DYNAMIC AUSCULTATION OF BUILDINGS AND SEISMIC INTEGRITY THRESHOLD ASSESSMENT

Stephane HANS1, Claude BOUTIN2, Erdin IBRAIM3 and Pierre ROUSSILLON4

SUMMARY

Recent results of in situ measurements and their interest for a seismic assessment of existing buildings are presented and analysed. The methodology is exposed on three steps. First an experimental program was performed in order to justify the use of vibration data collected in situ for identifying the actual dynamic behaviour of usual intact buildings built according to the common practise. The response to ambient vibrations, harmonic excitation and shock loading is recorded on intact buildings but also after their structure or their vicinity was modified. Taking advantage of the demolition, the tests enable to determine the actual influence of the light work elements, full pre-cast facade panels, bearing masonry walls, and the presence of neighbouring joined buildings. These experiments realized on seven real buildings show that information gathered from ambient measurements provide reliable and efficient data of real interest for a clear understanding of the actual building behaviour. Second it is shown that the experimental modal characteristics obtained on regular concrete buildings are described successfully by suited classical or uncommon continuous beam modelling consistent with the internal structure. In a given direction of motion the key identification parameter of the relevant modelling is the frequency distribution of the two (or three) first eigenfrequencies. Third, the advantage of integrating these previous developments in the vulnerability assessment is presented and discussed. Choosing the maximum tensile strain of concrete as damage criterion for key structural elements, a maximum level of the ground acceleration (in regulation meaning as French PS92) can be determined. This so-called Seismic Integrity Threshold is directly related to the onset of structural damages. This new approach is illustrated by using the in situ records of one of studied buildings.

This work underlines the advantages of using ambient vibrations survey for the vulnerability assessment of existing buildings.

Keywords: in situ test monitoring; existing buildings; ambient vibrations; continuous beam; integrity threshold.

1. SEISMIC ASSESSMENT OF EXISTING BUILDING

In France like in lots of other countries, the question of the assessment of existing buildings has recently taken a great importance for several reasons like: (i) the seismic behaviour of a large number of existing buildings is uncertain because whether they were constructed before the first regulations against seismic risks whether the evolution of these last made their design obsolete; (ii) the public decision-makers need for reliable diagnosis to build the strategies to reduce the impact of a seismic event.

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Several statistical methodologies were developed to assess the cities vulnerability [Benedetti et al., 1988] [Spence, 1992] [ATC 40, 1996] [Risk-UE, 2003]; essentially based on the in situ observation of building damages, they are very useful to realize realistic scenarios at large scale, but only in high seismic countries for which a sufficient amount of data is available. For low seismic countries (like France) or for the study of a particular building, these approaches are not relevant.

The present paper sums up the work presented in [Hans et al., 2003] and [Boutin et al., 2003]. Taking as start point the fact that the main difficulty is the lack of information on existing buildings, the followed idea was:

(i) to use experimental investigations on real buildings before and during their demolition to identify both their dynamics and the different factors which can influence this one,

(ii) to propose a practical criterion useable in a vulnerability diagnosis, based on both experimental data and simple beam modelling.

Obviously, this approach does not claim to provide a very accurate modelling but, on the basis of experimental observations, the main goal is to propose simplified, though realistic procedure useable in a first diagnosis level.

2. EXPERIMENTAL ELEMENTS

2.1 In situ monitoring methods

The principle consists in recording the responses of the structure with synchronised accelerometers (sensibility of $8 \times 10^{-6}$g) in the bandwidth 0-50Hz. The used excitations enable to cover a large range of motion amplitude:

(i) Ambient vibrations: generated by artificial (traffic jam …) or natural (wind…) sources, similar to a white noise, it generates very low acceleration (of order of $10^{-3}$g at the top of the building); used from a long time, this method actually spreads due to its simplicity;

(ii) Harmonic forcing: an oscillator generating a horizontal sinusoidal force induces accelerations of order of $10^{-2}$g at the top;

(iii) Shocks: impacts are applied on the building by a mechanical engine; this non destructive (outside the impact point) technique can be simply implemented and tests easily repeated; the level of top accelerations reaches $10^{-2}$g, hundred times the ambient level.

Whatever the excitation is, the behaviour of the tested structure remains in elastic domain.

2.2 The monitored buildings

The eight tested buildings, located in suburbs of Lyon, were in good conditions and their demolition was just a consequence of an urban reorganization. Their architectural and engineering conception is representative of the period 1965-1975, during which a great amount of buildings was erected. Among their typical characteristics, let us underline: a structural regularity in plan (transverse and longitudinal symmetries) and in elevation (same layout of elements and no transparency at first level) and a weak amount of steel in concrete elements (noticed during the demolition). This article focuses only on five buildings, briefly described Figure 1 and below :

(i) Building C: eight storeys, floors and shear walls in reinforced concrete, lengthwise bracing mainly provided by two full precast façade panels and the shaft walls of the lift;

(ii) Buildings D-E-F: five levels, floors in reinforced concrete, walls in masonry of light (external walls) or heavy (inner walls) parapens (poor concrete); a 5cm polystyrene pointing between each building;

(iii) Building G: sixteen storeys, floors and walls in reinforced concrete, precast panels in façade, separated of a similar ten storied building by a 5cm pointing;

The foundation soil, identical for the whole of buildings, was mixed gravels and clay deposits with good mechanical properties ($v_s \approx 300$ m/s at 5m depth).

2.3 The investigated system and the processing techniques

The in situ measurements naturally lead to the modal characteristics of the Soil-Structure System (SS). Obviously, for stiff soil as in our case, the soil-structure interaction is low and can be neglected for the first modes. However, when possible, it is interesting to determine the own modal characteristics of the structure on fixed base (SB), corresponding to its intrinsic properties. Assuming a low ISS, this can be done, from ambient measurements, by suppressing the motion of rigid body (taped at the base).

The signals are processed with several usual techniques performed in both spectral and time domain:
Ambient tests: response spectra and transfer functions are built from the average of different signals and peak-picking method and pass-band width lead to the modal parameters (3S); in time domain, shapes and damping are deduced from the autocorrelation functions.

Harmonic tests: the temporal signals in forced regime are used to build the amplitude response; in free regime, amplitude ratio (of extremes) and logarithmic decrement lead to mode shapes and dampings.

Shocks tests: the same methods are performed and a study by Cauchy wavelet [] allows to quantify the weak non-linear effects.

3. EXPERIMENTAL OBSERVATIONS

3.1 The intact buildings

As an example, some results (SS characteristics) for intact buildings C and G are gathered in Tables 1&2 and in Figure 2. These results given by different excitation methods agree, and this observation is general for all tested buildings. This confirms that, even if the excitation amplitude wins a factor thousand from ambient to shocks tests, the structures systematically respond by their quasi-elastic behavior. As a consequence, the simple ambient test is sufficient to identify the whole of the quasi-elastic behavior of a structure.

A weak non-linear effect, limited to 5%, is detectable from frequencies – damping – values (but not from the shapes) as a weak decrease – increase – according to the increase of amplitude solicitation (from $10^{-5}g$ to $10^{-2}g$). Such observations have already been done by the past [ ] [ ]. This softening effect is similar to the behavior of geo-materials. Nevertheless, it remains weak.

Let us examine the soil-structure interaction. From the SS modal shapes (Fig.2), which include the soil participation, a displacement of the base is clearly visible, increasing from the first to the higher modes. Figure 3 are compared the SS and SB – i.e. Structure on fixed Base – longitudinal modal shapes for building G. The associated SB modal frequencies are (2.15;7.25;14Hz), higher than for the SS system (2.08;7;12.8Hz respectively) cause of the softness induced by the soil. Furthermore, the differences grow up from the first to the third mode. This is consistent with the increase of the modal rigidity with modal number inducing a stronger interaction.

3.2 The modified buildings

Experiments on intact buildings may be of interest for seismic assessment, but, due to the lack of reliable information on the real conception, several points remains uncertain (like the role in the dynamics of particular elements, the presence of neighbouring building …) what can generate difficulties in the theoretical interpretation. For this reason, tests were also performed after modifying the buildings. If general conclusions cannot be drawn from the particular studied cases, the trends will nevertheless be useful for buildings presenting similar configurations.
Table 1: Experimental mode parameters (3S) of building C in longitudinal direction.

<table>
<thead>
<tr>
<th>Mode number</th>
<th>Monitoring method</th>
<th>Frequency (Hz)</th>
<th>Damping (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mode 1</td>
<td>Ambient</td>
<td>4.3</td>
<td>2.85</td>
</tr>
<tr>
<td></td>
<td>Free Osc.</td>
<td>4.25</td>
<td>2.4</td>
</tr>
<tr>
<td></td>
<td>Harmonic</td>
<td>4.19</td>
<td>2.6</td>
</tr>
<tr>
<td></td>
<td>Shock</td>
<td>4.18</td>
<td>4</td>
</tr>
<tr>
<td>Mode 2</td>
<td>Ambient</td>
<td>13.4</td>
<td>3.5</td>
</tr>
<tr>
<td></td>
<td>Shock</td>
<td>12.8</td>
<td>4</td>
</tr>
<tr>
<td>Mode 3</td>
<td>Ambient</td>
<td>21</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Shock</td>
<td>22.5</td>
<td>3.8</td>
</tr>
</tbody>
</table>

Table 2: Experimental mode parameters (3S) of building G in both horizontal directions.

<table>
<thead>
<tr>
<th>Mode number</th>
<th>Monitoring method</th>
<th>Longitudinal direction</th>
<th>Transversal direction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>frequency (Hz)</td>
<td>damping (%)</td>
</tr>
<tr>
<td>Mode 1</td>
<td>Ambient</td>
<td>2.08</td>
<td>2.5</td>
</tr>
<tr>
<td></td>
<td>Free Osc.</td>
<td>1.96</td>
<td>2.6</td>
</tr>
<tr>
<td></td>
<td>Harmonic</td>
<td>1.94</td>
<td>2.3</td>
</tr>
<tr>
<td>Mode 2</td>
<td>Ambient</td>
<td>7</td>
<td>2.5</td>
</tr>
<tr>
<td></td>
<td>Free Osc.</td>
<td>6.75</td>
<td>2.6</td>
</tr>
<tr>
<td></td>
<td>Harmonic</td>
<td>6.73</td>
<td>2.4</td>
</tr>
<tr>
<td>Mode 3</td>
<td>Ambient</td>
<td>12.8</td>
<td>12</td>
</tr>
<tr>
<td>Mode 4</td>
<td>Ambient</td>
<td>20</td>
<td>4</td>
</tr>
</tbody>
</table>

Figure 2: Experimental mode shapes (3S)– at left, longitudinal direction of building C, middle and left, longitudinal and transverse directions of building G
3.2.1 Effect of light work elements

What is the contribution of light work elements in the dynamics? The demolition of building G, whose concrete was intended to be recycled, has given the possibility to investigate this question. As any pollution of concrete was admit, the light work elements were taken off before the demolition, i.e. plaster or bricks walls of 5-7cm thick, windows, doors … Ambient tests were performed on the cleared structure and only a slight (but measurable) decrease of eigenfrequencies of about 2-4% was observed. Evaluating to about 1% the loss of mass, this corresponds to a decrease of the story stiffness of about 5%. This effect can therefore be neglected in first analysis.

Figure 3 : Building G, longitudinal direction – Comparison of SS and SB modal shapes (displacement at base for SS being nullified)

Figure 4 : Modification of building F by suppressing of a masonry wall.
3.2.2 Importance of masonry parpen walls

The role of masonry walls is not always clear, due to the imperfect knowledge of the material properties and the quality of connections with the other elements. So building F was modified by suppressing the north wall of light parpens from the base to the top (Fig. 4). Shocks tests were performed before and after this operation and corresponding spectra are presented Figure 4. Before modification, the longitudinal and transverse directions were decoupled (even if the first eigenfrequencies were close). After, as the contribution of this wall in longitudinal stiffness is negligible, the longitudinal eigenfrequency (5.2Hz) is the same as before. But this absence modified strongly both the transverse stiffness (5.15 to 4.5Hz) and the symmetry of the structure, what implies a coupling of the directions (appearance of a pick at 4.5Hz in longitudinal spectra) and a torsion motion. From a simple discrete model, the experiment results enable to assess the equivalent modulus of the heavy and light parpen, respectively to $E_{HP} \approx 2.7 \text{ GPa}$ and $E_{LP} \approx 1.5 \text{ GPa}$, 10 times smaller than for the concrete. As a conclusion, despite the presence of internal walls of heavy parpens, the contribution of the light parpen is very significant and can not be neglected.

3.2.3 Role of full precast panels in facade

The progressive demolition of full precast panels on building C was also realized. The test consisted in destroying one panel and in measuring the longitudinal building response under shocks, and so on … Figure 5 shows the decrease of the first eigenfrequency. By a simple modelling based on discrete shear beam, the contribution of the two panels to the shear story stiffness is evaluated between 20 and 25%, what means they are not negligible. Let just underline that they are slightly reinforced and would not present any ductility under seismic loading.

![Figure 5: Modification of building C by progressive suppressing of façade panels.](image)

3.2.4 Influence of neighbouring joined buildings

Here is investigated the possible dynamic influence between close buildings. Ambient measurements were performed in building D in presence of (1) buildings E & F, then (2) after demolition of F, and finally (3) after both E and F destruction; the first eigenfrequencies decreased respectively of 9% and 5% in longitudinal and transverse directions (Table 3). The origin of this building coupling has to be found in the transmission of motions and stresses throughout the soil.

<table>
<thead>
<tr>
<th>Building(s) in presence</th>
<th>D-E-F intact</th>
<th>D-E intact F demolished</th>
<th>D intact E-F demolished</th>
<th>Total decreasing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal first frequency of D (Hz)</td>
<td>5.6</td>
<td>5.4</td>
<td>5.08</td>
<td>9 %</td>
</tr>
<tr>
<td>Transversal first frequency of D (Hz)</td>
<td>5.65</td>
<td>5.47</td>
<td>5.35</td>
<td>5 %</td>
</tr>
</tbody>
</table>

Table 3: Structure-soil-structure interaction
4. BEAM-LIKE MODELLING

The aim of this section is to determine simple but relevant beam modelling whose validity could be easily checked with experimental data. The analysis is here focused on regular concrete buildings made of a sufficient number of stories (at least five).

4.1 Quasi-Timoshenko beam model

The behaviour of a regular structure of height \(H\) made of elastic material (of Young’s modulus \(E\)) is governed by the spatial distribution of columns, bearing walls and floor diaphragms. In a horizontal direction, the deformations result from both bending (\(EI\)) and shear (\(K\)) contributions of the storey. Furthermore, the rotation inertia of a storey is assumed negligible. These basic assumptions lead to a specific Timoshenko beam. The non dimensional parameter \(C\), defined as \(\frac{EI}{KL^2}\) where \(L=\frac{2H}{\pi}\), evaluates the two contributions and defines the nature of the Timoshenko beam. When \(C \to 0\), the model degenerates in a Euler-Bernoulli beam and when \(C \to \infty\), in a pure shear beam.

The model is developed in [Michel, 2006], article referenced n°1246 in ECEES book. The useful characteristic of Timoshenko beam is that it exists a bijective relation between the sequence of eigenfrequencies and the non dimensional parameter \(C\). Thus for example, to one ratio \(f_2/f_1\) corresponds only one value of \(C\). This observation provides a very simple way to identify the nature of the beam behaviour from the measured sequence of eigenfrequencies or even from the \(f_2/f_1\) ratio.

4.2 Simple a priori assessment of the Timoshenko beam parameter

A direct assessment of the needed parameters (i.e. bending EI, shear K stiffness and the linear mass \(\Lambda\)) can be derived from some basic information on the structure (drawings) and from following assumptions on the constitutive material and internal deformations:

(i) connections perfectly rigid between the structural elements,
(ii) usual value for concrete: \(E = 20\) GPa, \(\nu = 0.2\) and \(\rho = 2.3\) t/m³,
(iii) infinitely rigid floors (assumption only partially justified and just used in a first level of analysis).

The storey stiffness are derived by imposing between two rigid floors whether a differential horizontal drift whether a differential rotation angle. The stiffness of each element is deduced from usual strength of materials static formulae, and right combinations lead to the researched stiffness. This procedure is applied for the two main directions of the structure.

Results are presented in Table 4 for buildings C and G. Clearly the value of parameter \(C \approx 20\) indicates a dominating shear beam behavior for building C in the longitudinal direction, and a Timoshenko beam behavior in both directions for building G \((0.1 < C < 2)\). 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 behavior.

<table>
<thead>
<tr>
<th>Building</th>
<th>C (non dimensional)</th>
<th>G (non dimensional)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Direction</td>
<td>Longitudinal</td>
<td>Longitudinal</td>
</tr>
<tr>
<td>Linear mass (t/m)</td>
<td>114</td>
<td>110</td>
</tr>
<tr>
<td>Shear parameter K (MN)</td>
<td>11895</td>
<td>27830</td>
</tr>
<tr>
<td>Bending inertia I (m⁴)</td>
<td>2140</td>
<td>1836</td>
</tr>
<tr>
<td>Parameter C</td>
<td>19.7</td>
<td>1.79</td>
</tr>
<tr>
<td>Estimated frequencies ((f_1; f_2; f_3)) with (E = 20) GPa</td>
<td>3.63 - 11.5 - 17.8</td>
<td>2.58 - 7.91 - 14.12</td>
</tr>
<tr>
<td>Experimental frequencies</td>
<td>4.45 - 14.1 - 23.5</td>
<td>2.15 - 7.24 - 13.97</td>
</tr>
</tbody>
</table>

4.3 Experimental estimate of the beam parameters

To complement the previous assessments, the beam parameters can also be estimated from the experimental eigenfrequencies.

(1) Building C: its longitudinal behaviour is assumed to be describe by a pure shear beam characterized by its shear stiffness \(K = E*S_s\); considering \(\Lambda\) and \(S_s\) as given data (deduced from drawings), only the modulus \(E\)
remains to be evaluated. A value of 31 GPa is found to fit the first experimental frequency (4.45 Hz), and the evaluated sequence of eigenfrequencies is also (4.45; 13.3; 21.8 Hz), close from experimental one (4.45; 14.1; 23.5 Hz).

(2) Building G: In its two main directions, two Timoshenko beam models were found to describe the dynamic behaviour. So a two parameters problem has to be solved, with for unknowns the parameter C and for instance the parameter K linked to the shear stiffness. As mentioned above, the ratio of the two first eigenfrequencies $f_2/f_1$ corresponds to a unique value of C, and thus by using the experimental ratio $f_2/f_1$, a experimental value of C is determined. Then, considering here again the estimated linear density and the geometry as given data, it is possible to fit the first experimental frequency by adjusting the Young’s modulus of the material. The fitting process performed independently for both directions leads to an identical modulus value of 21 GPa. Table 5 gives the results obtained by this procedure. The comparison with Table 4 shows that the fitting of frequency ratio improves the a priori estimate of parameter C for building G.

Having the beam parameters calibrated by the in situ data, the reliability of the modelling can be checked by complementary comparisons. For this purpose, the mode shapes and eigenfrequencies of higher modes are calculated and compared to the experimental data (Figure 6). The succession of eigenfrequencies is well described by the beam model, up to the third frequency for building C - with only the first frequency fitted- and up to the fourth for building G (Table 5) with only the two first frequencies fitted. Figure 6 presents the comparison of the experimental and theoretical mode shapes for two cases. Despite the fact that the modal shapes were not involved in the fitting process, there is a very good agreement between experiments and modelling, even for the modes associated to non-fitted frequencies. Furthermore the first mode curvature is consistent with the beam modelling: positive for shear beam as building C; with an inflexion for Timoshenko beam as building G in longitudinal direction; negative for beam with dominating bending effect as building G in transversal direction. Let us also mention that the Young’s modulus values determined (between 20 and 30 GPa) are realistic and seem to support the reliability of this approach. Finally, the consistency of these results obtained for three different cases leads to think that even moderately tall buildings can actually be considered as beams, whose parameters can be determined rather simply from basic information and in-situ measurements.

Table 5: Experimental fitting – Building G

<table>
<thead>
<tr>
<th>Building Direction</th>
<th>G Longitudinal</th>
<th>G Transversal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Experimental frequencies (Hz)</td>
<td>2.15 - 7.24 - 13.97 - 20.5</td>
<td>1.56 - 6.64 - 14.0</td>
</tr>
<tr>
<td>Experimental ratio $f_2/f_1$</td>
<td>3.37</td>
<td>4.26</td>
</tr>
<tr>
<td>Experimental C</td>
<td>0.50</td>
<td>0.13</td>
</tr>
<tr>
<td>Experimental Young’s modulus (GPa)</td>
<td>21</td>
<td>21</td>
</tr>
<tr>
<td>Fitted beam model frequencies (Hz)</td>
<td>2.15 - 7.24 - 13.96 - 20.1</td>
<td>1.56 - 6.64 - 14.0</td>
</tr>
</tbody>
</table>

Figure 6: Modelling-experiment comparison of longitudinal mode shapes for buildings C and G
SEISMIC INTEGRITY THRESHOLD METHOD

The question addressed here is: how could these in situ experimental data, consistently described by a simple 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 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. Below this level, the so-called Seismic Integrity Threshold (SIT), the structure remains elastic, i.e. undamaged. In the following, the principle of the SIT’s determination is described through an application to the building C.

5.1 Criterion for the onset of the structural damage

A key issue in this approach is to define the criterion for the onset of structural damage. In this paper, the discussion is especially focused on reinforced concrete structures. As the actual amount of steel reinforcements and their exact distribution in the sections are unknown for most of existing structures, it would not be relevant to adopt a stress criterion for reinforced concrete - expressed in maximum compressive or tensile stress - since such a criterion would include the effect of reinforcements. To overcome this difficulty, we use the well known fact that, independently of the presence of reinforcements, the concrete matrix cannot sustain tensile strains greater than a limit of 10.4 (m/m) (for usual concrete). Subsequently, this simple criterion of maximum concrete tensile strain is adopted. Below this limit, the concrete (and thus the reinforced concrete) remains intact; and above this limit, the cracking of the concrete begins and weakens the reinforced concrete elements. Let us underline that the choice of a strain criterion is consistent with the displacement-based methods and with the concept of maximum story drift.

5.2 Calculation of the SIT

The calculation of the SIT could be performed through common linear structural dynamic numerical methods, the model being fitted with the experimental data. However, in order to give a better insight of the method, all the process is illustrated using analytical shear beam which fits the longitudinal behaviour of Building C. Furthermore, only the first mode is taken into account considering that it is mainly responsible for the structural deformations.

The calculation consists in two main steps:

(1) Study of structural elements: a simple RoM approach is applied to determine the limit displacement between the extremities of an element for which the given failure criterion in deformation (ε = 10^{-4}). This is applied for each structural elements of a story and the minimum corresponds to the limit inter-story drift of the i-th story. Then introducing [U_i] and [ΔU_i], the normalized first mode eigenvector and the corresponding differential displacement vector, and if A is the amplitude of the first mode of vibration, the displacement [u_1] and differential displacement [Δu_1] vectors are: [u_i]=A*[U_i] and [ΔU_i]=A*[ΔU_i]. So are defined A_i = [ΔU_i]/[Δu_1], the amplitude that would trigger off the first structural damages at the floor level i. The minimum of A_i , named A^{lim}, corresponds to the amplitude which implies the first damage in the whole of structure.

(2) Use of seismic rules: here is used the determined model of dynamic behaviour – i.e. a shear beam. 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 = 1m/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), where p_1 is the participation factor of the first mode.

The SIT is reached when A(SIT) = A^{lim}, so: SIT = A^{lim}/(p_1*d(f_1)).

5.3 Case study and discussion

Considering a damping ratio of 5%, the calculated values of the SIT for building C in the longitudinal direction are presented in Table 6. The calculation has been done considering that the structure is settled on the two extreme site conditions, S0 (very good soil) and S3 (soft soil) as defined by the French seismic code. Moreover,
in order to investigate the sensibility of the SIT, the same calculations were developed for a fictitious building C’ identical to C but with a number of storey reduced to 4 (therefore more rigid with an higher first frequency).

According to the site conditions, the SIT values of building C 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 SIT values of building C’, less loaded because of its higher frequency, are higher than for building C (0.09 g to 0.18 g). If the SIT is lower than the acceleration required by the seismic code, it is believed that first damages would be induced by the reference earthquake. The French seismic code defines 4 zones, from zone (Ib) of weak seismicity to zone (III) of moderate seismicity and whose reference accelerations are respectively equal to 0.1 g and 0.35 g. It seems that this particular structure C is quite vulnerable whatever the site and seismic zone are. As an aggravating factor it should be noted that the most critical structural element is the ground floor unreinforced panel for which a brittle failure

can be expected. The gap between the SIT and the reference acceleration of the seismic zone provides an indication of the ductility needed by the building to resist the reference earthquake. The larger is the gap between these accelerations, the more attention should be paid to the structure, and further investigations may be necessary to engage.

**Table 6:** SIT values for buildings C and C’ - reference acceleration aref for each zones: (zone Ia: 0.1g), (zone Ib: 0.15 g), (zone II: 0.25 g), (zone III: 0.35 g).

<table>
<thead>
<tr>
<th>Site condition</th>
<th>Building C SIT (g)</th>
<th>Zone SIT &lt; aref</th>
<th>Building C’ SIT (g)</th>
<th>Zone SIT &lt; aref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Good soil $S_0$</td>
<td>0.05 $I_a \rightarrow III$</td>
<td>0.09 $I_a \rightarrow III$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soft soil $S_3$</td>
<td>0.08 $I_a \rightarrow III$</td>
<td>0.18 $II \rightarrow III$</td>
<td></td>
<td></td>
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6. CONCLUDING REMARKS

This study shows the interest of the small amplitude auscultation methods in the seismic diagnosis of existing structures. The several presented experiments prove the robustness and the reliability of the information collected through ambient vibrations which enable the identification of the leading and negligible phenomena. Even if the data are limited to the quasi-elastic domain, their knowledge is of very first importance. It would have been possible to use sophisticated methods to obtain an accurate description of the buildings. However, keeping in mind the necessity of assessing a large number of buildings, simple modelling based on few elementary, though physical assumptions, are favoured. These approaches give an approximate description sufficiently realistic for engineering purposes. In the same spirit, the concept of Seismic Integrity Threshold, which is based on real data, presents practical advantages. It minimizes the use of uncertain assumptions on the nonlinear post-elastic behaviour (at the three scales of the material, the structural elements and the structure) and it provides an acceleration level that can be easily compared with the reference accelerations given by the codes. From both reference accelerations and SIT values, the extend of the ductility needed by the structure can be evaluated. The confrontation of this value with the usual ductility of the present material could provide a good criterion to identify the most vulnerable structures.

7. REFERENCES


