# HOW FAR IN SITU MEASUREMENTS MAY HELP TO ASSESS BUILDING VULNERABILITY?

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**ABSTRACT:**

The use of noise data in seismic diagnosis of buildings is analyzed. From the responses to ambient noise, harmonic excitation and shocks, the dynamic behaviour of usual buildings is identified in the range of $10^{-5}-10^{-2}g$. Taking advantage of the demolition, the influence of the light work elements, full precast façade panels, bearing masonry walls and the presence of neighbouring joined buildings is determined. These experiments show that noise measurements efficiently provide reliable data of real interest for understanding the actual building behaviour. Then, the integration of these data in a vulnerability diagnosis is presented. It is shown that regular concrete structures are described by suited beam modelling. Thus for a given structure, taking into account the noise data, the adequate beam model and maximum tensile and compression strains of concrete and steel as damage criteria, two levels of ground acceleration can be determined, namely the Seismic Thresholds of Elasticity (STE) and of Yielding (STY). Quantify the levels that onset the structural damages and the plastic hinge may be a useful tool for vulnerability diagnosis. This concept presents the practical advantages to be based on real data, to minimize the introduction of uncertain assumptions and to provide an acceleration level easily comparable with the reference acceleration of the codes.

**KEYWORDS:** In situ monitoring, beam modelling, seismic vulnerability.

## 1. INTRODUCTION

The assessment of the state of existing structures now deserves a particular attention for many reasons among which the need of retrofitting according to new seismic rules … However, the lack of reliable information on the structure drastically increases the level of uncertainty of any diagnostic. A simple way to overcome these difficulties is to reduce the deficiency of information by means of dynamic monitoring tests (Hudson, 1970). The data extracted from experiments could help to define the leading phenomena that govern the behaviour of the real structure and, on the basis of pertinent observations, to propose simplified tough realistic models efficient for a first diagnostic level. Following this idea, this paper is an attempt to define a methodology illustrated and discussed at the light of experiments realized on two typical buildings. This work is a part of an program carried out on several existing buildings before and during their demolition (Boutin &al., 2005) (Hans, 2002&05). The aims of this study are: (i) identifying the dynamic behaviour of usual building designed according to the French rules and taking advantage of the demolition to analyse the actual influence on the modal characteristics of the light work elements like secondary dividing walls… (ii) determining an equivalent simple beam in relation with the internal structure, (iii) imagining a new tool called Seismic Thresholds (ST) integrating this experimental model and simple criteria on the constitutive materials.
The paper is divided in three parts. The first part is devoted to the experimental results obtained on the intact or partially modified building. In the second part, it is shown that a Timoshenko beam simply deduced from the internal structure describes the dynamic behaviour with a good accuracy. The ST method is then presented through an example and its interest for practical applications is discussed.

## 2. EXPERIMENTAL INVESTIGATIONS

### *2.1. Experimental procedure*

In situ auscultation methods consist in recording the response of the structure using accelerometers fixed at different level. Three different types of excitations were used to measure frequencies and to identify mode shapes and non-dimensional damping ratios: ambient mechanical noise (AN), harmonic excitation (H) and shock loading (S). An accurate description of these techniques is available in (Hans, 2005). What is worth noting is the fact that, although the acceleration level ranging between $10^{-5}$g (AN) to $10^{-3}$g (S) at the base of edifice, determined modal parameters are similar whatever the excitation used and the behaviour of tested building stays in elastic domain.

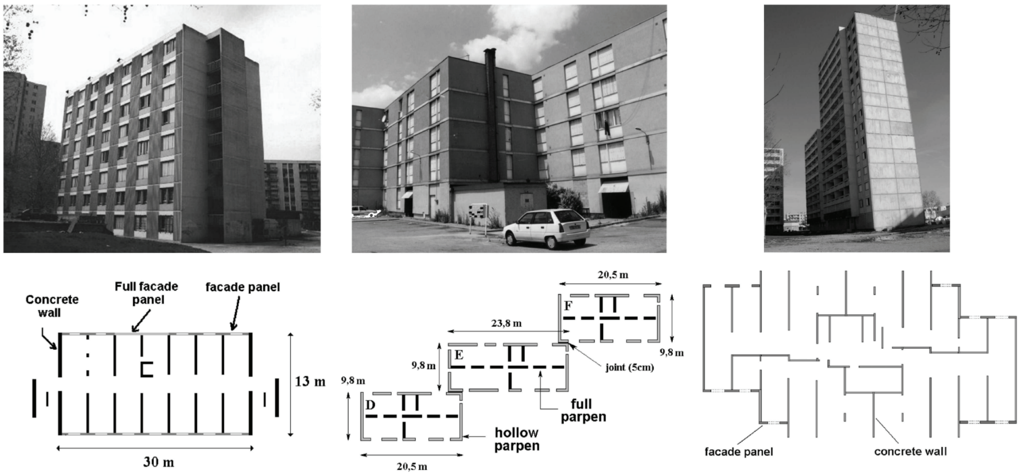


Figure 1. Photos and floor plans of building 1 (left), buildings 2-3-4 (middle) and building 5 (right).

Table 1. Characteristics of tested buildings

| | Building 1 | Building 2&4 | Building 3 | Building 5 |
|---|---|---|---|---|
| Number of stories | 8 | 5 | 5 | 16 |
| Height H (m) | 21.6 | 14.1 | 14.1 | 43.2 |
| Length L (m) | 30.0 | 20.5 | 23.8 | 31.4 |
| Width W (m) | 14.0 | 9.8 | 9.8 | 13.4 |
| H/W - H/L - L/W ratios | 1.5 – 0.7 – 2.1 | 1.45- 0.69 – 2.1 | 1.45 – 0.59 – 2.44 | 3.2 - 1.4 – 2.3 |
| Story specific mass (t/m$^3$) | 0.27 | 0.23 | 0.23 | 0.25 |
| Linear mass (t/m) | 112 | 46 | 53 | 110 |

### *2.2. Tested structures*

The tested buildings are presented in Figure 1 with a general view of their floor plan. The buildings 1 and 5 are made of floors and shear walls in reinforced concrete with precast facade panels; walls of buildings 2 to 4 are realized with masonry of parpens (full or light). Located in suburbs of Lyon and in good state, their demolition is subsequent to a urban re-structuring. Built in the sixties and seventies, they are representative of number of modern buildings constructed at this period. Typical characteristics are:

- the structural regularity in plan (with transverse and longitudinal plans of symmetry),
- the structural regularity in elevation (same structure of all the storeys),
- the weak amount of steel bars in the reinforced concrete elements (having the standard thickness of 15 cm) and the very poor steel reinforcement in the precast panels (verified during the demolition).

In Table 1, dimensions and slender parameters of the buildings are shown. The story density and the building linear mass, deduced from the plans and the usual density value of constitutive materials, are also presented. The building basement is founded on shallow foundations. The soil is a mixed gravel and clay deposits, which presents good mechanical properties (a measurement of the shear velocity gives around 300 m/s).

### *2.3. Framework*

The in-situ measurements naturally lead to the modal characteristics of the structure coupled with the soil. When the soil is of good quality, as in the present case, the contribution of the soil-structure interaction is weak and the modal parameters of the structure coupled with the soil (SS) are close to those of the structure lying on a rigid motionless basis (SB), at least for the first mode. Nevertheless, it is interesting to derive the own modal characteristics of the structure on fixed base, corresponding to the intrinsic properties of the structure. In the simple case of ambient vibrations, the intrinsic behaviour of SB structure is deduced by suppressing the rigid body motion induced by the base motion (Hans, 2005). For the harmonic forcing and the impact method for which the base motion is mainly due to the soil-structure interaction, the identification of the SB behaviour from the SS behaviour requires more sophisticated methods.

Table 2. Modal characteristics of tested buildings (Soil-Structure point of view)

| Mode | Monitoring method | Building 1 | | | | Building 5 | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | Longitudinal direction | | Transversal direction | | Longitudinal direction | | Transversal direction | |
| | | f (Hz) | ξ (%) | f (Hz) | ξ (%) | f (Hz) | ξ (%) | f (Hz) | ξ (%) |
| 1 | Ambient | 4.3 | 2.85 | 4.6 | 3.1 | 2.08 | 2.5 | 1.56 | 1.3 |
| | Harmonic | 4.19 | 2.6 | 4.56 | 3.4 | 1.94 | 2.3 | 1.48 | 1.5 |
| | Shock | 4.18 | 4 | 4.54 | 3.5 | - | - | - | - |
| 2 | Ambient | 13.4 | 3.5 | 17 | - | 7 | 2.5 | 6.6 | 4 |
| | Harmonic | - | - | - | - | 6.73 | 2.4 | 6.17 | 2.3 |
| | Shock | 12.8 | 4 | 15.7 | 5 | - | - | - | - |
| 3 | Ambient | 23 | 4 | - | - | 12.8 | - | 13.5 | 5 |
| | Shock | 22.5 | 3.8 | - | - | - | - | - | - |
| 4 | Ambient | - | - | - | - | 20 | 4 | - | - |

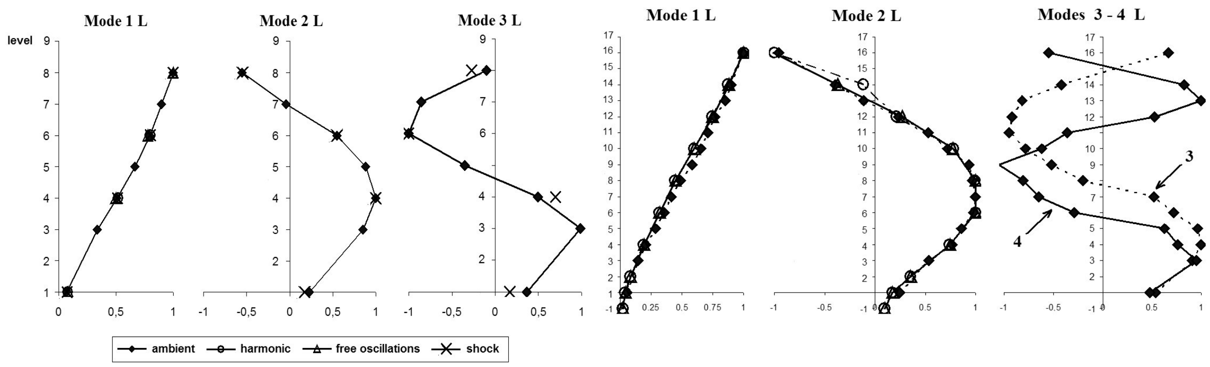


Figure 2. Longitudinal mode shapes of building 1 (left: three modes) and building 5 (right: four modes)

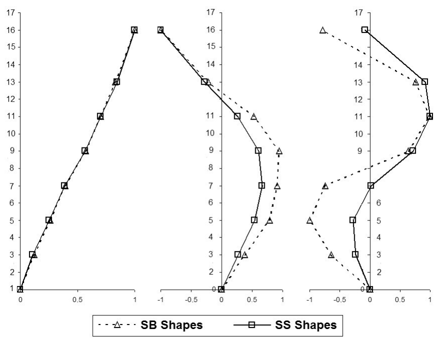


Figure 3. Comparison between (SB) and (SS) longitudinal mode shapes of building 5.

Table 3. Comparison between SS and SB modal frequencies of buildings 1 and 5.

| Direction | Mode | Building 1 | | Building 5 | |
|---|---|---|---|---|---|
| | | SS (Hz) | SB (Hz) | SS (Hz) | SB (Hz) |
| Longitudinal | 1 | 4.32 | 4.45 | 2.08 | 2.15 |
| | 2 | 13.5 | 14.1 | 7 | 7.25 |
| | 3 | 23 | 23.6 | 12.8 | 14 |
| Transversal | 1 | 4.6 | 4.66 | 1.56 | 1.56 |
| | 2 | - | - | 6.6 | 6.65 |
| | 3 | - | - | 13.5 | 14 |

### *2.4. Experimental results about intact structures*

The signals are processed with several usual techniques performed in both spectral (peak-picking and bandwidth) and temporal domain (autocorrelation functions). The experimental results obtained in terms of modal frequencies, damping ratio and modal shapes are presented in Table 2 and in Figure 2. For all the tested buildings, a very good agreement has been obtained between the results given by different vibration methods. These experimental findings confirm that from small amplitudes of ambient vibrations to significantly larger amplitudes reached with shocks, the structure presents the same quasi-elastic behaviour. Consequently, the ambient measurements are sufficient to determine the behaviour valid in the whole quasi-elastic domain.

The soil-structure interaction is now examined. The modal shapes presented Figure 2 are related to the SS system (Section 2.3) that includes the soil participation. Clearly a displacement of the base is visible, increasing from the first to the higher modes. The deduced characteristics of the structures on fixed base (SB) are compared in Table 3 and Figure 3 with SS modal characteristics for building 1 and 5. As expected, the SB eigenfrequencies are higher than SS ones, because of the softness induced by the soil. Moreover, the differences grow from the first to the higher modes. This is consistent with the increase of the modal rigidity of the structure with modal number, leading in turn to increase the soil-structure interaction:
- for the first and second modes, the modal structure rigidity is too low to initiate significant interaction with the soil, and effectively there is small differences between SB and SS modal characteristics,
- for higher modes, the modal structure rigidity grows quickly and larger modifications of the modal characteristics appear. It is worth to mention that, for studied building, this effect is rather limited (less than 5 %)

### *2.5. Experimental results about modified structures*

The periods of gradual demolition were systematically used to investigate several aspects as the level of participation of light work elements, masonry walls or facade panels, and the importance of interaction between neighbouring buildings. The detailed studies can be found in (Hans, 2005). Only the lessons are drawn hereafter:

1. Rule of light work elements (building 5): cause of the recycling of the concrete implying any contamination, all light work elements (doors, windows, dividing walls ...) were properly taken off before the demolition. A decrease of eigenfrequencies of about 3-4% was observed between intact and naked structure, what implies this effect can be neglected in first analyse.
2. Rule of masonry (building 4): in this building, the walls – constituted of light parpens – of one of the shorter facade was removed (on 4 levels), leaving intact the rest of the structure. A drastic decrease (-10%) of the first eigenfrequency in the direction of the wall plane has occurred whereas no modification in the perpendicular direction was observed. Furthermore, because of the off-centre position of these walls, a torsion mode has appeared. By inverse interpretation, the stiffness of each wall was evaluated to 25% of the storey stiffness. To conclude, despite the presence of internal walls of heavy parpen, the contribution of these light parpen walls (in their shear direction) is very significant and cannot be neglected. This result is of first importance for framed building with masonry infill.
3. Rule of precast facade panels (building 1): the participation of full precast facade panels was studied by their progressive removing in the first stories. A important and regular decrease of the first eigenfrequency in the direction of their plane was associated to a loss of about 20 % of the storey stiffness for each panel. This confirms that full panels in their shear direction cannot be neglected for interpreting the modal behaviour. Let us mention that, as the parpen walls, the panels are almost unreinforced and would not present any ductility.
4. Importance of neighbouring joined buildings (building 2): the possible mutual influences of close building was investigated in the group of the three similar buildings 2-3-4 : the ambient vibrations of the intact building 2 was recorded in presence of buildings 3 and 4, then again after demolishing 4, and finally after both 4 and 3 were destroyed. In both directions, the successive demolitions induced a systematic decreasing of the frequencies. The differences are not negligible and can reach 10 % in the more significant case. This experiment tends to show that the mutual influence, namely the structure-soil-structure interaction, may play a role, especially for close buildings presenting almost the same features (and eigenfrequency).

## 3. MODELLING OF DYNAMIC BEHAVIOUR

Can the fixed base behaviour of these regular buildings be reduced to the behaviour of an equivalent vertical continuous cantilever beam? The practical interest of this question specially arises for buildings presenting a sufficient number of storeys (say more than five), since a continuous description is much more convenient than a description with a large number of discrete variables. Moreover, a beam model provides a very synthetic way to condense the essential parameters of the dynamic behaviour. To model such periodic structures, a homogenization approach was used, whose developments and results are presented in (Boutin, 2003) and (Chesnais, 2008). It appears the dynamics of these structures is driven by three effects – the shear, the global bending and an inner bending. The dynamic behaviour is defined by the contrast of the corresponding stiffnesses (shear stiffness K, global bending ones EI and inner bending ones $EI_\mu$). Various types of beams are also obtained, from unusual as sixth degree when all effects are present to more usual as Timoshenko, Euler or shear beams when only one or two effects remain.

### *3.1. Timoshenko beam model*

Note that the inner bending effect can only participate if a gap in the bracing of the storey exists, like corridor going through the entire building (see building 1 for the transverse motions). Consequently, in absence of such a gap, regular buildings behave as a slender Timoshenko beam. This model results from two contributions: (i) the bending motion characterized by the bending parameter EI, (ii) the shear motion characterized by the shear parameter K. In harmonic regime, the horizontal translation motion U of the section (with $\omega$ the angular frequency and t the time) is governed by:

$$EI\, U^{(4)}(x) + (EI/K)\, \Lambda\, \omega^2\, U^{(2)}(x) = \Lambda\, \omega^2\, U(x) \tag{3.1}$$

By introducing the wavelength $L = 2H/\pi$ (i.e. the first modal wavelength of a clamped-free shear beam) and the variable change $y=x/L$, the equation (3.1) becomes:

$$C\, U^{(4)}(y) + C\, \Omega^2\, U^{(2)}(y) = \Omega^2\, U(y) \tag{3.2}$$

where $\Omega^2 = \Lambda\, \omega^2\, L^2 / K$ is a dimensionless angular frequency and $C = EI/KL^2$ is a dimensionless structural parameter. The dimensionless parameter C characterizes the nature of the Timoshenko beam. This latter degenerates into a usual Euler-Bernoulli beam when C = 0 and into a pure shear beam when C = +∞.

Table 4. Characteristics of buildings 1 and 5 – estimated from plans or (in bold) from experimental data.

| Building<br>Direction | Building 1<br>Longitudinal | Building 5<br>Longitudinal | <br>Transversal |
|---|---|---|---|
| Linear mass $\Lambda$ ($t/m^3$) | 114 | 110 | |
| Shear parameter K (MN) | 11895 | 27830 | 115600 |
| Bending inertia I ($m^4$) | 2140 | 1836 | 354 |
| C parameter | 19.7 | 1.79 | 0.08 |
| Estimated frequencies (with E = 20 GPa) | 3.63 – 11.5 – 17.8 | 2.58 – 7.91 – 14.12 | 2.24 – 10.54 – 23.07 |
| Experimental frequencies | 4.45 – 14.1 – 23.5 | 2.15 – 7.24 – 13.97 – 20.5 | 1.56 – 6.64 – 14.0 |
| ***Experimental ratio $f_1/f_2$*** | **-** | ***3.37*** | ***4.26*** |
| ***Experimental C*** | **-** | ***0.5*** | ***0.13*** |
| ***Experimental E (GPa)*** | ***31*** | ***21*** | |
| ***Refitted frequencies*** | ***4.45 – 13.3 – 21.8*** | ***2.15 – 7.24 – 13.96 – 20.1*** | ***1.56 – 6.64 – 14.0*** |

### *3.2. Assessment of the Timoshenko beam parameters for buildings 1 and 5:*

To go further in the possible identification of the building with a beam, we have now to specify how the shear (K) and bending (EI) stiffness and linear density ($\Lambda$) can be determined. Two types of estimation are used:
(i) the first based on the plans of the structure, on simple assumptions on the storey deformations – rigid floor – and by taking usual values for specific mass of concrete ($2.3 t/m^3$) and for Young's modulus (20 GPa),
(ii) the second based on the repartition of the experimental eigenfrequencies, leading to a unique value of the C parameter and the value of the first frequency to estimate the material modulus.
Table 4 gives the numerical results obtained for building 1 (in the longitudinal direction) and building 5 in both longitudinal and transversal directions. The values of parameter C, obtained by the first method, indicate a dominating shear beam behaviour for building 1 (C=20) in the longitudinal direction, and a Timoshenko beam behaviour in both directions for building 5 ($0.1 < C < 2$). The first three frequencies calculated from these estimations are comparable with the experimental values (at least for these modes), meaning that this simple approach can provide a reasonably good description of the building behaviour.

The second method applied on building 1 leads to estimate a Young's modulus to 31 GPa and a better fitting of the modal frequencies is obtained. Concerning the building 5, the fitting process performed independently for both directions leads to an identical modulus value of 21 GPa; and a very good estimation of the three (or four) first frequencies is obtained. Furthermore, the experimental and theoretical modal shapes – presented in (Chesnais, 2008) – are very similar.

## 4. METHOD OF SEISMIC THRESHOLDS (ST)

The question addressed here is: how could these in situ experimental data, consistently described by a beam modelling suited to the structure, contribute to a seismic diagnosis? Since a reliable description of the (quasi) elastic building behaviour (including all the mechanically active elements) is available, the first idea consists in determining the limit of this elastic domain. More precisely, it is intended to estimate the seismic acceleration level – related to the normalized earthquake spectra given by the code - which generates the onset of first structural damages. Beneath this level, called Seismic Threshold of Elasticity (STE), the structure remains elastic, i.e. undamaged. In the same spirit, the second idea is to estimate the level corresponding to the onset of plastic hinge, called Seismic Threshold of Yielding (STY). For determining these thresholds, *strain* criteria for concrete and steel are adopted. Note that, in addition to simplicity, this choice overcomes the lack of information on the amount and position of steel bars. In fact, this deficiency avoids the use of stress criteria that should necessarily include the reinforcements. Note also that strain criteria are consistent with the displacement-based vulnerability assessment methods (Priestley, 97) or the concept of maximum story drift (Gulkan, 96).

### *4.1. Criteria for the onset of structural damages*

As for the onset of structural damage, a criterion of maximum tensile strain of concrete is taken. In fact, whatever the amount of reinforcements, the concrete matrix cannot sustain tensile strains greater than $10^{-4}$ (m/m) (for usual concretes). Below this limit, the concrete (and thus the reinforced concrete) remains intact; above, the cracking of the concrete begins and weakens the reinforced concrete elements. Note that the maximum tensile strain can be adapted for other material (e.g. parpen).

### *4.2 Criterion for the onset of plastic hinge in reinforced concrete columns*

As for the onset of plastic hinge, a criterion of maximum elastic strain of steel and compression strain in concrete is taken. Indeed, the failure by compression of the concrete matrix and the yielding of usual steel bars begin for strains greater than $10^{-3}$ (m/m). Note that up to this strain level, the damage of the concrete in the columns remains limited: typically, the unconfined concrete cover is ejected at the extremities of the columns. This localized reduction of the effective section and inertia acts as a softening of the connections. This effect being limited, its impact is weak on the global stiffness of the structure. Thus it can be admit in first approximation that after the onset of concrete damages and before the onset plastic hinges, the first modal shape and frequency are almost unaffected (say a decrease around of 10 percent), at least for framed structures. On this point, remind that for buildings of 5 storeys and more, if the stiffness of the first level is divided by 2 (resp. 5), the frequency is only reduced of 20% (resp. 40%). This might not apply to walls, whose shear cracks affect the whole element.

### *4.3 Seismic Thresholds of damage*

The Seismic Threshold of Elasticity (STE) is deduced from:
- the quasi-elastic behaviour identified from ambient vibration tests,
- the beam model deduced from experiments suited to the building structure,
- the damage criterion of concrete.

It is clear that, if the structure was purely elastic, its temporal response to any seismic shaking could be fully determined. Obviously, this response will coincide with that of the real structure until the onset of damage, afterwards 'elastic' and 'damaged' responses begin to diverge. Consider now signals respecting the normalized earthquake spectra, whose amplitudes are characterized by the normalized accelerations. In the quasi-elastic domain, the response of the building increases proportionally to the amplitude of the signal, i.e. the normalized acceleration. This will be true until a first damage appears in concrete somewhere in the structure. At this moment, the corresponding normalized acceleration will be called the Seismic Threshold of Elasticity (STE). Note that the localization of the first structural damages can be achieved from the deformed structural shape defined according to its quasi-elastic vibration modes.

The Seismic Threshold of Yielding (STY) is defined in a similar way. However the additional assumption of weak non linear effect before yielding must be introduced. The threshold expressed in terms of normalized acceleration is reached when somewhere in the structure the *strain in the vertical direction* (of the steel bars) reaches the elastic strain limit of steel.

### *4.4 Calculation of the thresholds using the first mode approximation*

In the frame of these assumptions, the calculation of the thresholds could be performed through common linear dynamic numerical methods, the model being fitted by the experimental data. In order to give a better insight of the method, the calculations are performed using the first mode approximation. This latter consider that the first mode is mainly responsible for the structural deformations. This simplification can be partially justified: (i) the distribution of seismic energy is such that for buildings with well separated eigenfrequencies (e.g. 4.45 Hz and 14.1 Hz for building 1), the maximum of energy is concentrated on the first mode, (ii) the participation factor, then the effective amplitude, is smaller for higher modes.

It is then possible to straight estimate the amplitudes of the first mode displacements, which would lead to the onset (i) of structural damages and (ii) of plastic hinges. Using the normalized elastic response spectra provided by the codes, these modal amplitudes can be converted into an acceleration levels, i.e. the STE and STY.

### *4.5 Practical interest of the Threshold values*

The Seismic Threshold of Elasticity corresponds to a seismic elastic limit, whereas the safety strategy against earthquake is based on ductility. In these conditions, it is important to clarify why the knowledge of the STE could be of interest for the assessment of the vulnerability:

- as the STE value is based on measured data and does not require any supplementary assumptions on the post elastic behaviour, the uncertainty is minimized,
- the fact that the STE is associated with normalized elastic response spectra should mean a real benefit for earthquake engineering practitioners. Moreover, site effects could be easily integrated by using specific spectra suited to the site,
- the comparison of the STE value with the level of acceleration required by the seismic code gives an assessment of the ductility that the structure should be able to develop; this can be a useful tool to identify the more critical cases, or to define a strategy of reinforcement,
- for the large number of buildings made of materials of low (slightly reinforced concrete) or very low (masonry) ductility, the STE value can be a close indicator of the acceleration level leading to severe damages (nevertheless the reserves of stability brought by hyperstatism may preserve from collapse),
- finally, for specific buildings that should be kept in service, the STE value should help to estimate if the damage level remains acceptable.

Beyond the Seismic Threshold of Elasticity, for framed reinforced concrete buildings, the post-elastic behaviour is expected to induce limited damages in the columns up to the STY level. Beyond STY, significant damages can be expected depending on the ductility potential and its effective use during earthquake:

- if the quasi-elastic mode shape clearly shows a level which concentrates the deformation (for instance in presence of a ’transparent’ level), the strength will essentially depends on the local ductility at this level, the ductility of other parts of the building remaining almost unemployed,
- if the design ends up in a regular mode shape, the ductility will be activated in the whole building, and after the onset of plastic hinge (STY), the strength can be estimated using a push-over analysis (assuming or knowing the amount and disposition of steel reinforcement).

To sum up, if the STE is lower than the acceleration required by the seismic code, it is believed that first damages would be induced by the reference earthquake. The gap between the Thresholds values and the reference acceleration of the seismic zone provides an indication of the ductility needed by the building to resist to the reference earthquake. The larger this gap is, the more attention should be paid to the structure.

### *4.5. STE calculation for building 1*

As an example, the STE value is determined for the building 1 in the longitudinal direction, whose dynamic behaviour is well described by a shear beam. The first step consists in estimating the maximum inter-story drift $\Delta u^{lim}(i)$ by level i what be supported by structural (bearing or not) elements. Considering the damage criterion $\varepsilon = 10^{-4}$ for concrete, the determination of $\Delta u^{lim}(i)$ is done by simple calculation for each element, and by taking the minimum for each storey. Then let introduce $[U_1]$ and $[\Delta U_1]$, the normalized first mode eigenvector and the corresponding differential displacement vector, respectively. If A is the amplitude of the first mode of vibration, then the differential displacement $[\Delta u_1]$ vectors is: $[\Delta u_1] = A[\Delta U_1]$. The amplitude $A_i$ that would trigger off the first structural damages at the floor level i is calculated by writing: $\Delta u^{lim}(i) = A_i\ [\Delta U_1]$. Finally, the amplitude $A_{lim}$ which triggers off the first damage in the whole building is the minimum of the $A_i$ values. At this stage of the analysis, it remains to transform the value of $A_{lim}$ into an acceleration level. Conveniently, the seismic codes give the normalized elastic response spectra, i.e. the maximum response of a series of single-degree of freedom oscillators (SDOF) submitted to signals conform to the seismic spectra, with a reference acceleration of $a^* = 1 m/s^2$. According to the modal analysis, if $d^*(f_1)$ is the maximum SDOF's displacement response given by the normalized elastic response spectra at the 1st mode frequency $f_1$, then the amplitude of modal response of the structure will be, for a standardized acceleration $S.a^*$: $A(S) = S.p_1.d^*(f_1)$ in which $p_1$ is the first modal participation factor ($p1 = \pi/4$ for pure shear beam). The STE is reached for a standardized acceleration STE $a^*$ such that $A(STE) = A_{lim}$, i.e., $STE = A_{lim} /(p_1.\ d^*(f_1))$. Therefore, the STE, i.e. the level of ground acceleration corresponding to seismic elastic limit of the building can be derived according to the code recommendations.

### *4.3. Results and discussion*

Considering a damping ratio of 5%, the STE values of building 1 in the longitudinal direction are presented in Table 5 considering that the structure is settled on different site conditions, from $S_0$ (very good soil) to $S_3$ (soft soil). To investigate the sensibility of the STE, the same calculations were developed for a fictitious building 1* identical to building 1 but with a number of storey reduced to 4 (therefore more rigid with an higher first frequency). It can be seen that, according to the site conditions, the STE values of the building 1 range between 0.05 g and 0.08 g. This order of magnitude is in agreement with the post-earthquake observations which showed that below 0.1 g, there are very limited structural disorders in common concrete buildings. The STE values of building 1*, less solicited because of its higher frequency, take higher values (0.09 g to 0.18 g).

Table 5. STE values for buildings 1 and 1* - reference acceleration $a_{ref}$ for each zones: (zone $I_a$: 0.1g), (zone $I_b$: 0.15 g), (zone II: 0.25 g), (zone III: 0.35 g).

| Site condition | Building 1 | | Building 1* | |
|---|---|---|---|---|
| | STE (g) | Zone such STE > $a_{ref}$ | STE (g) | Zone such STE > $a_{ref}$ |
| Good Soil ($S_0$) | 0.05 | none | 0.09 | none |
| Soft Soil ($S_3$) | 0.08 | none | 0.18 | $I_a$ & $I_b$ |

## 5. CONCLUSION

This study shows the interest of the small amplitude monitoring methods in the seismic diagnosis of existing structures. The several experiments prove the robustness and the reliability of the information collected through ambient vibrations that enable the identification of the leading and negligible phenomena. These experimental data showed also the efficiency of Timoshenko beam to explain the dynamic behaviour of tested buildings. Lastly, Seismic Thresholds method based on these two aspects gives a first level of assessment analyse.

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