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APPLICATION OF ELASTOMERIC DAMPERS TO STEEL MOMENT RESISTING FRAMES

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Abstract. *In this paper the hysteretic behaviour of elastomeric dampers and their effect on steel moment resisting frames was evaluated. A new constitutive model was proposed, based on the Generalized Maxwell Model (GMM), which was able to capture the characteristics of the dampers under dynamic loading in a range of frequencies and amplitudes. This model was incorporated into the Finite Element (FE) Software OpenSees [1], and was used to validate the dampers behaviour. A simple steel moment resisting frame was modelled, and analytically tested under under strong ground motions, scaled according to Eurocode 8 [2] response spectrum. Both analytical and experimental results show the efficiency and the effectiveness of the elastomeric dampers with regard to structural response.*

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1 INTRODUCTION

Although current anti-seismic concepts are based on linear elastic analysis, it is well known that under large seismic loads the structure will experience high non-linearities, and levels of damage. This is due to the dissipation of the seismic energy through the hysteretic behaviour of the structural members, and the actual non linear nature of the materials used, which lead to permanent deformations and plastic hinges. The traditional design concept consists of a combination of strength and ductility in order to provide resistance to lateral loads [3, 4]. For major earthquakes, the structural design engineer relies upon the inherent ductility of conscientiously detailed buildings to prevent catastrophic results [4]. This design approach is acceptable because of economic considerations provided, of course, that structural collapse is prevented and life safety is ensured [3]. However, sometimes the conventional seismic philosophy cannot easily be applied to all types of structures, the performance criteria may indicate the structure to remain linear or to have a minimal damage, while at the same time there are numerous structures with insufficient strength or ductility which need strengthening in order to cope under current seismic needs. Passive energy dissipation devices have the ability to improve the seismic behaviour of buildings[5, 6, 7, 8, 10] by reducing drift, force and deformation demand on the structural elements, which are responsible for providing lateral load resistance, in addition to reducing velocity and acceleration demands on non-structural components[9].

The evaluation of the hysteretic behaviour of two EDs, provided by TARRC (the Tun Abdul Razak Research Centre, UK), is the main focus of this paper. Elastomers, like viscoelastic (VE) material highly depend on the strain amplitude, frequency, and ambient temperature. However, Lee[5] showed that elastomeric dampers are less sensitive to frequency, a fact which makes them even more effective. Their capability of hysteresis energy stems from shear deformation, since the material is bonded between steel plates (Fig.1)

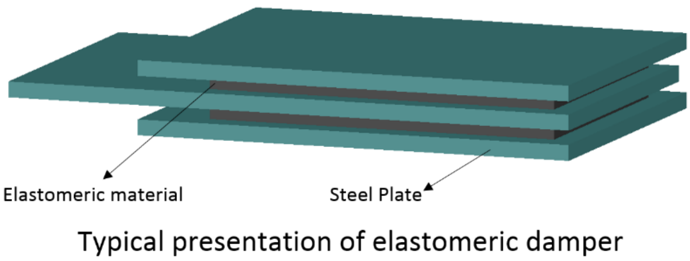

Figure 1: Typical Elastomeric Damper.

A time-domain constitutive equation has been developed, based on the Generalized Maxwell Model, taking into account non linearities. Sinusoidal strain commands at different amplitudes, and frequencies consisted the basis of the characterization tests which were used to calibrate this model, which was also incorporated into FE OpenSees[1], to capture the seismic response of a simple moment resisting frame. Five ground motions were used to test the dynamic response of the structure, which were scaled in frequency domain in order to match the response spectrum of EC8. Finally, comparative figures and tables are presented to show the efficiency of the elastomer.

2 ELASTOMERIC DAMPER AND CHARACTERIZATION TESTS

Two elastomeric dampers were provided by the Tun Abdul Razak Research Centre (TARRC). They consist of a rubbery material bonded between steel plates. The overall dimensions of the rubber material are 230 mm in the longitudinal direction (coinciding with the loading direction), 180 mm in the transverse direction, and 11.75 mm width (Fig. 2).

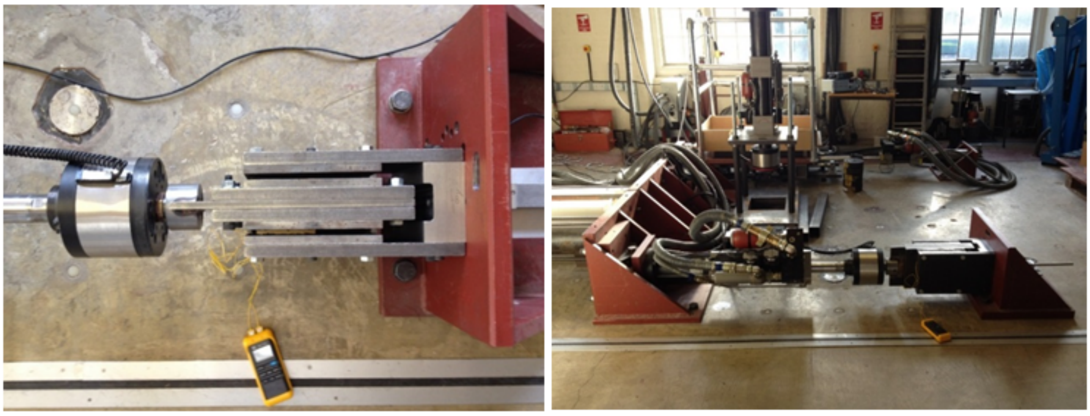

Figure 2: Experimental Rig.

A 100 kN servo-hydraulic actuator was used to evaluate the elastomer's characteristics under a range of displacement amplitudes and frequencies. The two dampers were both connected with a central steel plate, which was in turn connected with the actuator and provided the total movement of the dampers, while the outer steel plates were fixed (Fig.2). Therefore, the two dampers were tested simultaneously in a symmetric arrangement and the whole hysteretic behaviour was performed through shear deformation. A thermocouple was also attached to the elastomer in order to measure the ambient temperature, but also to capture any increase in temperature during the tests. It is well known that rubber-like materials exhibit non linear behaviour that depends on strain amplitude, ambient temperature, loading frequency, and loading history [5]. Hence, the experimental tests were focused on verifying non linear dependant on these parameters. Among these parameters, it is has been shown that strain dependence is the dominant factor [5, 11].

In order to investigate the strain amplitude and frequency dependence a series of tests was carried out based on sinusoidal displacement histories at the EDs. In order to avoid large initial displacement and velocity ramping cycles were implemented. Therefore, each history had 6 ramped cycles and 20 full sinusoidal cycles. This process was repeated for frequencies 0.25-4.0Hz, and for strain amplitudes 10%-50%. The maximum strain amplitude was decided to be kept within 50%, to avoid any potential permanent damage, severe cracking, or debonding of the elastomer. Fig. 3 shows a representative example of a sinusoidal displacement command of 40% strain amplitude and frequency of 3Hz. The tests were conducted at room temperature (19^{o}C-20^{o}C) in order to minimize the effect of temperature variation.

Based on the stress-strain hysteretic loops (Fig. 4), created for each frequency and amplitude, the equivalent shear modulus, G_{eq}, and loss factors, n_{eq}, were obtained for every test. These parameters are considered to be standard evaluation parameters of passive dissipation devices. The equivalent shear modulus, G_{eq}, is associated with the maximum magnitude of damper stress,$\tau_{\text{D,max}}$, and the maximum damper strain, $\gamma_{\text{D,max}}$ as:

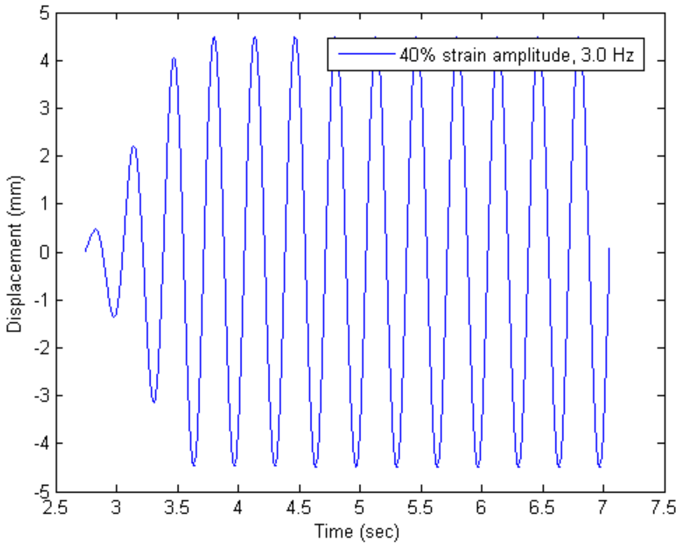

Figure 3: Typical sinusoidal displacement history used for the characterization tests of the elastomeric damper.

$$G_{eq} = \frac{\tau_{Dmax} - \tau_{Dmin}}{\gamma_{Dmax} - \gamma_{Dmin}} \tag{1}$$

where $\tau_{\mathrm{D,min}}$ is the minimum damper stress, and $\gamma_{\mathrm{D,min}}$ is the minimum damper shear strain. The loss factor, n_{eq}, can be determined as follows:

$$n_{eq} = \frac{1}{2\pi}\frac{E_d}{E_s} \tag{2}$$

where E_{d} is the energy dissipation, and E_{s} is the strain energy. E_{d} can be determined by integrating the hysteresis loops, and E_{s} can be calculated from the maximum strain and the stress corresponding to maximum strain. Although the first two cycles have higher stiffness, no significant stiffness degradation is observed within the remaining eighteen full cycles. Hence, E_{d} and E_{s} were calculated when the stress versus strain hysteresis loops were stabilized. The final values of G_{eq} and n_{eq} were summarized and are presented in Table 1 and Figs 5 and 6.

Table 1: Mechanical Properties of the EDs

Shear Strain %	f=0.25Hz		f=0.5Hz		f=1Hz		f=2Hz		f=3Hz		f=4Hz	
	Geq (Mpa)	**neq**	**Geq (Mpa)**	**neq**	**Geq (Mpa)**	**neq**	**Geq (Mpa)**	**neq**	**Geq (Mpa)**	**neq**	**Geq (Mpa)**	**neq**
10	1.676785	0.4072	1.656063	0.4140	1.77003	0.4094	1.911551	0.4020	2.002003	0.3982	2.067903	0.3944
20	1.295131	0.3551	1.258352	0.3520	1.344416	0.3535	1.43804	0.3504	1.506741	0.3489	1.559415	0.3475
30	1.08946	0.3335	1.074266	0.3330	1.139418	0.3344	1.225987	0.3359	1.27763	0.3332	1.321105	0.3322
40	1.046107	0.3390	0.990114	0.3262	1.05134	0.3298	1.121038	0.3306	1.166776	0.3286	1.208876	0.3269
50	0.945939	0.3010	0.937152	0.3044	0.986017	0.3082	1.049216	0.3104	1.094721	0.3115	1.132333	0.3090

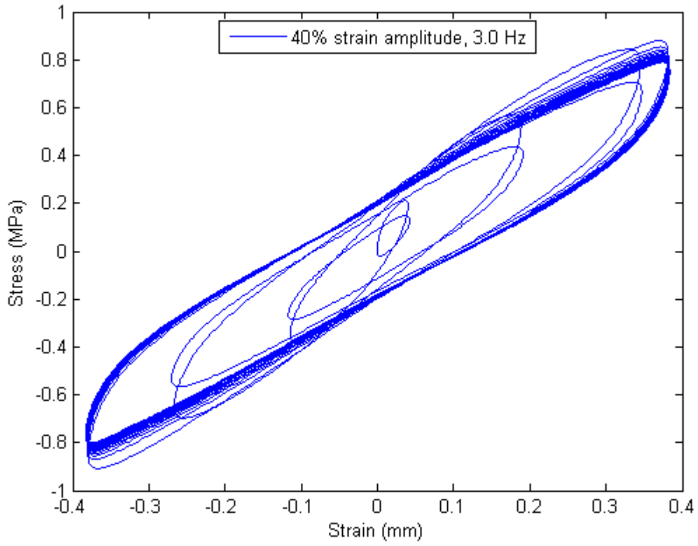

Figure 4: Stress-Strain Hysteretic loop.

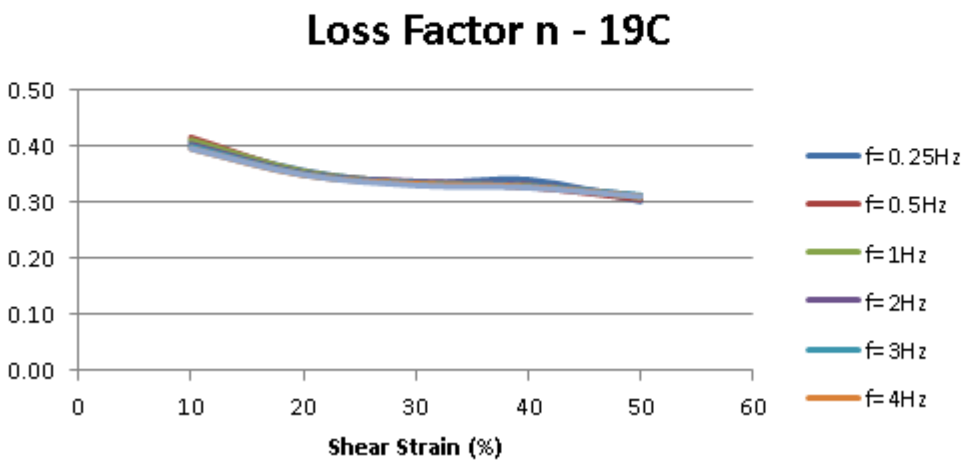

Figure 5: Loss factor Frequency and Amplitude Dependence.

It can be clearly seen that the dominant factor which affects the elastomers dynamic behaviour is the amplitude and not the frequency, especially in the case of loss factor where it practically remains the same regardless any change in frequency. It is also noticeable that when the strain amplitude increases above 30% the shear storage modulus tends to a constant value.

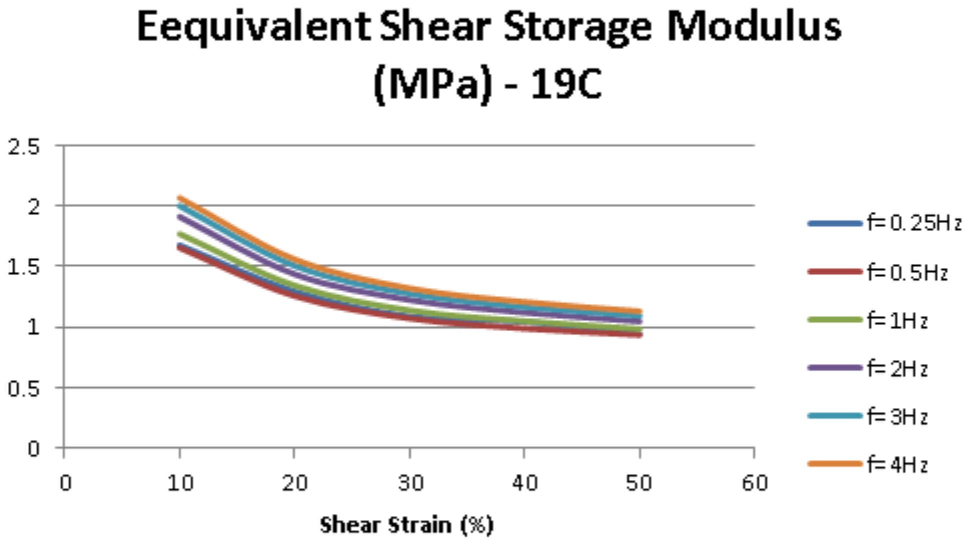

Figure 6: Shear Storage Modulus Frequency and Amplitude Dependence.

3 HYSTERETIC MODEL FOR ELASTOMERIC DAMPER

As discussed and showed earlier, the elastomer's dynamic behaviour depends on the frequency and strain amplitude. In order to capture the material's characteristics several models have been proposed until now by researchers, either based on Fractional Derivatives [12], or Bouc-Wen model [9, 13, 14], while rate-dependent and rate-independent model was the main focus of Lee [5]. Moreover, Chopra [15] tried to model a non-linear damper in time domain. However, the conventional way of modelling elastomeric behaviour is the one that FEMA [16] proposees, where an equivalent stiffness and loss factor is assumed for the elastomer based on the natural frequency of the building.

3.1 Linear Generalized Maxwell Model

The proposed model for the hysteretic behaviour of the ED is based on the well known Generalized Maxwell Model (GMM). GMM has been proposed as numerical representation of VE materials [17, 18] and can be represented graphically in the following figure:

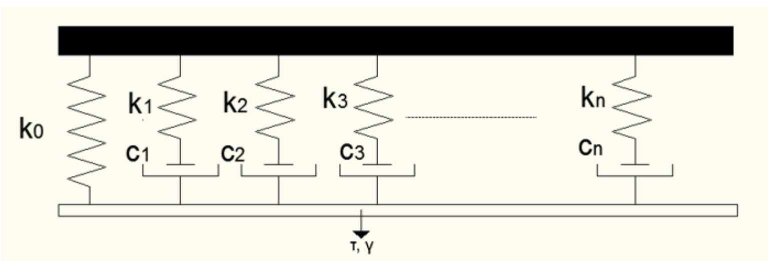

Figure 7: ELinear Generalized Maxwell Model (GMM).

The linear GMM consists of linear spring, with spring stiffness parameter called k_0, in parallel with N Maxwell elements. Although, it is very easy to extract the analytical force-displacement relationship when only one Maxwell element is used, things are becoming much more complicated when more than two elements are being used. Even though an analytical expression has been developed when N Maxwell elements are used, this is not the purpose of this paper, since only one Maxwell element is going to be used in this case. The time relaxation for the i_{th} Maxwell element is denoted, τ_{i}, and is equal to $k_{\mathrm{i}}/c_{\mathrm{i}}$. In the case of one Maxwell elements the total force is:

$$F = F_0 + F_1 \tag{3}$$

From Eq. (3):

$$F_1 = F - F_0 \tag{4}$$

where F is the total force applied at the GMM. For every individual Maxwell element, i, the following equation can be derived:

$$F_i' = k_i u_i' - \frac{k_i}{c_i} F_i \tag{5}$$

where F´ and u´ denotes the differentiation of the force and the displacement with respect to time. The first derivative of (3) gives:

$$F' = F_0' + F_1' \tag{6}$$

Substituting (5) into (6) for i=1:

$$F' = F_0' + k_i u_1' - \frac{k_1}{c_1} F_1 \tag{7}$$

In the GMM case, u1= u. Hence, substituting from Eq. (4), Eq.(7) becomes:

$$F' = F_0' + k_1 u_1' - \frac{k_1}{c_1} F + \frac{k_1}{c_1} F_0 \tag{8}$$

which leads to:

$$F + \frac{c_1}{k_1} F' = F_0 + \frac{c_1}{k_1} F_0' + c_1 u_1' \tag{9}$$

3.2 Modified Generalized Maxwell Model

It was noticed that GMM was only adequate to describe only VE behaviour, while also was proved to be inefficient when the frequency or the amplitude was changing (It was only able to capture VE behaviour only for specific values of frequency and amplitude). In order to take into account the non linear part of the elastomer, a non linear damper was added, while F_0 consists of a linear spring. Karavasilis [9] proposed a modified stiffness based on the maximum displacement to take into account the softening of the elastomer:

$$k_{mod} = k_a e^{-\frac{u_{max}}{u_{ref}}} + k_b \tag{10}$$

where k_{a}, k_{b} and u_{ref} are constants, and u_{max} is the average of the maximum absolute deformation amplitudes in the negative ($u_{\mathrm{max,n}}$) and positive ($u_{\mathrm{max,p}}$) directions, as follows:

$$u_{max} = \frac{u_{max,p} + |u_{max,n}|}{2} \tag{11}$$

It is worth mentioning that the values of $u_{\text{max,p}}$ and $u_{\text{max,n}}$ are updated for every time step and are being fed into the constitutive equation to calculate the overall response. It was found that in this case, the ED's behaviour is depended on both the maximum displacement obtained during the loading history. Hence, the following parameters were selected to form the Modified Generalized Maxwell Model (MGMM):

- The linear GMM obtained from equation (9)
- The final version of F_0 can be calculated as:

$$F_0 = k_0 u \tag{12}$$

- The modified stiffness from equation (10)

Hence, the proposed hysteretic model can be determined as:

$$F + \frac{c_1}{k_1} F' = k_0 u + \frac{c_1}{k_1} k_0 u' + c_1 u'_1 + k_{mod} u \tag{13}$$

There are six total parameters that need to be determined: k_1, c_1, k_0, k_{a}, k_{b}, u_{ref}. In order to optimize these parameters the Levenberg-Marquardt algorithm was used to minimize the error between the experimental values and the force from the analytical model [19] based on sweep amplitude sinusoidal tests (the amplitude was ranging from 10%-50%), which were carried out for each frequency. These tests were determined, so the optimized parameters to be valid for each displacement and frequency, and were based on proposals from CASCADE[20]. Figure 8 shows a typical example of both the seep sinusoidal command displacement and the EDs response based on a frequency of 3.0 Hz. Table 2 provides the parameters of the hysteretic model, while figures 9, and 10 show that data obtained from the sweep sinusoidal tests and the hysteretic model are in a very good agreement. However, it should be noted that the proposed material model is validate only for sinusoidal loading, under a range of different amplitudes and frequencies. Since seismic loading can be decomposed into harmonic waves of different amplitudes and frequencies, the model is deemed to capture the essential characteristics of the damper under random loading[9].

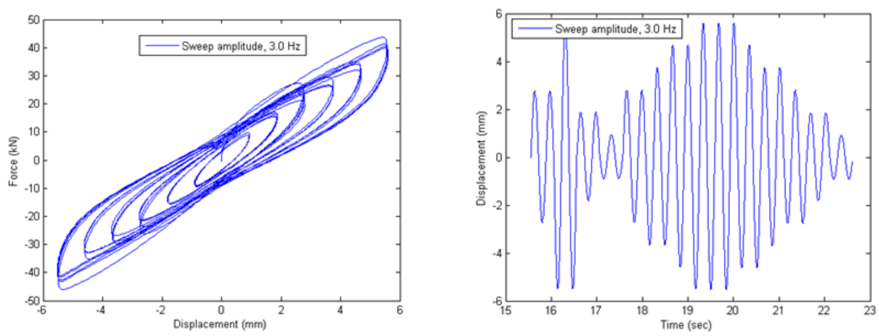

Figure 8: Sweep Amplitude Test for f=3.0Hz

Table 2: Optimized Model Parameters

k1	c1	k0	ka	uref	kb
0.514578	0.015148	0.000107	3.741913	7.966884	3.613982

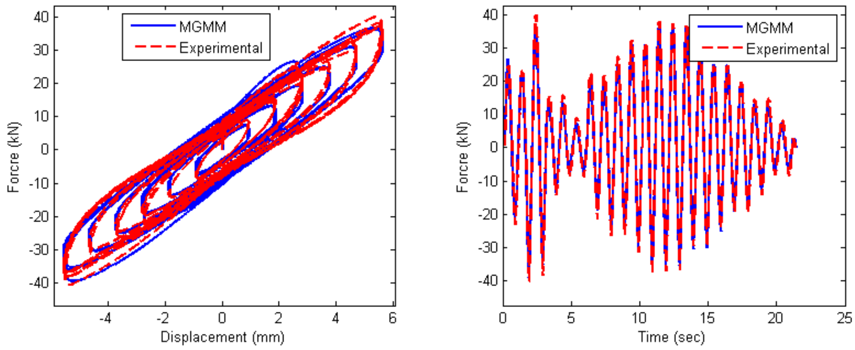

Figure 9: Comparison of force between experiment and MGMM model for sweep amplitude test for 1Hz

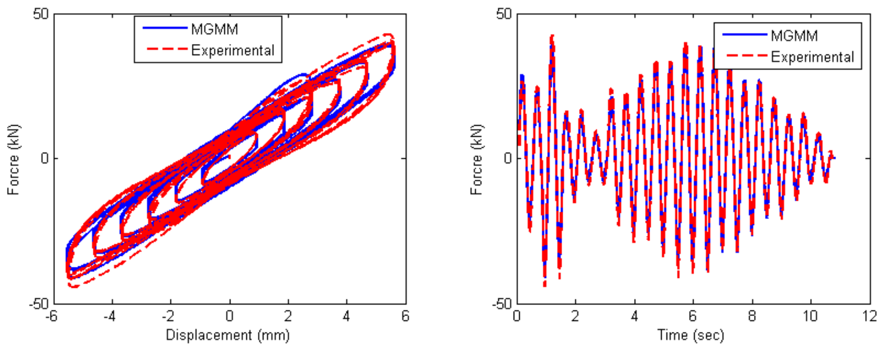

Figure 10: Comparison of force between experiment and MGMM model for sweep amplitude test for 2Hz

4 ANALYSIS

4.1 Prototype Building

A simple moment resisting frame was selected in this study, assuming seismic zone 3 (design ground acceleration=0.36g). The effect of EDs on the seismic performance of simple-conventional moment resisting frame (MRFs) was examined using the FE Software OpenSees for the 2D nonlinear analysis of the MRF. In the latter case, EDs and diagonal braces were also incorporated into the 2D MRF. The steel sections used here are IPE200 for the beam and HEM220 for the columns, while section SHS-150x12.5 was assigned to the bracings.

4.2 Scaling Ground Motions

According to EC8, scaled ground motions should be used for non linear analyses. Table 3 shows the ground motions used in this paper, while figure 12 presents the response spectrum of the ground motions, which were scaled in order to match the response spectrum of EC8 for a 10% probability of exceedance in 50 years. The scaling process was based on method of Fahjan[22] and Karabalis[23]. The scope of this method is the response spectrum of the

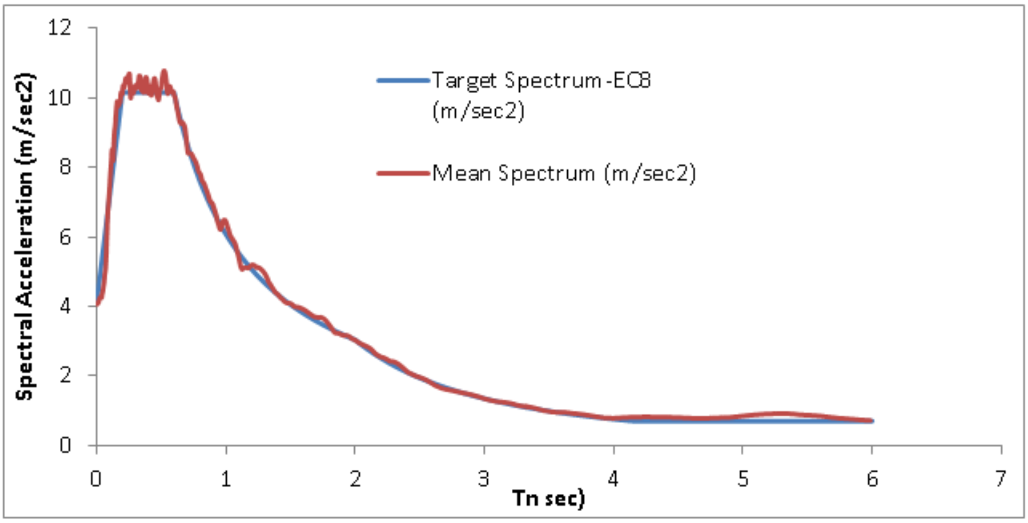

Figure 11: Mean Response Spectrum of the scaled ground motions

time history to gradually match the response spectrum of EC8 for a range of periods $0.2T_1$-$2T_1$ (where T_1 is the natural period of the building), by increasing some components of the spectrum index, and reducing some others. In essence, this method does not create new ground motions, but it modifies selected records in order to satisfy the criteria set by EC8.

Table 3: Ground motions used for non linear analysis

Earthquake	Station Name	Component	Magnitude	Distance (km)
Borrego 1942	El Centro Array #9	BORREGO_B-ELC000	6.5	56.88
Kern County 1952	LA - Hollywood Stor FF	KERN.PEL_PEL090	7.36	114.62
Imperial Valley-02 1940	El Centro Array #9	IMPVALL.I_I-ELC180	6.95	6.09
Northwest Calif-02 1941	Ferndale City Hall	NWCALIF.C_C-FRN045	6.6	91.15
Imperial Valley-0 1979	Niland Fire Station	IMPVALL.H_H-NIL090	6.53	35.64

4.3 Numerical Models

2D nonlinear models were developed in order to evaluate the effect of EDs on the MRF, with the FE software OpenSees. The beams and columns were modelled as distributed plasticity non-linear elements, with fiber sections. The stress strain relationship of the material is assigned to each fiber, and through integration at specific points along the element the active moments, and forces were obtained. The effect of the concrete slab was taken into account with diaphragms at each floor, but no stiffness contribution was considered for the seismic response. The base of the MRF was assumed to be fixed, the beam-column connections fully rigid, and the braces pinned, while no soil structure interaction was considered. A bilinear (elastic-perfectly plastic) model was used for the steel material, and fiber section was used in each element, except for the damper. The braces were assumed very stiff and were modelled as linear elastic elements. A Rayleigh damping matrix was used to model inherent damping of 2% at the first two modes of vibration. Each nonlinear analysis was extended beyond the actual earthquake time, so the correct residual displacements can be clear. Regarding the representation of the ED, concentric diagonal dampers were added to the initial MRF. A zero length element connected between the top of the braces and the middle of the beam was used to represent the damper. The constitutive

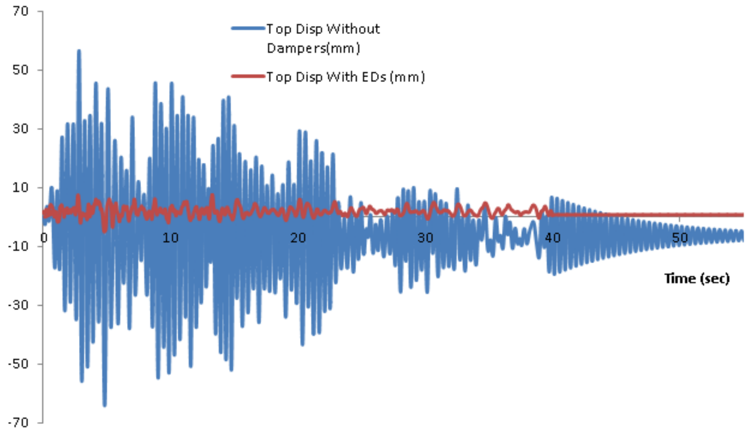

Figure 12: Displacement Time History Northwest Calif ground motion

model developed in equation 16 was incorporated into OpenSees, assuming is a zero length element. As it was previously mentioned the parameters assigned to the zero length element were based on the sweep amplitude tests. Regarding the design of the braces, a ratio, , of the braces stiffness to story stiffness equal to 7 was selected, so that the braces wont buckle under the forces transmitted by the dampers, and at the same time wont increase much the total weight of the structure. Karavsilis [9] carried out a parametric analysis in order to optimize the value of , and very good results were obtained using a range of values between 5 and 10, which justify my previous assumption. This way, braces are sized to be stiff enough so that the story drift produces elastomers deformation, rather than brace deformation[5].

4.4 Seismic Response

Both the MRF and DMRF (Damped Moment Resisting frame)were tested under the scaled ground motions for the Design Basis Earthquake (DBE), corresponding to a 10% probability of exceedance in 50 years and the results are illustrated in the following figures which evaluate the EDs efficiency during Northwest Calif ground motion. It can be clearly seen that the beneficial effect of the EDs used in this frame had a huge influence on the overall drift, while the base shear of the frame was reduced from to 235 kN to 32 kN (these values correspond to the mean values of the individual base shear for each ground motion). At the same time the residual displacement dropped almost to zero, and yielding was also avoided when EDs were implemented. The dynamic behaviour of the EDs is also presented here.

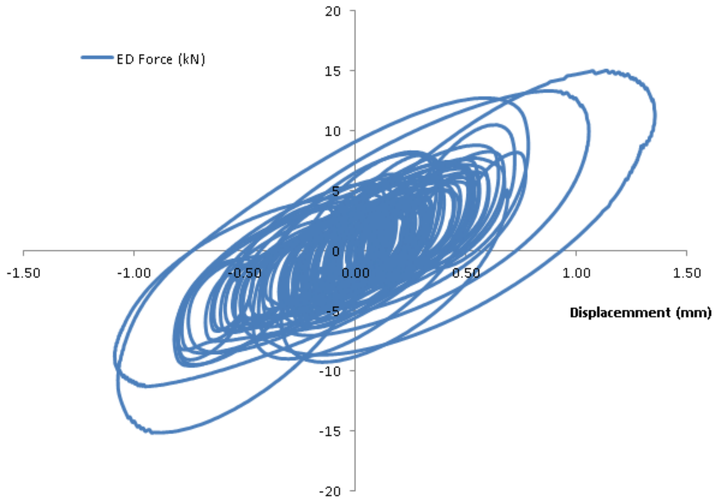

Figure 13: ED hysteretic loop during Northwest Calif ground motion

5 CONCLUSIONS

In this study the evaluation the constitutive model for describing the EDs dynamic behaviour and their effect on a simple steel moment resisting frame was examined . EDs exhibit both elastic and viscous mechanical behaviour, and can be simply modelled (especially for preliminary analyses) assuming the Kelvin-Voigt model, based on equivalent stiffness, keq, and equivalent damping ceq. However, a more sophisticated modes was developed here, and was incorporated into OpenSees in order to evaluate EDs' dynamic behaviour and their effect on steel frames. Nonlinear time history analyses were carried out for DBE level. Fahjan proposed method was used to scale ground motions to the DBE. In terms of displacements, and shear forces DMRF exhibited very satisfactory seismic behaviour, while plastic hinges didn't occur during any ground motion.

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