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ADVANCED NEGATIVE STIFFNESS VIBRATION ABSORBER COUPLED WITH SOIL-STRUCTURE INTERACTION FOR SEISMIC PROTECTION OF BUILDINGS

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Abstract. *Throughout the past decades, seismic isolation of structures has been studied rigorously as an approach to reduce seismic demand and mitigate structural damage. Research in this field has progressed significantly, starting from the use of simple elastomeric bearings for the decoupling of the superstructure from the base, to more complex devices that incorporate the use of an additional oscillating mass and negative stiffness elements. Characteristic examples of these devices are the Tuned Mass Damper (TMD) and the Quasi-Zero oscillators (QZSs). The KDamper is a novel passive vibration absorption concept, based essentially on the optimal combination of appropriate stiffness and mass elements, which include a negative stiffness element. The concept can be implemented using a system of prestressed elements in order to obtain the negative stiffness effects and might prove to achieve a significant advantage for seismic retrofitting.*

In this paper, the extended KDamper system is placed at the ground level and the soil-structure interaction (SSI) between the building foundations and superstructure is utilized as a means to effectively transfer the dynamic forces of the KDamper to the building and to distribute the required displacements between the structural elements and foundation, thus leading to absorption of the vibration energy. The configuration of the KDamper properties and tuning of its stiffness elements are studied herein. The effectiveness of the KDamper is subsequently investigated in view of the performance of a 4-storey building. A series of artificial accelerograms and real earthquake time histories are used as excitation. Results indicate the beneficial role of SSI on the reduction of the spectral acceleration and displacements, hence placing the concept as a compelling alternative to existing seismic protection technologies.

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1 INTRODUCTION

During the latest years, seismic codes have significantly been altered with the intention to lead to resilient structures that perform better under seismic loads. To this end, seismic isolation has been one of the main approaches aiming to reduce structural seismic accelerations and, in this way, lead to earthquake-resistant design [1-6]. Therefore, a number of isolation devices from simple elastomeric bearings to more sophisticated roller bearings have been developed and advanced within the years. However, in order to isolate the building from its base, large displacements are required that are not always acceptable, hence rendering this system inadequate for retrofitting.

Seismic protection research focuses lately on devices which include: (a) negative stiffness elements (Negative Stiffness (NS) Devices and "Quazi-Zero Stiffness" (QZS) oscillators) and (b) Tuned Mass Dampers (TMDs). The concept of introducing additional oscillating masses (TMDs) has been employed in a variety of systems including skyscrapers and structure bases aiming to protect the structure from the dynamic effects of environmental loads (e.g. wind, wave and seismic) [7-11]. The main drawback of this method is the need of large additional masses, compared to the mass of the structure itself, as well as the detuning of the TMD parameters that affects the performance of the structure within the years.

A promising class of absorbers is based on increasing the damping by the appropriate introduction of negative stiffness elements. Proposed by Antoniadis et al. [12], the KDamper is a novel passive vibration absorption concept, based essentially on the optimal combination of appropriate stiffness and mass elements, which include a negative stiffness element. This vibration control system has been examined for the protection of bridges [13-17] wind turbines [18, 19] and structural systems [20,21]. In addition, the KDamper is implemented as an Absorption Base (KDAB) in the bases of structures [22-28]. In the recent work of Kapasakalis et al. [28], the KDAB is proven to be a possible supplement to BI, or can be alternatively implemented as a "stiff absorption base".

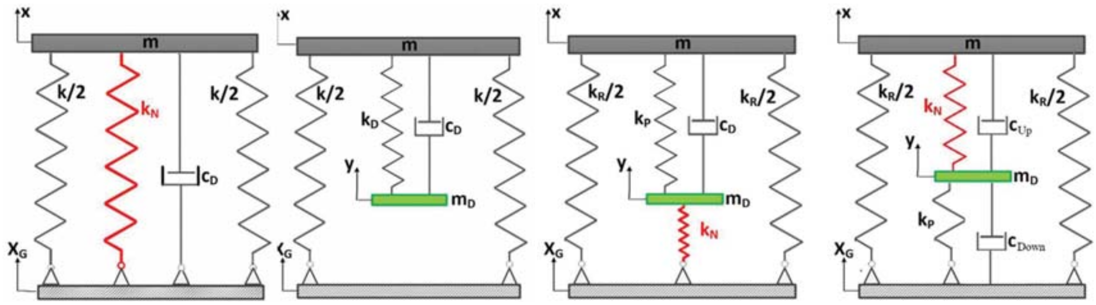
In this study, the EKD is implemented as a seismic protection measure within the base of a typical residential structure. The effects of the Soil-Structure interaction coupled with the KDamper damping properties are investigated and results indicate the significant benefits of the methodology at the seismic response of the structure.

2 OVERVIEW OF THE VIBRATION ABSORPTION METHODOLOGY

In this section, the extended KDamper concept (EKD) is presented and the fundamental principles of its function are provided in detail. A single degree of freedom (SDoF) oscillator is considered in which the KDamper is implemented between its rigid base and the oscillating mass. Throughout the paper, linear dynamic systems are considered to represent the stiffness elements of the KDamper while the effect of soil-structure interaction (SSI) is represented using non-linear springs.

2.1 The KDamper Vibration Absorber

Figure 1, illustrates the fundamental layout of various vibration isolation and damping concepts described in the literature and presented herein to facilitate the understanding of the KDamper concept. All strategies aim to minimize the response $x(t)$ of a low-damped (or undamped) SDoF comprising a mass m and static stiffness k due to the resulting motion of a base excitation $x_G(t)$. The Negative Stiffness Isolator (Fig. 1A) incorporates a negative stiffness element configured in parallel with the stiffness elements k of the system. To this end, the overall stiffness of the configuration is reduced to $k_{QZS} = k + k_N \leq k$, hence leading to potential significant decrease of the overall static stiffness and consequently, instability of the system.



(A) (B) (C) (D)

Figure 1. Schematic representation of the vibration absorption concepts considered: (A) Negative Stiffness (NS) isolator (or QZS), (B) Tuned Mass Damper (TMD), (C) KDamper concept and (D) Extended KDamper (EKD).

In a similar way to the Tuned Mass Damper (TMD, Fig. 1B), the KDamper (Fig. 1C) consists of an additional oscillating mass element (m_D) and two positive and negative stiffness elements k_P and k_N respectively. The negative stiffness element (k_N) serves as an indirect approach to increase the inertia of the oscillating mass (m_D) and consequently, increase the damping of the system. In addition, on the contrary to the NS isolator, the basic requirement of the KDamper is that the overall static stiffness of the system is maintained positive and, in this way, the risk of potential instability is eliminated. The total stiffness of the system with the KDamper can be expressed using the following formula:

$$k_R + \frac{k_P k_N}{k_P + k_N} = k_0 = (2\pi f_0)^2 m_{total} \tag{1}$$

Compared to the TMD, the KDamper uses an additional negative stiffness element k_N, which connects the additional mass to the base. Thus, the equation of motion of the KDamper becomes:

$$m\ddot{u}_S + \mathrm{k_R u_S} + \mathrm{m_D \ddot{u}_D} + k_N u_D = -(m + m_D)a_G \tag{2.a}$$
$$m_D\ddot{u}_D - C_D(\ddot{u}_S - \ddot{u}_D) - k_P(u_S - u_D) + k_N u_D = -m_D a_G \tag{2.b}$$

A closer examination at the equations above indicates that the additional negative stiffness element is an indirect way to increase the inertia of the KDamper mass m_D and to this end, increase the efficiency of the vibration absorption system without the need to incorporate large masses, contrary to the case of a conventional TMD.

2.2 Extended KDamper Concept (EKD)

The proposed vibration absorption concept is an extension of the KDamper referred to herein as the EKD system and is illustrated in Figure 1D. In a similar way to the KDamper, the EKD incorporates a system of masses, negative stiffness and positive stiffness elements as well as artificial dampers. The main variation would be the change of the system configuration where the positive stiffness spring (k_P) connects the damper mass (m_D) to the base of the system while the negative stiffness element (k_N), is attached between the damper mass (m_D) and the mass of the oscillator (m). Also, an additional artificial damper is adopted and placed in parallel with the negative stiffness element so that we end up having two dampers, namely C_{UP} and C_{DOWN}

Based on the original formulation of the KDamper expressions 2.a and 2.b, the following equations of motion for the EKD are derived:

$$m\ddot{u}_S + k_R u_S + m_D \ddot{u}_D + k_P u_D = -(m + m_D) a_G \tag{3.a}$$

$$m_D \ddot{u}_D - c_{UP}(\dot{u}_S - \dot{u}_D) - k_N (u_S - u_D) + c_{DOWN} \dot{u}_D + k_P u_D = -m_D a_G \tag{3.b}$$

As mentioned in the previous section, the stiffness parameters of the KDamper, and EKD respectively, are selected according to equation (1) in order to maintain the total stiffness of the system. However, an increase of the absolute value of k_N, or reduction of the absolute values of k_P or k_R, may endanger the static stability of the structure. In order to ensure that potential loss of the static stability is prevented, the possible variations of k_N, k_P and k_R should be taken into consideration in the design and optimization of the EKD parameters. The stiffness parameters of the system may present significant fluctuations due to numerous reasons, such as temperature variations, manufacturing tolerances, or non-linear behaviour of structural elements and the unpredictable behaviour of the foundation system (SSI). Consequently, an increase of the absolute value of k_N and/or a decrease of the values of k_P and k_R by a factor ε_N, ε_P and ε_R, respectively, may result in the system being unstable. This situation may happen in the following case:

$$(1-\varepsilon_R)k_R + \frac{(1-\varepsilon_P)k_P(1+\varepsilon_N)k_N}{(1-\varepsilon_P)k_P + (1+\varepsilon_N)k_N} = 0 \tag{4}$$

Contrary to the KDamper concept, which foresees variation only in the negative stiffness element k_N, in the proposed extension of the KDamper concept (EKD), a variation in all stiffness elements is taken into consideration. Assuming either the negative stiffness ratio kn=k_N/k_0 or the absolute value of the negative stiffness element (k_N) is known, using the equations (1) and (4), the rest of the stiffness elements result as presented below:

$$kr = \frac{-b - \sqrt{b^2 - 4*a*c}}{2a} \Rightarrow KR = kr * k_0 \tag{5.a}$$

$$kp = \frac{kn - kr*kn}{kr + kn - 1} \Rightarrow KP = kp * k_0 \tag{5.b}$$

Where parameters a, b and c are defined as:

$$(1-\varepsilon_R) = R\,;\ (1+\varepsilon_N) = N\,;\ (1-\varepsilon_P) = P \tag{6.a, b, c}$$

$$a = R(P-N)\,;\ b = kn*N(P-R) + R(N-P)\,;\ c = -P*N*\mathrm{kn} \tag{6.d, e, f}$$

Finally, assuming that the following parameters are known:

1) the additional mass, m_D, and
2) the variations ε_N, ε_P and ε_R of the stiffness elements

The unknown parameters of the system are:

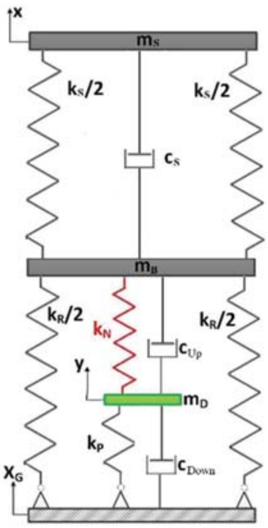
1) the nominal KDamper frequency f_0 (Eq. (1));
2) the negative stiffness ratio kn or the value of the negative stiffness element k_N and
3) the damping coefficients c_{DOWN} and c_{UP}.

3 EXTENSION OF THE EKD TO MULTI-STOREY STRUCTURES INCLUDING SOIL STRUCTURE INTERACTION (SSI) EFFECTS

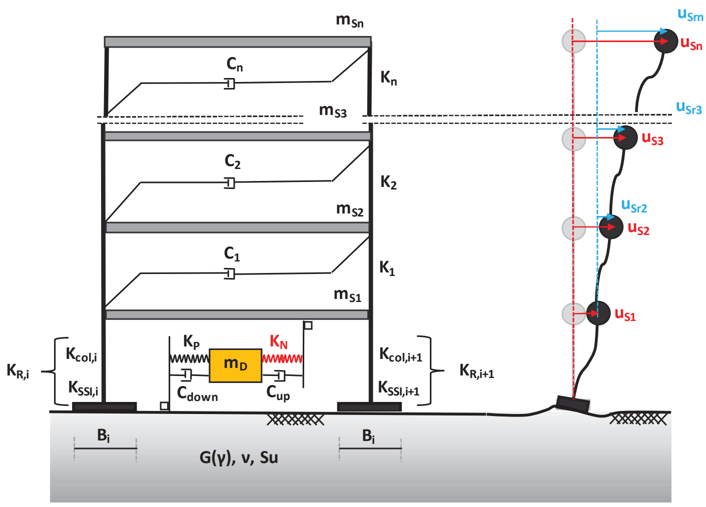
In this section, the concept of the EKD is extended to multi-degree of freedom systems (MDoF) and applied to multi-storey buildings with aim to control their response to seismic excitations. For this purpose, an extension of the vibration control strategy from a simplified SDoF system (Fig. 1D) to MDoF systems is presented. In addition, Soil-Structure Interaction effects are considered in the analysis of the structural system using non-linear springs derived from numerical analysis of shallow foundation systems.

Based on the methodology presented herein, a numerical application is performed on a 4-storey typical concrete structure in order to assess its behaviour under seismic loading and demonstrate the beneficial role of SSI along with the application of EKD as a vibration absorption method.

3.1 The EKD as a Seismic Base Absorber



(A)



(B)

Figure 2. (A) Implementation of the EKD as a seismic base absorber, (B) Multi-storey building with the proposed seismic vibration absorber system, EKD (sketch of the model) together with SSI effects.

The possible implementations of the KDamper and specifically of the Extended KDamper (EKD) might be numerous. In this study, the potential application of the EKD for seismic protection of structures is assessed and the EKD is implemented as a vibration base absorber. To this end, the EKD system is located between the base of the building and the superstructure. The corresponding configuration is illustrated in Fig. 2A and the equations of motion are derived as follows:

$$m_S(\ddot{u}_S+\ddot{u}_B)+c_S\dot{u}_S+k_Su_S=-m_Sa_G \tag{7.a}$$

$$m_B\ddot{u}_B+m_S(\ddot{u}_S+\ddot{u}_B)+c_{UP}(\dot{u}_B-\dot{u}_D)+k_N(u_B-u_D)+k_Ru_B=-(m_S+m_B)a_G \tag{7.b}$$

$$m_D\ddot{u}_D-c_{UP}(\dot{u}_B-\dot{u}_D)-k_N(u_B-u_D)+k_Pu_D+c_{DOWN}\dot{u}_D=-m_Da_G \tag{7.c}$$

where $u_S = x_S - x_B$, $u_B = x_B - x_G$, $u_D = x_D - x_G$. The natural frequencies of the subsystems and the rest of the parameters are defined as:

$$\omega_0 = 2\pi f_0 = \sqrt{k_0 / (m_S + m_B + m_D)}\ ;\ \omega_D = \sqrt{(k_P + k_N) / \mathrm{m}_D} \tag{8.a, b}$$

$$k_0 = k_R + \frac{k_P k_N}{k_P + k_N}\ ;\ m_D = \mu_D (m_S + m_B) \tag{8.c, d}$$

where ω_0 and ω_D refer to the natural frequencies of the total system and the EKD respectively, k_0 is the equivalent static stiffness of the EKD, m_B is the mass of the base, m_S the mass of the superstructure, m_D the mass of the EKD and μ_D the damper's mass ratio. In this study, the EKD parameters are selected according to Section 3.2 and Section 5 and refer to a typical 4-storey structure with total mass of *320 tn*. More details of the case study are presented in the following sections.

3.2 Extension of the EKD to Multi-Storey Structures

A planar n-storey building is considered and presented in the sketch of Fig. 2B, in which the proposed base absorber system, EKD is implemented for seismic protection between the base and the first floor of the structure. The following assumptions are made for the modelling of this system:

1. the total structure mass is concentrated at floor levels;
2. the slabs and girders on the floors are rigid when compared to the columns;
3. the columns are considered inextensible and weightless, providing the lateral stiffness of the structure;
4. the effect of soil-structure-interaction (SSI) is taken into consideration with the use of non-linear elastic springs coupled in series with the column stiffnesses of the first floor;
5. the superstructure is considered to remain within the elastic limit during the analysis.

As a result, the superstructure has n dynamic DoFs, represented by the relative to the ground displacements of the n-story masses m_{Sj} ($j=1,\dots, n$), as presented in Fig. 2B, which are collected in the array $[u_{Sr}](t)=[u_{Sr1}(t), u_{Sr2}(t), \dots, u_{Srn}(t)]^T$.

The parameter k_R refers to the coupling of the stiffness of the columns (k_{col}) connecting the foundation of the structure with the first floor together with the stiffness of the foundation system (k_{SSI}) that is derived due to the SSI effects. As a result, the total stiffness of the columns-foundations system is calculated as the k_R of the structure (Eq. 9). More details on the derivation of the k_R and incorporation of the SSI effects are provided in section 3.3 of this paper.

$$k_R = \frac{k_{SSI} k_{col}}{k_{SSI} + k_{col}} \tag{9}$$

The equations of motion Eq. (7) still apply, but now expressed in a matrix form, involving matrices with dimensions $r\times r$ with $r=n+1$:

$$[M][\ddot{u}(t)] + [C][\dot{u}(t)] + [K][u(t)] = -[\tau]a_G(t) \tag{10}$$

where the matrices and vectors entering Eq. (10) are defined as:

$$[K]_{(n+1)\times(n+1)} = \begin{bmatrix} [K_S]_{n\times n} & [0]_{n\times 1} \\ [0]_{1\times n} & [0]_{1\times 1} \end{bmatrix} + \begin{bmatrix} [0]_{(n-1)\times(n-1)} & [0]_{(n-1)\times 1} & [0]_{(n-1)\times 1} \\ [0]_{1\times(n-1)} & k_N + k_R - k_{col} & -k_N \\ [0]_{1\times(n-1)} & -k_N & k_P + k_N \end{bmatrix};$$

$$[C]_{(n+1)\times(n+1)} = \begin{bmatrix} [C_S]_{n\times n} & [0]_{n\times 1} \\ [0]_{1\times n} & [0]_{1\times 1} \end{bmatrix} + \begin{bmatrix} [0]_{(n-1)\times(n-1)} & [0]_{(n-1)\times 1} & [0]_{(n-1)\times 1} \\ [0]_{1\times(n-1)} & c_{UP} & -c_{UP} \\ [0]_{1\times(n-1)} & -c_{UP} & c_{UP} + c_{DOWN} \end{bmatrix}; \qquad (11)$$

$$[u(t)]_{(n+1)\times 1} = \begin{bmatrix} [u_{st}]_{n\times 1}(t) \\ u_D(t) \end{bmatrix};$$

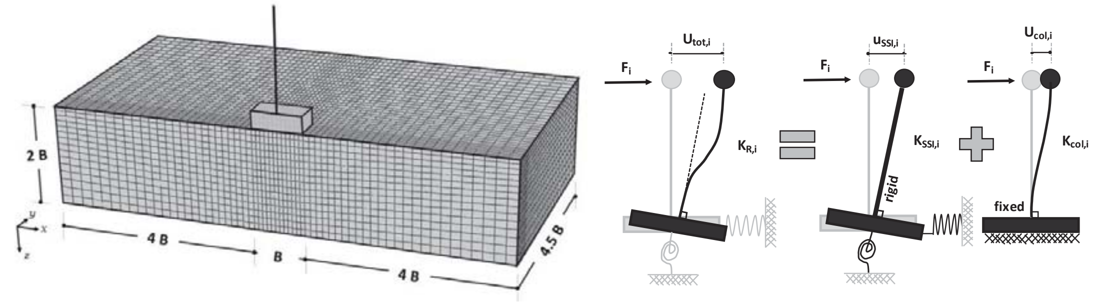
$$[M]_{(n+1)\times(n+1)} = \begin{bmatrix} [M_S]_{n\times n} & [0]_{n\times 1} \\ [0]_{1\times n} & m_D \end{bmatrix}; \; [\tau]_{(n+1)\times 1} = \begin{bmatrix} [M_S][\tau_S] \\ m_D \end{bmatrix}; \; m_{S,tot} = \sum_{i=1}^{n} m_i$$

where $[M_S]$, $[C_S]$ and $[K_S]$ are the n-dimensional matrices of mass, damping and stiffness of the original n-story building and $[\tau_S]$ is the $n\times 1$ influence vector of the superstructure associated with the ground motion $x_G(t)$. The parameter m_S now refers to the total multi-story structure mass and is $m_{S,tot}$ (Eq. (11)). This procedure can also be used in the case the n-story structure is mounted on a conventional or highly damped isolation base by appropriate modification of the matrices in Eq. (10).

3.3 Introduction of Soil-Structure Interaction (SSI) Effects

For the purposes of this study, the EKD is implemented as a vibration absorption technology positioned between the foundations and the first floor of the structure (Ground / 'Pilotis' level). As a consequence, the stiffness k_R (Fig. 2) is the resultant stiffness of both the columns and foundation system and can be expressed as in Eq. (9). The beneficial effect of the Soil-Structure Interaction (SSI) is taken into consideration with the use of non-linear spring stiffnesses (k_{SSI}) coupled in series with the stiffness of the ground floor columns (k_{col}). As a result, the benefits of a softer foundation response can be compared with the conventional approach of a fully fixed model.

The problem is analysed by employing the FE code ABAQUS and a typical foundation and column are modelled in 3D as depicted in Fig. 3A. The soil stratum is considered to be isotropic/homogeneous and modelled using non-linear 8-noded hexahedral continuum elements. The same is applied for the footing which is, however, considered to be elastic (Reinforced Concrete). The column is modelled using linear beam elements and is considered to be fixed on the top of the foundation. The interface between the foundation and the soil is considered to be 'fully bonded' with the foundation elements considered attached to the ground. The boundaries of the model have been strategically selected sufficiently far to avoid spurious boundary-effects on the system response. Static pushover analyses are subsequently undertaken in order to derive the non-linear soil-foundation stiffness relationships (k_{SSI}).



(A) (B)

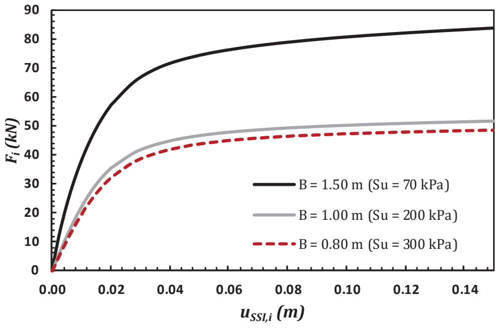
Figure 3. (A) A view of the ABAQUS 3D FE model for a rigid footing with B=1.5m, (B) A simplified approach to account for the SSI effects (horizontal and rotational stiffness of the soil) for horizontal displacement at the top of each column.

Non-linear soil behaviour and stress-strain relationship is described by a simplified kinematic hardening model that follows a Von Mises failure criterion with associated flow rule. The formulation of this modified Von Mises model is adopted as a subroutine in ABAQUS code and has been parametrised by Gerolymos and Gazetas [29] and Anastasopoulos et al. [30] in order to appropriately simulate the response of clayey materials. Despite its simplicity, this constitutive model has been validated against physical model testing hence ensuring its ability to capture realistically the foundation behaviour and the non-linearity of the soil stiffness.

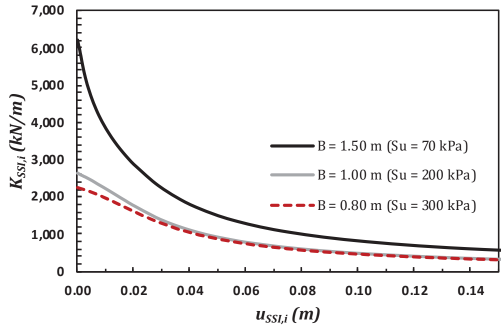
The foundation system is designed based on the properties and approximate loading of a typical residential 4-storey building as described in Chapter 5 of this paper. To this end, a series of parametric analyses have been conducted by adopting different foundation widths (B) and various material properties (S_u=70, 200 and 300 kPa) resulting in different FoS (Factor of Safety) against vertical loading. On the basis of empirical correlations, the initial, small strain, elastic modulus (E_0) of the soil was considered equal to $1800S_u$ and its non-linear behaviour is simulated realistically by the constitutive relations of the model.

As indicated in Fig. 3B, the equivalent soil-foundation stiffness (k_{SSI}) is calculated assuming a rigid column subjected to horizontal displacement at the top. The height of the column is equal to 3.5m, aligned with the height of the column considered in our case study (Section 5). As a consequence, the calculated stiffness (k_{SSI}), is a result of both the rotational and horizontal displacement of the soil footing and is expressed at the top of the rigid column.

The total stiffness (k_R) of the system can be then calculated as the resultant of the structural column stiffness (k_{col}) and the equivalent stiffness at the top of the beam due to the rotational and horizontal displacement of the footing (k_{SSI}). Results for a few foundation widths (B) and soil profiles that have been considered relevant for the study are presented in Fig. 4 below. The horizontal displacement at the top of the column is plotted against the reaction force (RF) and soil-foundation stiffness (k_{SSI}) respectively. All foundations considered below have a total FoS against vertical loading between 3.5 and 4 thus ensuring the stability of the structure under static loading. For the purposes of the study, the footing geometry selected represents a relatively soft foundation that allows deformation of the soil-foundation system during earthquake excitation as a means to reduce the forces transmitted to the structural members. This design is based on the idea of "rocking" foundations which has been investigated using numerical and physical models by several researchers [31-34].



(A)



(B)

Figure 4. (A) Reaction force (F) versus horizontal displacement at the top of the rigid column, (B) Soil-Foundation Stiffness (k_{SSI}) horizontal displacement at the top of the rigid column.

4 OPTIMIZATION OF THE EKD PARAMETERS

After the equations of motion of the EKD and structural system are derived, the aim is to determine efficient parameters for the proposed configuration in order to achieve an optimized vibration control strategy. As stated in the previous sections, the parameters f_0, k_N, c_{DOWN} and c_{UP} are the free design variables taken into consideration in the optimization problem, while parameters such as the additional damper mass m_D and the stability factors ε_N, $\varepsilon_{P,}$ and ε_R are considered known. For the optimization process, a novel metaheuristic algorithm, harmony search algorithm (HS), is adopted. In this study, the optimization problem is formed according to the seismic design codes. The constraints and objective function are selected from the time-domain responses. The optimisation procedure is described in greater detail in the following sections.

4.1 Selection of Seismic Excitations Compatible to the Seismic Design Codes for the Analysis and Optimization of the EKD

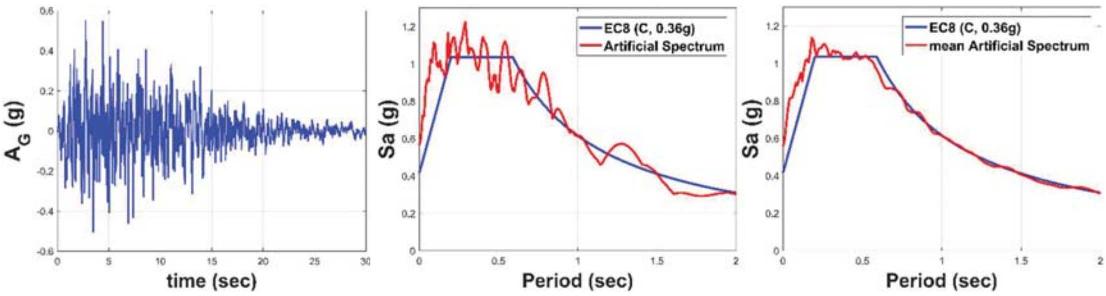
The calculation of structural displacements, velocity and acceleration due to seismic excitation is usually undertaken according to seismic design codes (e.g. EC8) and the design response spectrum. As a consequence, the seismic intensity measures are within specified limits for a system with given properties (fundamental structural period and damping ratio). The parameters that mainly control these limits are the ground conditions, the expected seismic intensity, the damping ratio of the system as well as the fundamental period of the structure.

However, the application of such approach and the use of the code's Design Response Spectra for the selection of the extended KDamper parameters, is not considered adequate, since the application of the KDamper leads to MDoF systems with multiple eigenfrequencies. Therefore, analysis in the time-domain is required for the optimal design of the proposed base absorber. Strong earthquake records can be generated from various types of accelerograms such as synthetic records obtained from seismological models, real earthquake excitations and artificial accelerograms, compatible with a specific design response spectrum.

4.2 Generation of Artificial Accelerograms

In this section, a presentation of the details of the generation of artificial accelerograms based on the design response spectra is undertaken. This is a task that has been widely studied by researchers, overviews of which can be found among others in Giaralis and Spanos [35] and Cacciola and D'Amico [36]. For the purposes of this study, the approach followed is based on generating a sample of artificial accelerograms with acceleration response spectra in accordance with the EC8 design response spectra. Artificial, spectrum-compatible accelerograms are generated using the SeismoArtif Software [37]. In this paper, the Artificial Accelerograms are designed to match the EC8 response spectra incorporating the following seismic properties: ground type C, spectral acceleration 0.36 g, spectrum type I and importance class II.

An artificial accelerogram is illustrated in Fig. 5A, along with its acceleration response spectrum (S_a) (Fig. 4B). A sample of 15 artificial accelerograms is generated and plotted in Fig. 5C along with the EC8 response spectrum for comparison with the design guidelines. An accurate match is observed with the EC8 acceleration response spectrum with characteristics: spectral acceleration 0.36 g, ground type C, spectrum type I and importance class II. More specifically, the percentage deviation is below *10%* in the range of periods from *0.2* to *2* sec, which are considered relevant for the structural performance.



(A) (B) (C)

Figure 5. (A) Artificial Accelerogram, (B) Acceleration response spectrum of Artificial Accelerogram and (C) Mean Artificial acceleration response spectrum of the 15 generated Artificial Accelerograms compared to the EC8 acceleration response spectrum.

4.3 Evaluation of Optimization Constraints and Limitations

The design of the proposed control strategy must be efficient and realistic at the same time. For this reason, proper constraints that refer to the structural dynamic responses and therefore the EKD components must be applied. The structure's first floor drift is set as the objective function (mean of 15 maximum responses). The system is subjected to 15 Artificial Accelerograms generated according to section 4.1, with a mean PGA of *0.519g*.

Furthermore, the given parameters of the system with the EKD, i.e. the additional mass m_D and the stability factors ε_N, ε_P, and ε_R, as well as the limits of the free design variables sought in the optimization problem, f_0, k_N, c_{DOWN} and c_{UP}, must be within reasonable technological capabilities in order for design to be realistic. In particular:

i. In the previous work of KDamper, the stability factor ε_N that relates to the variation of the negative stiffness element k_N, is taken as *5%*. In this study, a variation in all stiffness elements is considered, making the design of the proposed vibration absorption concept rather conservative. The stability factors ε_P and ε_N are selected equal to *10%*, and the stability factor that relates to the stiffness element k_R is selected equal to 25%. The potential 25% stiffness reduction was selected based on an initial linear analysis with the EKD implemented in the structure assuming rigid footing. In this way, the anticipated displacement of the column top was calculated and the expected reduction of the k_{SSI} stiffness due to this deformation was estimated (approximately 25%, refer to Fig. 4B).

ii. The purpose of this design is to implement the proposed control strategy for seismic protection of building structures. Since KDamper can be implemented between the floors of structures, there are no strict limitations regarding the additional mass. According to the previous work of KDamper implemented as an Absorption Base (KDAB) of multi-story building structures with the same characteristics, [28], a *5%* additional mass is efficient and realistic.

iii. The negative stiffness element k_N, in the proposed configuration of EKD, is realized with pre-compressed springs. Based on previous indicative designs of KDamper with pre-compressed springs by Antoniadis et al. [12] and Kapasakalis et al. [27], for a total superstructure mass of *300 tn*, a realistic value of k_N is *-34171 kN/m*. In this work, the negative stiffness element absolute upper value is *|30000| kN/m*.

iv. The damping coefficients maximum value is set equal to *600 kNs/m* for the whole configuration (multiple KDamper devices can be implemented), therefore, common linear damping devices can be used.

v. The negative stiffness element stroke (X_{S-D}) is set as a constraint, with an upper limit value of *15 cm* which, based on previous work of Kapasakalis et al. [25], proves to be a realistic value for the design of the negative stiffness element with pre-compressed springs.

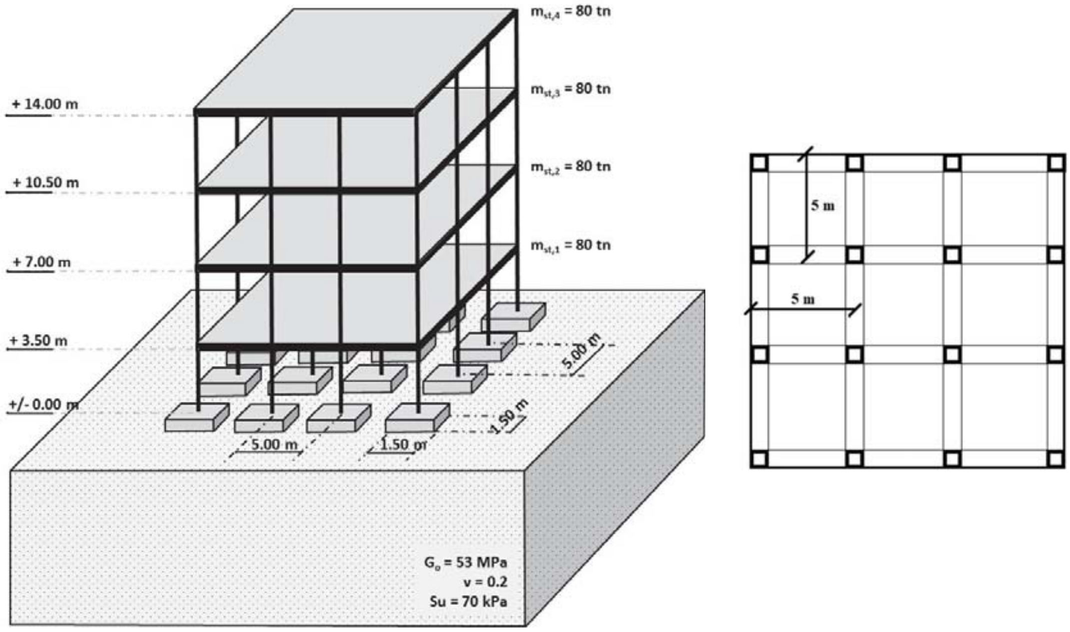
Finally, the limits of the free design variables are: 1) the nominal KDamper frequency f_0 (*Hz*) [*0.15 1.5*], 2) the negative stiffness element k_N (*kN/m*) [*-30000 -1*] and 3, 4) the damping coefficients c_{DOWN} and c_{UP} (*kNs/m*) [*1 600*].

5 NUMERICAL CASE STUDY ON A TYPICAL 4-STOREY STRUCTURE

A numerical case study has been conducted considering a 4-storey typical residential building. The structural system has 4 dynamic DoFs represented by the relative displacements of the 4 storeys with respect to the ground. The elastic modulus of reinforced concrete (assuming long-term cracked conditions) is equal to E=26 GPa. The mass of the building is considered to be concentrated at the floor levels, with m_i=80000 kg denoting the mass of the 4 storeys (i=1,…,4), while the columns are assumed to be weightless.
The superstructure's damping coefficients are mass and stiffness proportional (Rayleigh damping), with ζ_{Si}=0.02 (i=1,…,4).

The geometry of the model is presented in Fig. 6 below. A 15x15m floor in plan-view has been considered with 0.3x0.3m square columns. Analysis for different foundation widths (B) and soil material properties has been undertaken and results for a single case are presented herein. Therefore, the material under investigation has an undrained shear strength equal to S_u=70 kPa and a G_0=53 MPa. The footing width (B) is considered as 1.50m and a total FoS against vertical loading equal to 3.8 is calculated, hence ensuring the static stability of the system. For more details regarding the selection of the material properties refer to Section 3.3.

The soil-structure interaction is regarded in the excitation analysis as explicitly detailed in section 3 of this paper and the parameters of the EKD are optimized based on the methodology described in section 4. A Matlab-based code is incorporated to model the structure and extended KDamper system as a series of masses and springs; linear and non-linear representing the structural elements and soil-foundations respectively. A number of artificial and real seismic excitations are subsequently applied and the response of the structure is investigated. Results of the analysis are presented in Fig. 6 and Table 1 below.



(A) (B)

Figure 6 (A) Typical 4-storey residential building investigated as a case study, (B) Plan view of a typical floor of the building considered in this case study.

Structural System	Ensemble of 15 artificial accelerograms, PGA = 0.24 g					L'Aquila (2009), PGA = 0.40 g				
	$a_{s,4}$ (*g*)	$u_{s,1}$ (*cm*)	$u_{col,1}$ (*cm*)	$u_{SSI,1}$ (*cm*)	V_b (*kN*)	$a_{s,4}$ (*g*)	$u_{s,1}$ (*cm*)	$u_{col,1}$ (*cm*)	$u_{SSI,1}$ (*cm*)	V_b (*kN*)
Building with fixed footings	1.18	3.91	3.91	-	2,540	1.13	2.35	2.35	-	1,527
Building with SSI footings	1.08	6.67	2.02	4.65	1,310	0.89	5.72	1.85	3.88	1,204
Building with KDamper & fixed footings	0.35	3.32	3.32	-	640	0.29	3.60	3.60	-	745
Building with KDamper & SSI footings	0.29	3.29	1.36	1.93	518	0.29	4.50	1.61	2.90	400

Table 1: Dynamic response of the four examined residential building systems (with fixed footings, with SSI footings, with KDamper and fixed footings, with KDamper and SSI footings) for an ensemble of 15 artificial accelerograms (mean of 15 max values), and a real near fault earthquake record (L'Aquila, 2009, Mw = 6.3).

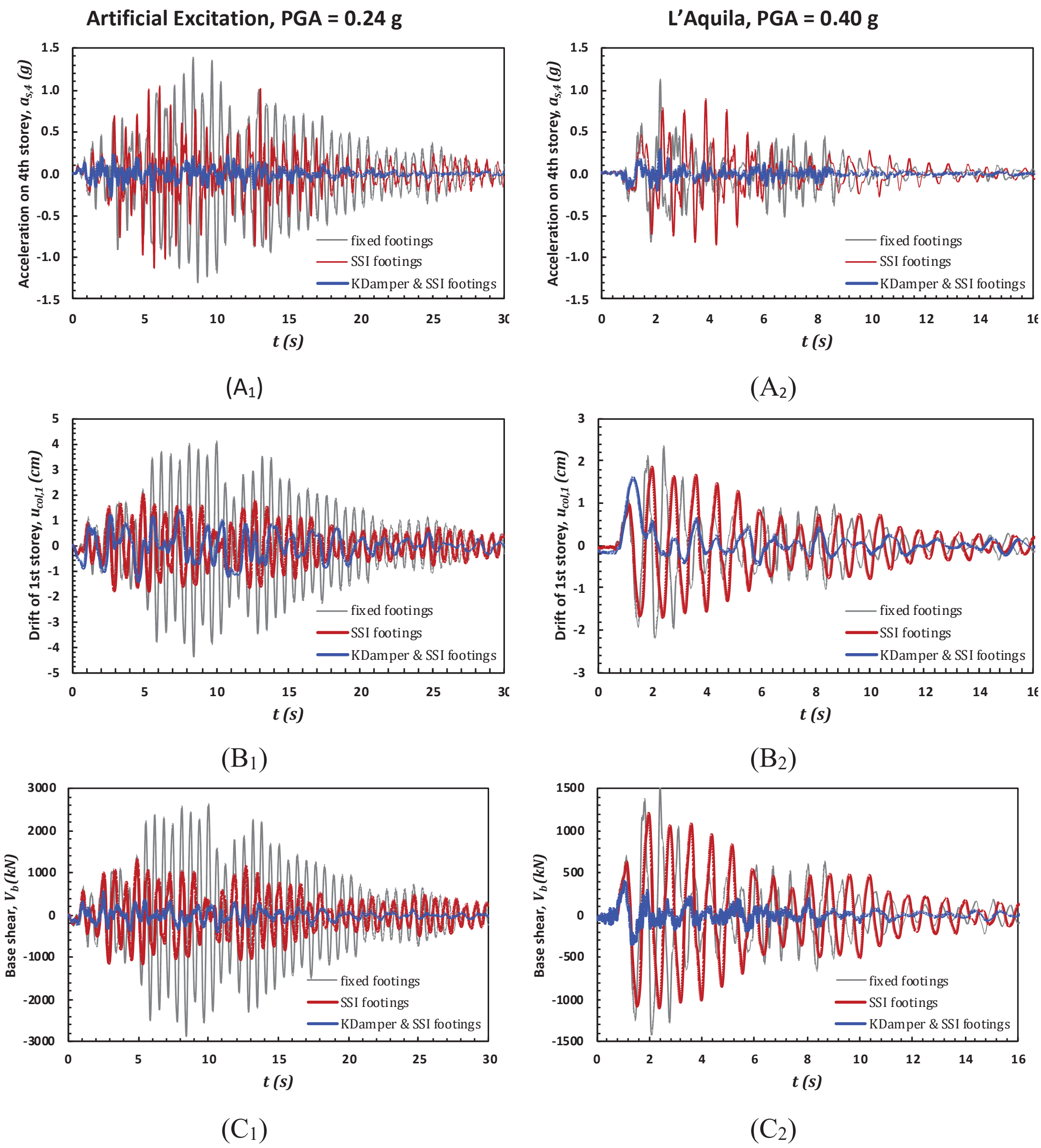
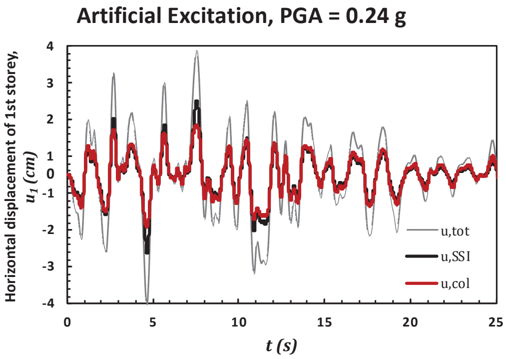


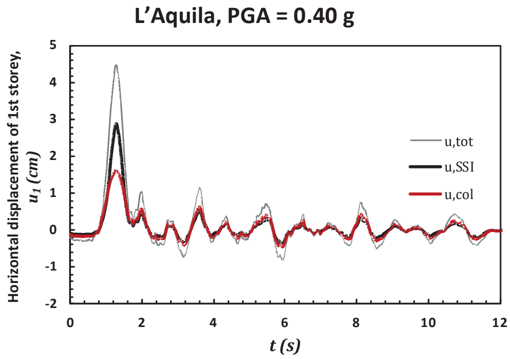
Figure 6. Comparative results in terms of (A) Absolute acceleration of the top storey of the structure (B) Horizontal displacement (drift) of the 1st floor and (C) Base shear-force of the structure for one artificial accelerogram (A_1, B_1, C_1) and the L'Aquila earthquake record (A_2, B_2, C_2) respectively.

A comparison between the response of the structural model considering a) fixed foundations, b) SSI effects by incorporating the non-linear stiffness of the soil and c) the extended KDamper coupled with SSI effects is undertaken for one of the artificial accelerograms (matching the EC8 design spectrum) and a real earthquake excitation. It can be observed that, the application of the extended KDamper along with the SSI effects lead to the main following results:

1. The EKD together with the SSI effects appear to improve the structural response under a dynamic seismic excitation, indicating that the non-linearity of the overall system stiffness parameters does not lead to detuning of the damper;
2. A 40-70% reduction of the absolute acceleration values of the building storeys is achieved, hence remaining within the structural limits;
3. The base shear values appear to be significantly reduced in accordance with the absolute acceleration values;
4. The total horizontal displacement of the first floor is slightly reduced compared to the fixed footings. However, as indicated in Fig. 7 below, due to the rotational and horizontal movement of the foundation the total displacement is distributed between the column and the footing (SSI) and consequently structural flexural displacements are reduced and remain within an acceptable region.



(A)



(B)

Figure 7. Distribution of the first storey total horizontal displacement due to the flexural deformation of the column (drift) and deformation due to SSI effects (horizontal and rotational movement of the footing), for the case of an artificial accelerogram (A), and the L'Aquila record (B)

As it can be observed in Fig. 7, although the displacement of the first storey for the case of an artificial accelerogram (u_{tot}) reached 3.36 cm, it was distributed both in the column, in the form of flexural deformation (u_{col}), and in the footing, in the form of predominantly rotational, and also translational displacement, due to the SSI effects (u_{SSI}). The maximum flexural deformation was a mere 1.37 cm, which is almost three times lower than the flexural deformation that was demanded in the case of the building with no seismic protection. The 1st-storey displacement is primarily undertaken by the SSI mechanism, meaning that during the shaking, the equivalent soil-foundation stiffness of a single footing 1.50 m x 1.50 m has dropped below the structural column stiffness of a single column, equal to 4,000 kN/m. In general, the distribution of the stiffness in the column-footing-soil system governs the distribution of the deformation in these components. It should be highlighted that for the case of either wider surface footings, or for footings whose rotational and horizontal displacement have been kinematically constrained, as in the case of footings connected with strap beams, the equivalent soil-foundation stiffness of the system increases dramatically, and hence the displacement is not distributed among the column and the foundation. Thus, the total displacement is undertaken by the columns leading to additional loading of the structural elements.

6 CONCLUSIONS

In this paper, the extended KDamper (EKD) is implemented as a seismic protection vibration control system located at the base of a typical building structure. The beneficial effects of soil-structure interaction (SSI) of a relatively soft foundation system along with the EKD as excitation absorber are investigated. An analytical approach for the selection of the EKD parameters aiming to optimize acceleration response, under base excitation, is considered and the initial structural system is introduced along with the vibration absorber concepts. A sample of Artificial Accelerograms with acceleration response spectra compatible to EC8 is generated, and a few real earthquake shaking records are considered. The concept is applied between the foundation system and the first floor of a 4-storey building that is examined as a case study herein. Results show significant reduction of the structural accelerations, drifts and base shear forces, hence placing the concept of a coupled EKD-SSI system as a compelling methodology for seismic protection of structures.

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