apply to this dual lateral load resisting system. This super added value may arise from two factors; a) the influence of beam and column rigidities on the buckling and ultimate load bearing capacity of panel, and b) the influence of panel on the frame. This paper studies the latter effect, otherwise known as the gusset plate effect in shear walled frames.

INTRODUCTION

A steel plate shear wall (SPSW) is a lateral load resisting system, which consists of vertical steel plate infill walls connected to surrounding beams and columns and installed in one or more bays along the full or partial height of structure to form some kind of a cantilevered wall. SPSW subjected to cyclic inelastic deformations exhibit high initial stiffness, behave in a very ductile manner and dissipate significant amounts of energy. These characteristics make them suitable to resist seismic loadings. SPSW not only can be used in the design of new buildings but also in retrofitting existing construction.

It is assumed that steel panels in SPSW systems experience shear deformations when the structure is subjected to lateral loads. According to some research studies, the ultimate lateral load bearing capacity of SPSW is evaluated by simple addition of the ultimate capacity of the discrete frame brought about by the formation of plastic hinges, plus that of detached infill panels derived as yield zone forming along the diagonal tension field. However, the super added value arising from interaction effect between surrounding frame members and infill panels is not accounted for. This super added value is mainly caused by two factors; a) the influence of surrounding member rigidities on the load bearing capacity of infill panel, and b) the effect of infill (or rather an active part of it) on the frame capacity. The interactive behaviour of the infill panel and surrounding members is extremely complex. It is only in recent years that developments in computers and numerical analysis have enabled researchers to study the nonlinear post-buckling behaviour of such systems up to failure. The objective of this paper is to evaluate the stiffening effect of the remaining post-yield active part of infill panel on the frame capacity, through an incremental nonlinear finite element analysis.

RESULTS

This study is carried out in two parts. At first, a single bay, single storey SPSW is analyzed under lateral loading. Thereafter, the two related components, i.e. the discrete frame and the detached infill panel were analyzed separately. Based on the results, it is observed that the infill panel partially yields before the frame. Yield points in infill panel soon spread along the diagonal tension field. The surrounding frame remains somehow ineffective until a diagonal yield zone is utterly formed inside infill. The ultimate capacity of individual frame was greatly improved by gusset effect and the initial stiffness of stiffened frame decreased as yield width grew. As illustrated in Figure 1, when the yield width was small, the ultimate capacity of stiffened frame was almost equal to the ultimate capacities of discrete frame plus detached modified infill panel, but for larger bands the ultimate stiffness dropped below the stiffness of the modified detached infill panel.

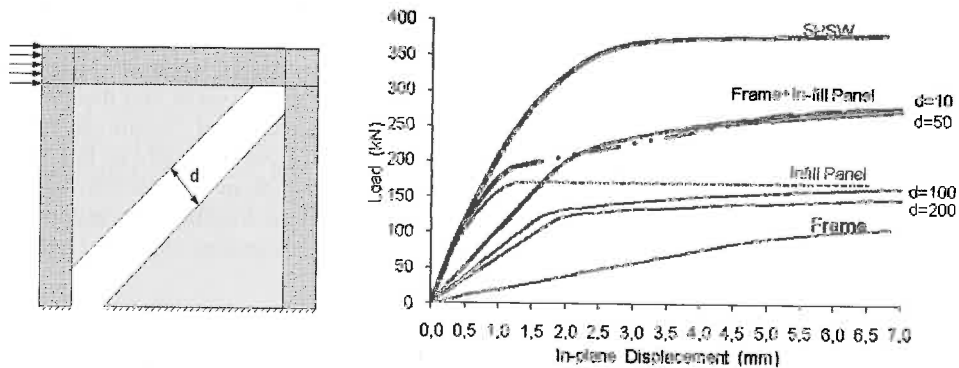


Figure 1: Load vs. in-plane displacement of SPSW and stiffened frames

CONCLUSIONS

The nonlinear post-buckling behaviour and ultimate strength of a single bay single story steel plate shear wall with specific regards upon the bilateral effects between boundary frame members and infill panel was studied. First, it was concluded that there was an interaction effect between frame and infill panel. The ultimate capacity of SPSW was higher than the ultimate capacity of the sole frame and the detached infill.

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REF: S2603_A0580

HARMONIZING EFFECTIVE LENGTH K-FACTORS BETWEEN EUROPEAN AND AMERICAN CODES OF PRACTICE

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SYNOPSIS

In this paper a comparison is made between European (EC2 and EC3) and American (ACI 318 and AISC 360) codes of practice with regard to the effective length K-Factors in assessing critical loads of slender columns. Discrepancies between the codes are apparent since the evaluation of an individual column end restraints is quite different, except between the American codes. What this means, however, is that the derivation of the expressions used in the calculation of K-Factors are fundamentally different. This paper presents a method of comparison between each code which is based in finding a common link in the assessment of end restraints and finds that the effective length which is obtained is in essence the same, apart some numerical errors.

INTRODUCTION

The concept of effective length is a useful tool in individual stability checks of columns in multi-storey frames. It essentially is a mean of comparison between the critical load of a member subject to any type of end restraints and its corresponding theoretical Euler load. As such, the assessment of the K-Factors depends solely on the column's end restraints. Since the exact equations to calculate the effective length are implicit transcendent expressions it becomes difficult to quickly reach a solution. European codes offer approximate numerical fittings to the exact equations and the American codes offer alignment charts to overcome this fact. ACI goes so far as to present approximate expressions which contradicts its own alignment charts (equal to AISC's), but since these equations are at odds with the exact formulation of the problem presented in the charts, they will not be addressed in this paper. Although different in nature, all of the following equations attempt to characterize the end restraints of a column.

$$\eta_i = \frac{\sum \left(\frac{I}{l}\right)_c}{\sum \left(\frac{I}{l}\right)_c + \sum \left(\frac{I}{l}\right)_B} \quad k_i = \frac{\theta}{M} \cdot \frac{EI}{l} \quad G_i = \frac{\sum \left(\frac{EI}{l}\right)_c}{\sum \left(\frac{EI}{l}\right)_B} \quad \psi_i = \frac{\sum \left(\frac{EI}{l}\right)_c}{\sum \left(\frac{EI}{l}\right)_B}$$

EC3 (1) EC2 (2) AISC (3) ACI (4)

This paper derives the relations between these expressions and compares them in terms of the resulting K-Factor showing that they are essentially the same.

RESULTS

The main result of the analysis is that the numerical correlations of the European codes adjust well to the solutions given by the AISC's exact equations. Also, as one can see from figure 1, the EC3 expressions are overall more conservative than EC2's.

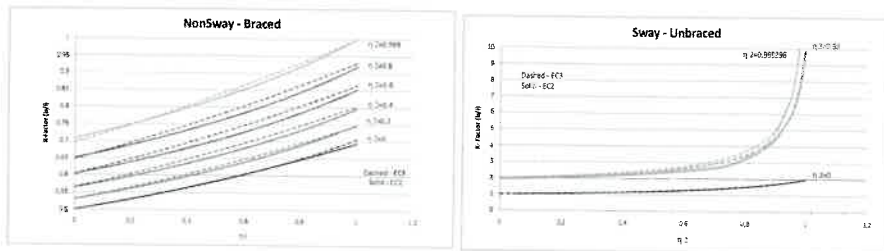


Figure 1 Comparison between EC2 and EC3 expressions on K-Factors for Non-sway and Sway Columns

The maximum difference between these two codes reaches 3% for non-sway columns and 15% for sway columns. Since both procedures are a result of equations adjusted to transcendental expressions, one can state that the observed differences are a result of numerical adjustments.

Comparison between these two codes and AISC (and therefore also ACI) is made in the full version of this paper on a qualitative basis. To this effect, charts are plotted, that are very similar to those provided by Annex E of Eurocode 3, which provide the variation of both end restraint indexes for a given K-Factor. Since the difference between the end restraint indexes given by each code is not representative of the difference between their respective K-Factors, a comparison such as the one provided by Figure 1 is not possible.

CONCLUSIONS

The results obtained allow for the conclusion that the critical buckling loads of individual columns assessed in each code of practice are very closely related and are practically the same, apart some numerical errors.

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REF: S2604_A0581

A PARAMETRIC STUDY ON A RC FRAME BASED ON "PUSHOVER" ANALYSIS

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ABSTRACT

The work developed herein is being done in the context of a Master of Science thesis, fundamentally an application of a series of pushover analyses on two-dimensional RC frames, which are part of an office building. Due to its simplicity, the structural engineering profession has been using the nonlinear static procedure. Since inelastic behavior is intended in most structures subjected to strong earthquake loading, the use of nonlinear analyses is essential to capture behavior of structures under such extreme seismic effects.

The purpose of this work consists on a detailed parametric study, varying the number of stories and its height, and also the bay width. The values assumed for the bay width and for the stories height are 5, 6 and 7 meters and 3, 3,5 and 4 meters, respectively.

Regarding the stated objective, several commercial packages universally used in the design of civil engineering structures were used, namely SAP 2000, SEISMOSTRUCK and MIDAS.

The current analysis will not be directly made over the final structure of the respective RC frame, whose structural skeleton is constituted by two bays and two stories. In this context, the study is divided in four steps, which goes from the simple RC frame with only one bay and story, extended to the final stage of the structure. As the RC frame is inserted in an office building, logically there are no masonry infill panels on the first floor. By the other side, on the second floor, the masonry infill panels are influent over the seismic response of the structure. In structures subjected to intensive lateral loads, masonry infill panels have much influence over the structure response, as occurs during an earthquake.

In order to represent the influence of the masonry infill panels, the "Equivalent Tie Method" will be used in the following work, which was proposed by Stafford Smith and Carter [1]. It will be also done a sensitivity study about the effects caused on the capacity curves, considering several values for the tie width, such as the proposed by Riddington and Stafford Smith [2] and finally the proposal presented by Paulay and Priestley [3].

The influence of other parameters, on the structural behavior of the RC frame, is also pretended to be analyzed. Those parameters are defined to be the confinement of the structural elements (columns and beams) and the length and location of the plastic hinges (Park and Paulay [4], Priestley and Park [5] and Priestley et al [6]) forming near the end of the structural elements, because bending moment is the predominantly force in the structure. Finally, it is also important to see the non linear material behavior of the structure when submitted to different load patterns, such as: uniform, modal and triangular. Only the two first ones are clearly suggested in the Eurocode 8.

Finally the conclusions of the study elaborated in this work are presented and future developments are pointed-out in order to deepen the pushover analysis over two dimensional RC frames.

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REF: S2605_A0582

NON LINEAR ELASTIC AND MATERIAL ANALYSES OF IMPERFECT CHS COLUMNS (STEEL TUBES AND CONCRETE FILLED TUBES)

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SYNOPSIS

In this article the strength capacity of CHS steel tubes is compared with that of similar CHS concrete filled steel tubes. The tubular columns were filled with two types of concrete: one of normal strength class C25/30, and the other of a higher strength class of C45/55. The tubes had initial imperfections of manufacture and handling, measured in stations along two perpendicular planes. Two types of analyses can be performed: a nonlinear elastic analysis (geometric non-linearity) and an elastic-plastic analysis (material non-linearity), in order to characterize strength capacity gains and deformation ranges.

NON LINEAR ELASTIC AND MATERIAL ANALYSES OF IMPERFECT CHS COLUMNS: *STEEL TUBES*

The columns analyzed were made of steel S235, external diameter of 0.50 m and thickness of 0.01 m, but with variable length to simulate different slenderness ratios.

As initial imperfections pattern, the maximum deformation at mid-length was attributed according to EC3 (Fig. 1). The buckling strength curves obtained were also compared with theoretical curves of Euler, Rankine-Gordon and with multiple curves of resistance (according to EC3 design code).

For columns with end eccentricities, three cases of eccentricity magnitudes (e) were analyzed for different fractions of the gyration radius (i): $e = i/10$, $e = i/20$, $e = i/40$; (Fig. 2). In any of these, the results are compared with those obtained with software TBCOL (Barros, 1983).

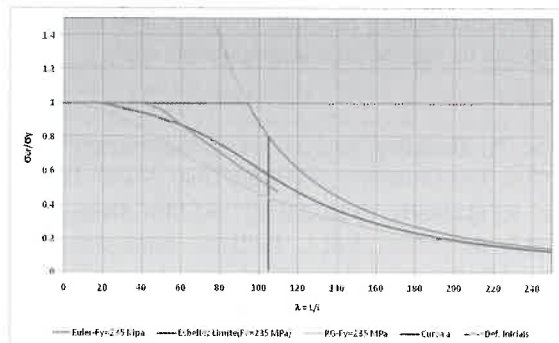


Fig.1 - Carrying capacity for columns with initial imperfections: Euler curve, Rankine-Gordon curve, Buckling curve 'a'

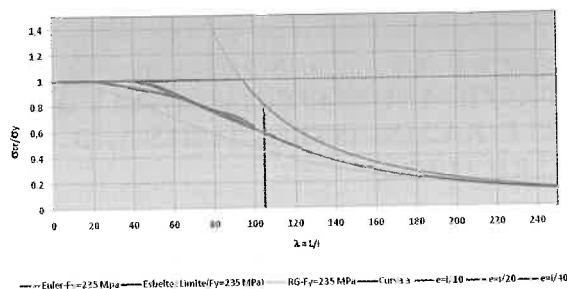


Fig. 2 - Carrying capacity for columns with load eccentricities: Euler curve, Rankine-Gordon curve, and theoretical curve

NON LINEAR ELASTIC AND MATERIAL ANALYSES OF IMPERFECT CHS COLUMNS: CONCRETE FILLED STEEL TUBES

Under this heading the non linear elastic behavior of three groups of CHS columns were compared: solely made of steel, and filled up with concrete of two resistant classes (C25/30 and C45/55). The solely steel column was analyzed by TBCOL while the filled CHS columns were analyzed using MIDAS software (Fig. 3). The columns analyzed were made of steel S235, external diameter of 0.50 m and thickness of 0.01 m, modeled with a total length of 13,8 m as it is possible in jacket offshore structures.

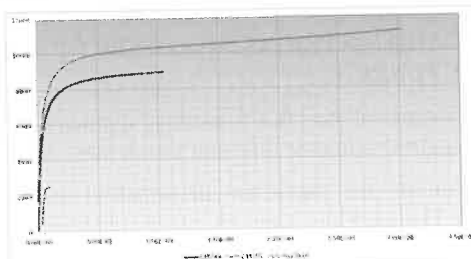


Fig.3 - Comparative carrying capacity of a modeled *steel* and *concrete-filled steel* long tube

Additionally, 18 concrete-filled steel tubes with lengths of 1,6-1,7-1,8 meters, with 2 concrete classes of resistance and with specific initial deformation patterns were also modeled in MIDAS software under non-linear elastic analysis. The material nonlinearity corresponding to elastic-plastic behavior is also assessed and wherever possible results are compared with column data of an experimental testing program at FEUP. The gain in capacity and rigidity for the different columns can be assessed by this parametric study.

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SOME RESEARCH ON CONTROL OF VIBRATIONS IN CIVIL ENGINEERING UNDER COVICOCEPAD PROJECT

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SYNOPSIS

This paper provides information on some R&D within COVICOCEPAD project approved in the framework of Eurocores program. It addresses the use of TMD's, TLD's, base isolation devices, MR dampers and a hybrid technique using both devices together. Some results are provided associated with calibration of a MR damper at FEUP, as well as its inclusion in a small scale laboratory set-up with proper equations of motion of the controlled smart structure. Applications of TMD's devices to two civil engineering structures under dynamic and seismic actions are also outlined.

PASSIVE CONTROL USING BASE ISOLATION (BI) OR TLD/TMD DEVICES

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 its effectiveness as mitigation approach for dynamic loading. However the limitations that these devices/methodologies have, encouraged the study and development of more advanced control systems based on active, semi-active or hybrid control devices (Figure 1).

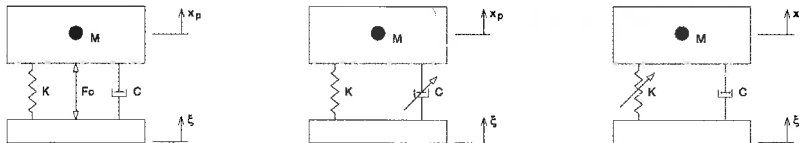


Figure 1: Active and semi-active vibration control strategies.

The classical design approach for the base isolation devices accounting for horizontal seismic component is also used. In this case the equation of motion (1) has the matrix identifications (mass, damping and stiffness matrices) given in equation (2), where c_b is the damping coefficient of the isolation system, c_s is the damping coefficient of the fixed structure and k_s is the stiffness of the fixed-base structure.

$$M\ddot{X}(t) + C\dot{X}(t) + KX(t) = -M\{I\}a(t) \quad (1)$$

$$M = \begin{bmatrix} m_b & 0 \\ 0 & m_1 \end{bmatrix}, \quad C = \begin{bmatrix} c_b + c_s & -c_s \\ -c_s & c_s \end{bmatrix}, \quad K = \begin{bmatrix} k_b + k_s & -k_s \\ -k_s & k_s \end{bmatrix} \quad (2)$$

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 2). Tuning the natural frequency of liquid sloshing with the natural frequency of structure, results in the optimization of the effectiveness of the damper (Barros and Corbi [1] [2]).

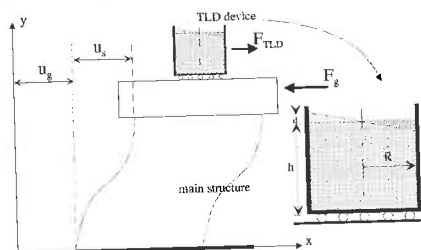


Figure 2: SDOF shear frame equipped with TLD device.

SEMI-ACTIVE CONTROL USING MR DEVICES

To study the behaviour of a MR damper some experiments were carried out on a MTS universal testing machine, of the Mechanical Engineering Laboratory at FEUP, with the MR damper device RD-1005-3 supplied by LORD Corporation (Figure 3). According to the device specifications it has a capacity to provide a peak to peak force of 2224 N at a velocity of 51 mm/s with a continuous current supply of 1 A (Barros et al. [3]).



Figure 3: Magneto-rheological damper RD-1005-03 test setup at FEUP.

Parameter	Value
Extended length	208mm
Device stroke	± 25 mm
Max. Tensile force	4448N
Max. temperature	71°C
Compressed length	155mm
Response time	<10ms
Max. Current supply	2A

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About the Book

As the engineering community continues to cross the boundaries of known practices, materials and manufacturing techniques into the frontiers of new functional materials, environments and applications, the opportunities for catastrophic failures will inevitably increase. If our knowledge of how to engineer systems, structures and components to minimize or prevent catastrophic failure is to keep pace with modern manufacturing technologies, the demanding applications, and the intolerance of a safety conscious society, we must continue our efforts to develop and use superior materials, apply reliable analytical techniques and validate these with sound experimental tools.

This book contains the Abstracts of papers presented at the 3rd International Conference on Integrity, Reliability and Failure, held at the Faculty of Engineering, University of Porto, Portugal, 20-24 July 2009. This is part of a prestigious series of Integrity Reliability and Failure conferences coordinated by the International Scientific Committee on Mechanics and Materials in Design. The conference attracted over 260 participants with 310 accepted academic and industrial submissions from 45 different countries around the world. These papers were presented in July 2009 in the magnificent city of Porto, Portugal, and the conference themes focused on design, nanotechnology, advanced materials, computational and structural mechanics, experimental mechanics, structural safety, energetic systems, and case studies.



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