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Design criteria for structural monitoring system: a preliminary approach

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ABSTRACT

Sensor technological enhancement and Information and Communication Technologies (ICT) meet together nowadays in the concept of Smart Cities, i.e. cities that offer an additional value to their inhabitants and visitors, such as monitored safety of infrastructures. Structural Health Monitoring (SHM) appeared as a tool for structural diagnosis of buildings or civil engineering facilities, for both static and dynamic behaviours. Early applications required difficult and costly experimental setups. However, new sensor typologies, such fibre optics or MEMS, and wireless communications offer today a whole new scenario of possibilities, such as the permanent seismic monitoring of historical heritage. Nonetheless, the design of a SHM system often comprises a numerical model representative of the monitored structure, and useful for a better selection of the characteristics and location of each sensor. This approach is usually slow and costly, hence simplified criteria are proposed here to make a rapid preliminary design of a dynamic SHM system based on the evaluation of dynamic parameters (e.g. modal participation factors or natural frequencies). The design methodology is illustrated through its use in the development of a distributed network of monitored structures at L'Aquila.

Keywords: *structural monitoring, design criteria, structural dynamics, cultural heritage, smart city.*

1 INTRODUCTION

Structural Health Monitoring (SHM) appeared as a tool for structural diagnosis of buildings or civil engineering facilities, for both static and dynamic behaviours. A monitoring system, permanent or temporary, together with a dynamic analysis, may be useful to determine the damage level suffered by architectural heritage after a seismic event [1], or evaluate the influence of retrofitting works on a civil engineering structure [2]. This type of performance is usually costly and comprises complex experimental setups. However the development of microelectronics, such as MEMS sensors, has opened a new variety of possibilities to design an SHM system, depending on the sensitivity requirements of each particular case.

Recently, the concept of Smart Cities (SC) has been proposed as a new urban development strategy [3], in which through the use of Information and Communication Technologies (ICT) the city itself can adapt to actively improve the life quality of its citizens and visitors. Among the possible services a SC can offer different systems such as adaptive traffic management systems able to guarantee the correct operation of emergency services [4]; a framework for traffic congestion reduction via an adaptive road routing service based on traffic lights control [5]; or the integration of various sensor networks distributed along a city, in order to promote a coordinated management of city resources [6].

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The INCIPIT project, currently under development in the city of L'Aquila (Italy), is trying to create a network of monitored facilities along a fibre optic ring under construction after the 2009 earthquake, which severely damaged most of the buildings in the city centre. In this case, L'Aquila SC is based on a fibre optic network, which will be useful, among other applications, to control and diagnose the structural condition of some key public buildings distributed along the city. In a future emergency situation this network could serve to develop the intervention strategy based on the knowledge of the status of each monitored structure. One of the objectives of this project is to establish some guidelines for a SHM systems design, in order to shorten the implementation time of these systems, which usually require a previous numerical model to accurately select and locate the sensor network.

2 ESTIMATION OF THE STRUCTURAL DYNAMIC PROPERTIES

The structural dynamic behaviour can be usually analysed using discretized models in order to reduce the dynamic system to a set of ordinary differential equations (ODEs). This system of ODEs can be written as Eq. (1), if linear elastic behaviour and linearized kinematics hypothesis are assumed.

$$\mathbf{M}\left(\ddot{\mathbf{u}}_g(t)+\ddot{\mathbf{u}}(t)\right)+\mathbf{C}\dot{\mathbf{u}}(t)+\mathbf{K}\mathbf{u}(t)=\mathbf{f}(t). \tag{1}$$

Where $\mathbf{u}$, $\dot{\mathbf{u}}$ and $\ddot{\mathbf{u}}$ are the nodal displacements, velocities and relative accelerations vectors; the properties of the system are defined by **M**, **C** and **K**, which correspond to the mass, damping and stiffness matrices respectively; $\ddot{\mathbf{u}}_g(t)$ refers to the accelerations imposed by ground motion and $\mathbf{f}(t)$ represents the external forces vector. After a modal analysis, the solution of Eq. (1) can be rewritten in an uncoupled system with *n* independent ODEs, Eq. (2), as a function of modal displacements z_i:

$$\ddot{z}_i(t)+2\omega_i\xi_i\dot{z}_i(t)+\omega_i^2 z_i(t)=\Gamma_i^l f(t)-\Gamma_i^g a_g(t). \tag{2}$$

$$\omega_i^2=\frac{K_i}{M_i} \text{ and } \xi_i=\frac{C_i}{2\sqrt{K_i M_i}}. \tag{3}$$

In this case, the undamped modal frequency ω_i^2 and damping ratio ξ_i can be estimated as a function of the modal parameters, i.e. mass (M_i), stiffness (K_i) and damping (C_i), Eq. (3). The contribution of each mode to the structural response can be evaluated using the modal participation factors, Γ_i^l and Γ_i^g, for a generic dynamic loading and unidirectional ground motion respectively. These factors can be calculated with Eq. (4):

$$\Gamma_i^l=\frac{\boldsymbol{\phi}_i^T\mathbf{f}}{M_i} \text{ and } \Gamma_i^g=\frac{\boldsymbol{\phi}_i^T\mathbf{MTr}}{M_i}. \tag{4}$$

In which $\boldsymbol{\phi}_i$ and $M_i=\boldsymbol{\phi}_i^T\mathbf{M}\boldsymbol{\phi}_i$ are the modal shapes and mass respectively; **f** corresponds to the nodal distribution of a generic dynamic loading; and **Tr** is the product between the transfer matrix (**T**) and a boolean vector (**r**), used to select a specific direction. Hence the ground acceleration can be expressed as $\ddot{\mathbf{u}}_g(t)=\mathbf{T}\mathbf{a}_g(t)=\mathbf{Tr}a_g$.

The solution to Eq. (1) can be achieved assessing the contribution of each vibration mode separately. The displacements and accelerations corresponding to the *i*-th mode can be calculated with Eq. (5) and Eq. (6) respectively, based on the acceleration $A_i(t)$, or displacement $D_i(t)$, corresponding to each modal frequency ω_i.

$$\mathbf{u}_i(t) = \boldsymbol{\phi}_i z_i(t) = \Gamma_i^g \boldsymbol{\phi}_i D_i(t). \tag{5}$$

$$\ddot{\mathbf{u}}_i(t) = \boldsymbol{\phi}_i \ddot{z}_i(t) = \Gamma_i^g \boldsymbol{\phi}_i A_i(t) \cong \Gamma_i^g \boldsymbol{\phi}_i \omega_i^2 D_i(t). \tag{6}$$

The acceleration peak values $\ddot{\mathbf{u}}_{i,0}$ will determine the sensitivity requirements during sensor selection for SHM. Applying Eq. (7) the accelerations due to a seismic action can be calculated for each mode separately considering the normalized peak response acceleration $A_{i,0}$ of a simple oscillator for a given frequency and damping ratio.

$$\ddot{\mathbf{u}}_{i,0} = \max_t \left|\ddot{\mathbf{u}}_i(t)\right| = \Gamma_i^g \boldsymbol{\phi}_i A_{i,0}. \tag{7}$$

An analogue expression, Eq. (8), can be written for a generic dynamic loading, considering in this case the modal participation factor for this dynamic distribution Γ_i^l, as defined in Eq. (3).

$$\ddot{\mathbf{u}}_{i,0} = \max_t \left|\ddot{\mathbf{u}}_i(t)\right| = \Gamma_i^l \boldsymbol{\phi}_i A_{i,0}. \tag{8}$$

The global response can be obtained by combination of all modes responses according to SRSS rule.

$$SRSS:\ \ddot{u}_i = \sqrt{\sum_{j=1}^{n} \ddot{u}_{i,j}^2} \tag{9}$$

Therefore, if it is assumed that the peak response accelerations $A_{i,0}$ don't change very much between modes, the maximum response of the system can be simplified, and studied with only the modal participation factors and modal shapes, $\Gamma_i^g \boldsymbol{\phi}_i$ or $\Gamma_i^l \boldsymbol{\phi}_i$.

2.1 Natural Frequencies (NF)

Several authors and seismic design codes have given simplified formulation to estimate the fundamental period, or frequency, for regular buildings [7]-[17]. Table 1 summarizes some of the more recent formulas in this regard for masonry or reinforced concrete structures. The purpose of each author was to give a simple equation, in most cases considering only one parameter (the structural height *H*), to evaluate the natural period of a regular structure, which would be responsible for the main dynamic response. In other cases, like Eq. (14), more data are necessary for the evaluation of a shear walls building made in masonry [11]. The approach of each author has been either analytical, numerical or even based on experimental measures on real structures. Each one particularised the problem for some conditions, such us structural typology (bared or infilled RC frames, masonry walls, etc.) or geographic location (which can relate to construction techniques or material qualities). In this regard big differences can be observed between for example the formulas proposed by Hong and Wang [13] or Verderame et al. [17], for experimental measures on RC buildings in Taiwan and Italy, respectively. This fast period assessment is the first step of the methodology proposed in these guidelines. If there are some simplified equations to estimate the natural frequency of certain structures, there could also be simplified approaches to evaluate their dynamic response, i.e. accelerations levels under particular dynamic actions. Therefore, evaluated the accelerations, some recommendations can be made regarding the selection of optimal sensors, as a compromise between cost, performance and sensitivity.

Table 1: Simplified equations for natural frequencies assessment.

Authors	Year	Material	Formula	Ref	Eq.
Crowley & Pinho	2004	RC (PE)	$T_1 = 0.1H$	[7]	(10)
EC-8 / NTC 08	2004	RC	$T_1 = 0.075H^{3/4}$	[8][9]	(11)
		Masonry	$T_1 = 0.050H^{3/4}$		(12)
Faccio et al.	2009	Masonry	$T_1 = 0.0187H$	[10]	(13)
Goel & Chopra	2000	RC	$T_1 = 0.052H^{0.9}$	[11]	(14)
		Masonry	$T_1 = 0.0085\, H / \sqrt{\overline{A_e}}$ $\overline{A_e} = \frac{100}{A_b} \sum_{i=1}^{N_w} \left(\frac{H}{H_i}\right)^2 \frac{A_i}{1+0.83\left(\frac{H_i}{D_i}\right)^2}$		(15)
Guler et al.	2008	RC	$T_1 = 0.026H^{0.9}$	[12]	(16)
Hong & Wang	2000	RC	$T_1 = 0.0294H^{0.804}$	[13]	(17)
Masi & Vona	2010	RC	$T_1 = 0.085H$	[14]	(18)
		RC (PE)	$T_1 = 0.011H$		(19)
NSCE-02	2002	RC	$T_1 = 0.09n$	[15]	(20)
		Masonry	$T_1 = 0.06H\sqrt{\frac{H}{L(2L+H)}}$		(21)
Rainieri & Fabbrocino	2011	Masonry	$T_1 = 0.013H^{1.10}$	[16]	(22)
Verderame et al.	2009	RC	$T_1 = 0.071H^{0.96}$ $T_1 = 0.151H^{0.60}$	[17]	(23)
		RC (PE)	$T_1 = 0.272H^{0.75}$ $T_1 = 0.220H^{0.73}$		(24)

RC: reinforced concrete; PE: post-elastic; T_1: fundamental period; H: structural height; L: wall length; n: number of storeys; D: dimension in the direction under consideration; $\overline{A_e}$: equivalent shear area; A_b: building plan area; N_w: number of shear walls.

2.2 Modal Contribution Factor (MCF)

For this preliminary analysis three different simplified structural models have been studied, Fig. 1. The first one, regular structure (RS), is a two storey building with only one span. The second includes an irregularity in elevation (ISE), i.e. the first floor has two spans while the upper floor has only one. And the third is a one storey high building with four columns and an eccentric mass, thus generating an irregular structure in plan (ISP). If an undamped system is supposed, the motion equations for the RS case can be written as shown in Eq. (25). All parameters regarding, displacements, mass and stiffness are defined in the scheme of Fig. 1(a).

$$\begin{aligned} &m_{11}\ddot{u}_1 + k_{11}u_1 + k_{12}u_1 - k_{21}(u_2 - u_1) - k_{22}(u_2 - u_1) = 0 \\ &m_{22}\ddot{u}_2 + k_{21}(u_2 - u_1) + k_{22}(u_2 - u_1) = 0 \end{aligned} . \quad (25)$$

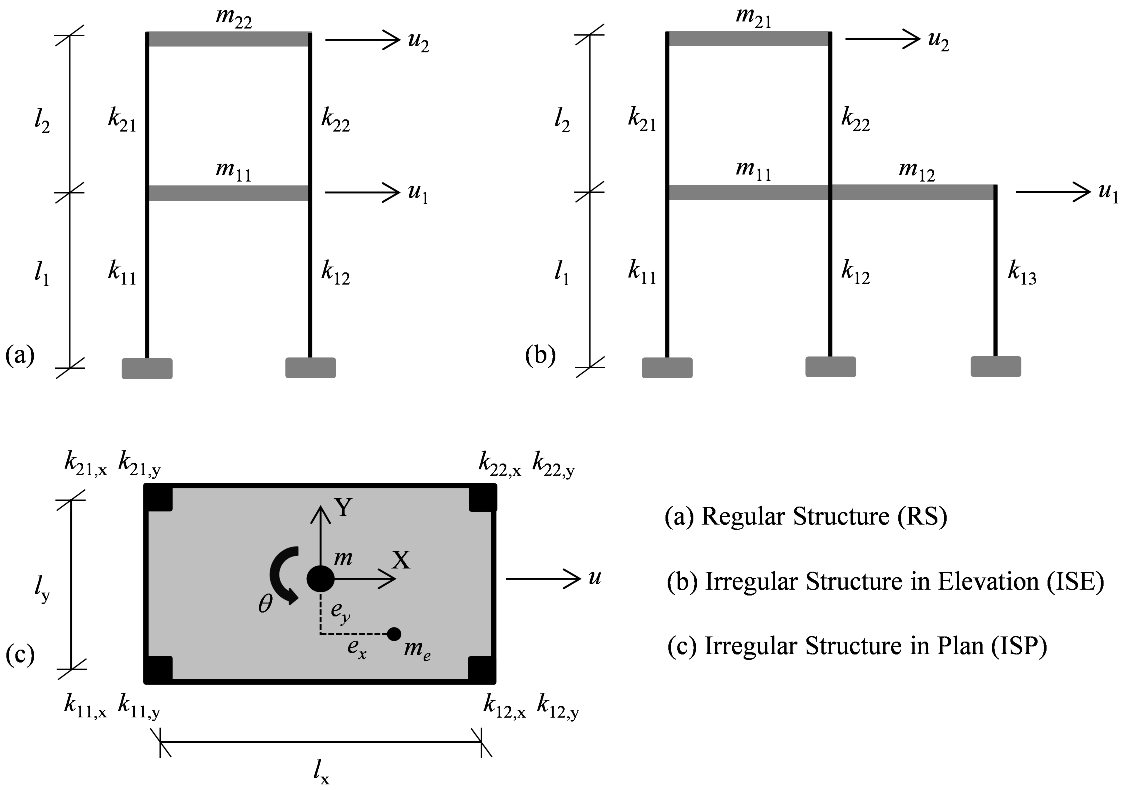

Figure 1 – Basic simple models representative of different structural typologies.

For the second case, ISE, the general equations can be written as Eq. (26), with similar notation, Fig. 1(b). And for the ISP case, the equations are included in Eq. (27), where I_ϑ and I_e are the inertias corresponding to the structural central mass m or the eccentric mass m_e, respectively. The stiffness of each support has been expressed as $k_{ij,x}$ or $k_{ij,y}$ where indexes *ij* refer to the number of the node; and *x* and *y* represent the maximum and minimum inertia axes respectively. In this case, all columns have been oriented for maximum inertia related to global X displacement (*u*).

$$\begin{aligned}&(m_{11}+m_{12})\ddot{u}_1+k_{11}u_1+k_{12}u_1+k_{13}u_1-k_{21}(u_2-u_1)-k_{22}(u_2-u_1)=0\\&m_{21}\ddot{u}_2+k_{21}(u_2-u_1)+k_{22}(u_2-u_1)=0\end{aligned}. \tag{26}$$

$$\begin{aligned}&(m+m_e)\ddot{u}+m_e\,e_y\,\ddot{\theta}+(k_{11,x}+k_{12,x}+k_{21,x}+k_{22,x})u=0\\&(I_\vartheta+I_e)\ddot{\theta}+\frac{l_y}{2}\theta(k_{11,x}+k_{12,x}+k_{21,x}+k_{22,x})\frac{l_y}{2}+\frac{l_x}{2}\theta(k_{11,y}+k_{12,y}+k_{21,y}+k_{22,y})\frac{l_x}{2}=0\end{aligned}. \tag{27}$$

All these equations, Eq. (25) to (27), can be rewritten assuming additional regularity conditions, such as regular mass or stiffness distribution. Hence, Eq. (25) can be expressed as Eq. (28) if $k_{11}=k_{12}=k_{21}=k_{22}=k$ and $m_{11}=m_{22}=m$ for a regular structure situation; for ISE case, Eq. (26) may be expressed as Eq. (29) if $k_{11}=k_{12}=k_{21}=k_{22}=k$ and $m_{11}=m_{22}=m$; and for the ISP example, Eq. (27) will be rewritten as Eq. (30) if $k_{11,x}=k_{12,x}=k_{21,x}=k_{22,x}=k_x$ and $k_{11,y}=k_{12,y}=k_{21,y}=k_{22,y}=k_y$.

$$m\begin{bmatrix}1 & 0\\ 0 & 1\end{bmatrix}\begin{pmatrix}\ddot{u}_1\\ \ddot{u}_2\end{pmatrix}+2k\begin{bmatrix}2 & -1\\ -1 & 1\end{bmatrix}\begin{pmatrix}u_1\\ u_2\end{pmatrix}=\begin{pmatrix}0\\ 0\end{pmatrix}. \tag{28}$$

$$m\begin{bmatrix}2 & 0\\ 0 & 1\end{bmatrix}\begin{pmatrix}\ddot{u}_1\\ \ddot{u}_2\end{pmatrix}+k\begin{bmatrix}5 & -2\\ -2 & 2\end{bmatrix}\begin{pmatrix}u_1\\ u_2\end{pmatrix}=\begin{pmatrix}0\\ 0\end{pmatrix}. \tag{29}$$

$$\begin{bmatrix}m+m_e & m_e e_y\\ 0 & I_\vartheta+I_e\end{bmatrix}\begin{pmatrix}\ddot{u}\\ \ddot{\vartheta}\end{pmatrix}+\begin{bmatrix}4k_x & 0\\ 0 & l_y^2k_x+l_x^2k_y\end{bmatrix}\begin{pmatrix}u\\ \vartheta\end{pmatrix}=\begin{pmatrix}0\\ 0\end{pmatrix}. \tag{30}$$

Using these equations a sensitivity analysis has been made to assess the influence of the mass variation on the structural response. For this purpose a modal analysis has been performed to estimate the natural frequencies (ω_i) and modal shapes (ϕ_i). Afterwards the modal contribution factors have been calculated with Eq. (4) for a unidirectional ground motion or generic dynamic loading (i.e. ambient vibration). The former assumes $\mathbf{Tr}=\begin{pmatrix}1\\ 1\end{pmatrix}$ for RS and ISE cases, while $\mathbf{Tr}=\begin{pmatrix}1\\ 0\end{pmatrix}$ for ISP in order to consider only translation displacements in the ground motion direction. The latter adopts a dynamic load distribution value $\mathbf{f}=\begin{pmatrix}1\\ 1\end{pmatrix}$ in the absence of a more detailed analysis. The nodal acceleration for each mode of vibration has been determined with Eq. (7) and (8) assuming a unitary peak response acceleration for all modes, i.e. $A_{i,0}=1$. A SRSS modal combination for the total response of each node was made for all cases. It is worth noticing that this nodal acceleration is independent of the normalisation method for modal shapes.

All these calculations have been made assuming a steel frame with $k=806.4\cdot 10^3\,N/m$ and increasing values of m between 100 kg and 2000 kg. Fig. 2 includes the results corresponding to the first two cases, RS and ISE. The nodal acceleration has been represented versus the mass value of a single storey. Two cases has been represented separately, i.e. unidirectional ground motion, Fig.2(a), and generic dynamic load, Fig.2(b). First, Fig.2(a) shows the results for a seismic action analysis, in which the accelerations for the first and second stories are plotted. In both cases, the acceleration values didn't depend on the total mass of the model. Regardless of the actual mass

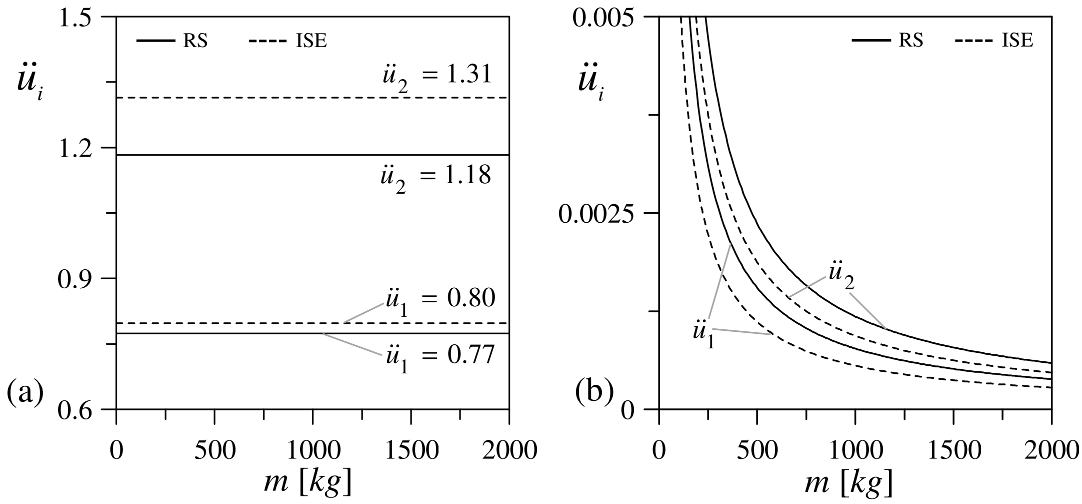

Figure 2 – Dependence of the floor normalised acceleration ($A_{i,0}=1$) by model mass: (a) unidirectional ground motion and (b) generic dynamic load.

Table 2: Modal accelerations for the models RS and ISE.

	Regular Structure (RS)		**Irregular Structure Elevation(ISE)**	
Mode	**1**	**2**	**1**	**2**
$\ddot{u}_{u1}$	0.724	0.276	0.761	0.239
$\ddot{u}_{u2}$	1.171	-0.171	1.283	-0.283

value, the maximum normalised acceleration was 1.31 and 1.18 for the ISE and RS respectively, i.e. accelerations were slightly higher (10% approx.) for the IRS model. These changes on the system's response are related to the modification of the modal parameters. Mass and stiffness matrices changed, as shown in Eq. (28) and (29), and therefore modal shapes and contribution factors were modified. The IRS model had higher stiffness in the first storey which led to two effects on its modal behaviour. First, modal shapes were slightly modified, and the relative displacements between first and second floors were increased, i.e. u_1 is proportionally lower (because it's stiffer than RS model) while u_2 is higher for both vibration modes. Furthermore, modal contribution factors were also modified, so second mode contribution increased and first mode diminished.

When both variables are combined using Eq. (7), for $A_{i,0} = 1$, nodal acceleration values for each mode can be obtained (Table 2), and this increase in the contribution of the second mode for the ISE model can be observed. If SRSS rule is applied total acceleration values as shown in Fig.2(a) are obtained. Therefore during the design of a SHM system only for seismic monitoring in a regular structure, or with irregularities only in elevation, the selection of sensors could be made without a thorough mass analysis, because the system response doesn't depend on this parameter. However, the mass value can modify the natural frequency and hence the $A_{i,0}$ value, and total accelerations. The effect for ambient vibrations (general dynamic load) was totally different, Fig.2(b). In this case models with higher mass showed much lower accelerations, as the modal contribution factor was reduced for increasing modal masses, Eq. (4), for a given dynamic force distribution. Besides, the acceleration levels were several orders of magnitude lower than the ground motion counterparts, but this depends on the force distribution $\mathbf{f}$ assumed. Consequently, if the monitored structure is more massive sensors should be more sensitive for an ambient monitoring, despite it wouldn't be necessary for a seismic monitoring.

Fig.3 includes the acceleration versus mass curves for ISP models with different eccentricity conditions. In this examples, only two DOF have been considered (*u* translation in X direction, Fig.1 and ϑ rotation around vertical axis), and two different situations have been calculated: one with different central masses (100 or 200 kg); and another where the eccentricity in Y direction changes between 1 and 1.5 m. In all these examples the effect of an increasing eccentric mass m_e has been assessed. An influence of this varying mass on the system respond can be mentioned as the main difference between these results and those shown in Fig.2. For the seismic response of ISP models, Fig.3(a), an increase on m_e resulted on higher nodal accelerations for both translational and rotational DOF. On the other hand, an increase on the initial mass, centred on the building plan, slightly decreases the total accelerations as m_e is higher. Actually, this decrease is only a temporary delay, i.e. if the initial *m* is higher an equivalent system with the same total eccentricity and lower m_e could be calculated. This effect can easily be seen in the asymptotic behaviour shown by both functions, in which for a $m_e = \infty$ the accelerations didn't depend on the initial value of *m*, as explained before. These asymptotic normalised accelerations were 1.47 and 0.99 for translations and rotations respectively. In any case, the existence of asymmetries in mass distribution can

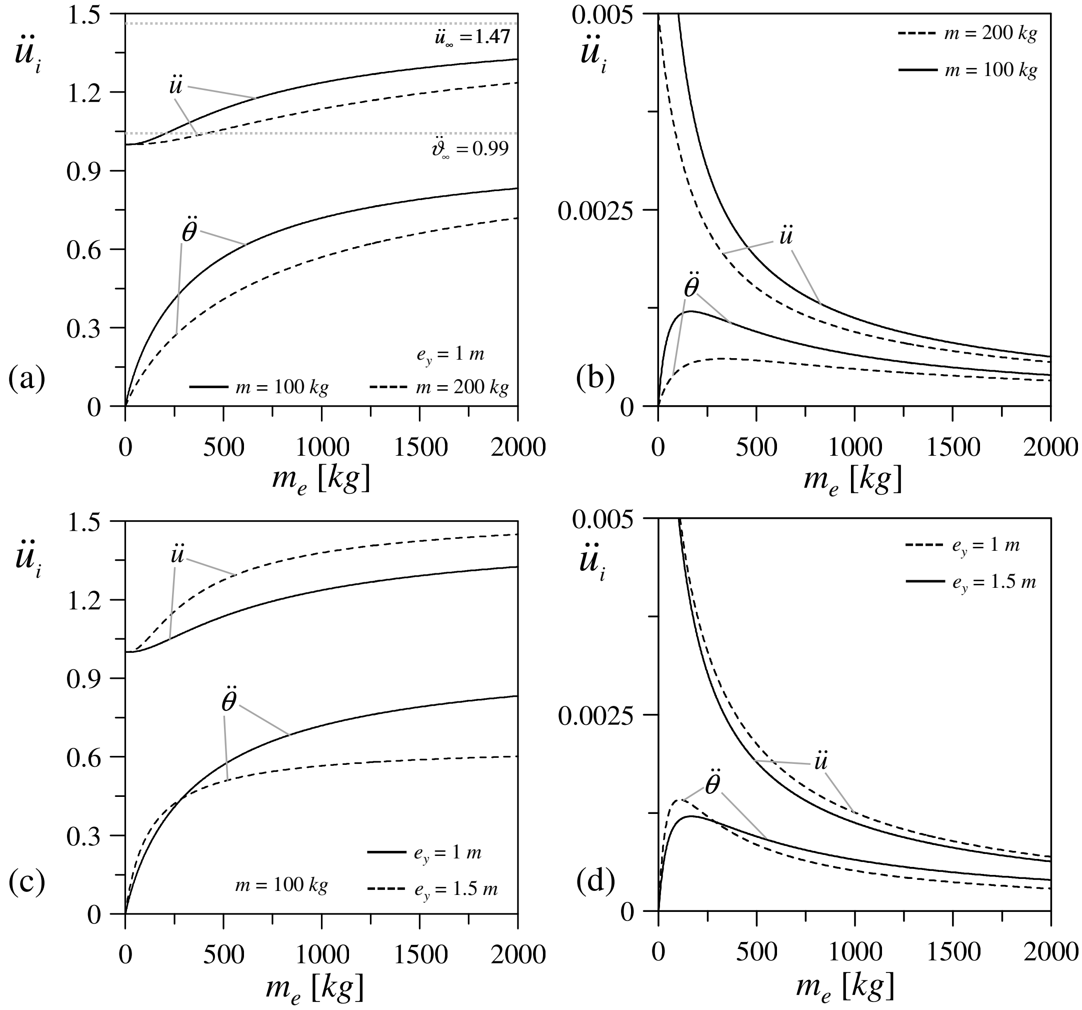

Figure 3 – Dependence of the floor normalised acceleration ($A_{i,0} = 1$) by the eccentric mass of ISP model: (a) and (c) unidirectional ground motion and (b) and (d) generic dynamic load.

increase the level of accelerations in a seismic monitoring scenario. Identical discussion can be made for the generic dynamic load distribution, Fig.3(b), in which higher centred mass lowered the accelerations for the same m_e. In this case, the asymptotic tendency makes the acceleration levels negligible, i.e. for $m_e = \infty$, translation and rotation acceleration tend to zero.

In the second case for eccentric masses, Fig.3(c), the maximum eccentricity is changed. The initial conditions, i.e. a centred mass m and an eccentric one m_e, can be represented as an equivalent system with only one eccentric mass, whose eccentricity varies from zero ($m_e = 0$) to e_y ($m_e = \infty$). Therefore this second scenario can be seen from a point of view where the only parameter that actually changes is the maximum eccentricity of the system. Thus for increasing eccentricities, the maximum acceleration (i.e. the asymptotic value for $m_e = \infty$) is increased. In particular, in the two cases calculated in this example, increasing e_y from 1 to 1.5 m, increases the maximum normalised acceleration ($A_{i,0} = 1$) for the translation DOF from 1.47 to 1.54.

3 INCIPIT PROJECT

The INCIPICT project, acronym of INnovatong City Planning through Information and Communication Technologies, regards the implementation of an experimental optical network integrated in a new infrastructure for the city of L'Aquila. Indeed, the reconstruction of L'Aquila, heavily damaged after the 2009 earthquake, is framed within the emergent concept of Smart City (SC) used to describe an area in which the life quality of citizens is improved by a pervasive employment of Information and Communication Technologies (ICT).

Among the different and transversal objectives of the project (dematerialization of Public Administration (PA) documents or management and continuity of PA's operations in emergencies) there is a specific work package regarding the innovative methods and services. These services comprise: (1) structural health monitoring (SHM), (2) disaster resilient and energy building automation and (3) cultural heritage enhancement. In particular, for the first task the idea is to implement a permanent and distributed SHM system easily accessible from the network. The most exciting challenge in medium to long-term concerns the complete integration between monitoring sensors network and monitored structure, until the implementation of intelligent systems, equipped with autonomous functions of self-analysis and self-diagnosis, together with energy self-sufficiency. The project will be realized in L'Aquila and will take advantages from the deep reorganization of the infrastructural system in the historic center of the city. The experimental optical network, Fig. 4(a), will have wireless access points, specifically addressed to the scientific community, and aimed at the development of new networking technologies and new services that can be based on such infrastructure.

Figure 4 – Experimental optical ring deployed in the center of L'Aquila (a). Two example of buildings belonging to the network: reinforced concrete (b), masonry (c).

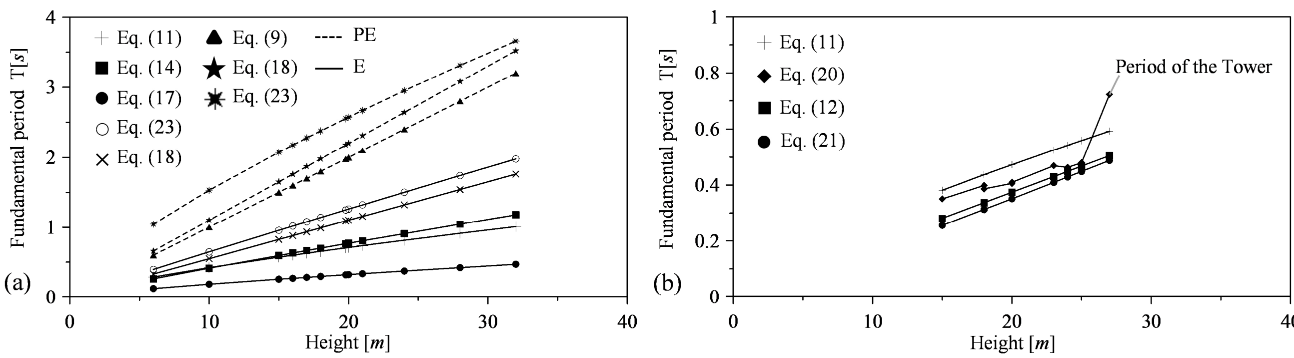

Figure 5 – Estimation of the fundamental period for the set of buildings belonging to the INCIPICT. project through the equations of table 1: (a) RC frames and (b) masonry walls structures.

Today, the network contains 31 buildings, 21 of which in reinforced concrete (RC), 9 in masonry and 1 in steel. In Fig. 4(b) and Fig. 4(c) there are two examples of buildings belonging to the network, respectively in reinforced concrete (Court of Appeals) and in masonry (Bank of Italy). At this stage of the project, one of the main problem concerns the rapid choice of the structural monitoring network for a large and varied number of buildings. The basic idea will be to create a link between the main geometrical (structural type, number of levels, total height, regularity in plan and elevation, presence of rigid floors) and structural (general criteria for the evaluation of natural frequencies, definition of criteria for the estimation of modal participation factors, evaluation of the main modal accelerations) characteristics and the optimal sensors. Moreover two buildings (one in RC and one in masonry) will be considered as benchmark in which will be realized a super sensorization useful to study in deep both the procedure for the rapid design of a SHM system and those for structural and damage identification. As explained before in 2.1., several authors have proposed different formulations to estimate the natural period of a structure. For example, based on structural typology, linear or power functions depending on buildings height were given in Table 1. Fig. 5 includes the evaluation of the fundamental period (T) for the buildings inside the INCIPIT project evaluated for several of the functions included in Table 1. Fig.5(a) represents those made in RC, and for the same building very different values of T were obtained. Therefore, the first approach of this network should be to narrow the possibilities to an equation adapted to the constructive characteristic of the city of L'Aquila. Afterwards, the proposed approach of assessing the modal accelerations for the SHM system design could be addressed.

4 PRELIMINARY CHOICE OF THE SHM SENSORS

In this paragraph is described a first and approximate attempt to correlate the numerical results obtained in the previous sections and the choice of the SHM sensors. Regarding the classification of the sensing device, a possible partition can be made on the base of their sensing performance. A first simplified division can be the following: class A (high performance), in which are collocated, for example the traditional force balance accelerometers, class B (medium performance), where the MEMS sensors with a closed loop control scheme can be inserted and class C containing MEMS sensors with open loop operation principle. In Fig. 6 is reported a matrix that links the choice of the SHM systems' performance once defined both the geometric characteristics (regular, irregular in elevation, irregular in plan) and the material (masonry, RC, steel or wood) of the building. In each box of the matrix is indicated the type of system for both seismic (subscript "S") and environmental monitoring (subscript "E").

High mass (Masonry/Church)	A_S - A_E	B_S - A_E	C_S - A_E
Medium mass (R.C.)	B_S - B_E	B_S - A_E	C_S - B_E
Low mass (Steel/Wood)	B_S - B_E	C_S - B_E	C_S - C_E
	RS	ISE	ISP

Figure 6 – Correlation between the structural typologies (geometry and mass) and the choice of the SHM systems' performance.

The main observations about this correlation are the following: (1) for the regular structure the choice of a setup with medium performance satisfies 66% of the cases and could be difficult to carry out both seismic and environmental monitoring for buildings with high mass; (2) for irregular structures are covered 75% of the cases if sensors belonging to a medium-low class are selected and likely only the monitoring under environmental noise could be unsuccessful; (3) the class of buildings with medium mass, in which, reasonably, the reinforced concrete frame structures can be collocated, the 83% of the possibilities is reached selecting a medium-low class. Instead, for buildings with high mass, 83% of the cases would require a medium-high class of sensors.

5 CONCLUSIONS

The paper has proposed a possible strategy to make a rapid design of a network of SHM systems for a wide area in which are collocated buildings with heterogeneous characteristics (as materials or geometric configurations different). It is based on the estimation of the buildings' main dynamical properties as the Natural Frequencies or the Modal Contribution Factor. Three simplified analytical models representative of the regular, irregular in elevation and irregular in plan structures have been hypothesized. For these models a parametric analysis has been performed to understand the dependence of the MCF by the mass (for the two first models) and the eccentric mass (for the last one model). The numerical investigation has been carried out for two types of excitations: generic dynamic load (environmental monitoring) and unidirectional ground motion (unidirectional ground motion). The results has been used to implement a matrix that links the different structural typologies and the choice of the SHM systems' performance. The methodology needs of further insights as for example the assessment of the influence of the buildings' height. Moreover it will be tested and checked in the case of the INCIPICT project.

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