A critical review of current approaches on the determination of seismic force demands on nonstructural components

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ABSTRACT: During and after an earthquake damaged nonstructural components pose severe threats to human lives and usually account for the majority of economic losses in non-residential buildings. Thus, seismic design of secondary structures should be duly considered, whereas seismic forces acting on components have to be known with appropriate accuracy. To serve this purpose four different approaches to determine horizontal seismic forces on components were reviewed: i) Simplified formulas contained in current code provisions; ii) Simplified formulas enhanced by refined determination of peak floor accelerations by means of modal superposition methods; iii) Decoupled time history analyses (floor response spectrum method) and iv) Coupled time history analyses. All methods were applied to a set of three realistic moment resisting steel frames, revealing large discrepancies between simplified and refined methods. Outcomes in terms of peak accelerations at the supporting structures’ floors as well as at the component itself are discussed. It was found that the use of strongly simplified formulas as they are suggested in current codes could lead to unsafe design of nonstructural components. It is further shown that calculation of floor accelerations, allowed as refinement of simplified formulas in some provisions, is not as straightforward as might be expected. Further results show that incorporation of heavy components in a coupled model with its supporting structure can lead to significantly reduced calculated acceleration values. In this context it is discussed why caution should be paid when Rayleigh damping approach is used. Finally recommendations are given regarding the applicability of simple or refined methods.

KEY WORDS: Nonstructural components; Peak floor accelerations; Floor response spectra; Modal superposition method; Response spectrum method; Rayleigh damping; Eurocode 8.

1 INTRODUCTION

Nonstructural components are nonbearing secondary structures usually attached to a primary supporting structure, e.g. mechanical, electrical or architectural equipment. They can suffer damage at lower earthquake intensities than the building itself and they usually account for significant economic loss and threat to human health during and after earthquakes. Loss of their functionality can lead to costly downtimes e.g. of industrial facilities or malfunction of lifeline systems. Consequently due to usually high investment costs and/or risk potential special attention should be paid to the seismic design of such secondary structures.

Nonstructural components can be subdivided into deformation or acceleration sensitive components, depending on susceptibility to relative displacements or to inertia forces caused by an earthquake - the latter group is the subject of this paper. For the determination of seismic force demands on nonstructural components different approaches with increasing complexity exist:

i) Simplified formulas suggested by current code provisions;
ii) Simplified formulas enhanced by refined determination of peak floor accelerations by means of modal superposition methods;
iii) Decoupled time history analyses (floor response spectrum method);
iv) Coupled time history analyses.

The simplest method i) is used most often in current design practice for ordinary and industrial structures, whereas the much more sophisticated floor response spectrum approach iii) is usually applied when dealing with critical facilities like high-risk industrial structures or nuclear power plants. Approach i) serves as convenient means when no or just little information about the component and the supporting structure is given. Contrarily dynamic building properties have to be known for method ii). Both approaches need a design response spectrum as input, which is handily available in earthquake codes. The calculations which have to be executed are of quasi-static nature; on the contrary the last two methods iii) and iv) utilize dynamic time step analyses and hence require acceleration time histories representing the design earthquake. Consequently detailed dynamic properties of the supporting as well as of the secondary structure have to be known. Sufficient reliability generally requires the use of a number of acceleration time histories; compared to the first two methods the effort is drastically increased. Inelastic behaviour can be explicitly accounted for in the last two methods iii) and iv), while its consideration in the methods i) and ii) is limited to an approximation by means of assumed modification factors.

In the study presented herein all 4 approaches outlined above have been applied to a set of three 3-bay moment resisting steel frames with different numbers of storeys (5, 10 and 15), which are part of realistic office buildings designed according to Eurocode 8 [1] in [2] and are illustrated in Figure 1. The buildings have a total seismic mass of 345 t, 703 t and
1064 t respectively. Modal periods and effective modal mass to total mass ratio of the first 3 modes are given in Table 1. As earthquake input the Eurocode 8 elastic response spectrum (type 1, soil class B, $a_g = 0.25g$), for which the case studies were designed, or 7 spectrum compatible artificially generated earthquake time histories have been used. More information on the frames and earthquake input can be found in [3].

In all four methods the component is represented by a single-degree-of-freedom system and assumed to be acceleration-sensitive, i.e. damage is mainly induced by inertia forces rather than displacement drifts. The results of each applied method are discussed in the following sections along with a more detailed description of each approach.

Figure 1. Investigated moment resisting frames.

Table 1. Dynamic properties of studied steel frames.

<table>
<thead>
<tr>
<th>Building</th>
<th>Mode $j$</th>
<th>5-st.</th>
<th>10-st.</th>
<th>15-st.</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_j$ [s]</td>
<td>1</td>
<td>1.12</td>
<td>81</td>
<td>2.03</td>
</tr>
<tr>
<td>$m_{ef}/m_{tot}$</td>
<td>2</td>
<td>0.34</td>
<td>11</td>
<td>0.68</td>
</tr>
<tr>
<td>[%]</td>
<td>3</td>
<td>0.18</td>
<td>4</td>
<td>0.39</td>
</tr>
</tbody>
</table>

2 DETERMINATION OF SEISMIC FORCE DEMANDS

2.1 Simplified approaches in current provisions

Simplified formulas contained in code provisions have usually the following format: the design peak ground acceleration is amplified resulting in the peak component acceleration, which multiplied by the component’s mass yields the equivalent seismic static force. This force is modified by an importance factor to take into account the hazard and economic value of the component. Further it can be diminished by a component reduction factor, called behaviour or response modification factor, which allows for lower design forces if proper ductility and energy dissipation capabilities are available. The core of these approaches is the peak component acceleration ($PCA$), which is determined by the amplification of peak ground acceleration ($PGA$), usually described by two effects: 1) the amplification of $PGA$ caused by the supporting structure resulting in the peak floor acceleration ($PFA$) and 2) the amplification of the $PFA$ caused by the component itself and resulting in the $PCA$. Thus, the approach of simplified code formulas can schematically be drawn in the following format:

$$PCA = PCA/PGA \cdot PFA/PGA \cdot PGA$$

(1)

The amplification factor $PCA/PFA$ is usually described by a constant factor or as a resonance function depending on the ratio of the component’s fundamental period to the supporting structure’s one (e.g. Eurocode 8)\(^1\). The variation of floor accelerations over the height of the building is assumed to increase linearly, according to a typical fundamental mode sway.

Some exemplary coded values of the amplification factors are given in Table 2. In order to focus on the problem which is subject in this paper all other factors as for example reduction factor and importance factor were not considered and thus have been set equal to 1.

Table 2. Range of amplification factors contained in coded simplified formulas.

<table>
<thead>
<tr>
<th>Provision</th>
<th>$PFA/PGA$</th>
<th>$PCA/PFA$</th>
<th>$PCA/PGA$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eurocode 8 [1]</td>
<td>1 - 2.5</td>
<td>1 - 2.2/2.5(^1)</td>
<td>1 - 5.5(^1)</td>
</tr>
<tr>
<td>ASCE 7 [4]</td>
<td>1 - 3</td>
<td>1 or 2.5</td>
<td>1 - 4</td>
</tr>
<tr>
<td>NZS 1170.5 [5]</td>
<td>1 - 3</td>
<td>0.5 - 2</td>
<td>$PCA \leq 3.5g$</td>
</tr>
<tr>
<td>NCh 2369 [6]</td>
<td>1 - 4</td>
<td>1 - 2.2</td>
<td>$PCA \leq 1.0g$</td>
</tr>
</tbody>
</table>
\(^1\) Depending on attachment height in relation to building height. See also footnote 1 below.

Amplification factors $PFA/PGA$ which resulted from the conducted time history analyses are shown in Figure 2 as mean values of all 7 earthquakes. Only the Eurocode 8 approach is shown for comparison, which is the less conservative among all codes concerning the $PFA/PGA$ amplification factor.

Figure 2. Mean of case study results obtained from time history analyses compared to Eurocode 8 approach.

Whereby the 5-storey building is relatively well approximated by the code the assumptions are overly

\(^1\) In the Eurocode 8 approach both amplification factors $PCA/PFA$ and $PFA/PGA$ are slightly condensed in one term, but the scheme in Equation (1) in general holds true.
conservative for the higher buildings. Since the Eurocode 8 approach as well as all other simplified code approaches contained in Table 2 do not consider building topology, they are not able to take into account the big scatter between buildings of different height. The profiles of normalized peak floor accelerations shown in Figure 2 in dependence of number of building storeys, i.e. the fact that higher buildings have much lower PFA/PGA amplification factors, has been proved to be of general evidence for regular structures in several other studies and also by evaluation of recorded field data (e.g. [8] [9] [10]). Suggestions have been made to amend current code provisions by taking building properties and thus different behaviour into account [11] [12].

Considering Figure 2 it is noted that behaviour of the buildings is in general elastic. Eurocode 8 deformation limiting demands for serviceability limit states are restrictive and thus can prohibit large inelastic behaviour, which leads often to significant overstrength in the case of moment resisting frames, caused by the typical flexibility of this structural typology [13] [14]. The ground motion intensity used herein, characterized by a PGA of 0.3 g did not lead to high nonlinear behaviour into the considered buildings [2]. However it is shown in [3] by means of incremental dynamic analyses with the same buildings studied herein, that strong plastification of the buildings can significantly decrease PFA/PGA amplification factors as well as floor response spectra discussed in section 2.3.

The PFA values represent the accelerations of very rigid components attached to the specific floor. To review the coded approaches for accelerations of more flexible components, the amplification factors PCA/PFA and PCA/PGA will be investigated in section 2.3 and 2.4 by comparison to results obtained by dynamic time history analyses. More refined approaches to determine the PFA as compared to the coded PFA/PGA amplification factors are discussed in the following section 2.2.

2.2 Determination of peak floor acceleration by modal superposition method

Instead of estimating the PFA by means of simplified formulas as described above, more realistic results can be achieved by using dynamic analyses such as modal superposition method. For example ASCE 7 and NCH 2369 allow for such a more refined method. However no guidance is given on how to apply response spectrum method to calculate accelerations; simply reference is given to the code section where this method is applied to obtain equivalent seismic forces to be applied on the building in a static analysis. However this is not as straightforward as might be expected, as will be shown with the following considerations.

By means of commonly used mode superposition method the relative responses of a structure are calculated according to Equation (2):

$$\{a_{\text{rel}}(t)\} = \sum_{j} \{\phi_j\} \cdot \Gamma_j \cdot a_{\text{rel},j}(t)$$

(2)

Whereby $a_{\text{rel}}(t)$ is the relative acceleration in respect to the moving ground as function of time, $\{\phi_j\}$ is the Eigenvector, $\Gamma$ is the modal participation factor and the subscript $j$ indicates the mode number. In contrast to usual static force calculations not the relative displacement response but the absolute acceleration response $a_{\text{abs}}(t)$ is of interest here, which can be obtained by adding the ground acceleration $a_g(t)$:

$$\{a_{\text{abs}}(t)\} = \sum_{j} \{\phi_j\} \cdot \Gamma_j \cdot a_{\text{rel},j}(t) + \{r\} \cdot a_g(t)$$

(3)

Here $r$ is the influence vector, filled with 1 for the degrees of freedoms in direction of the ground movement and with 0 otherwise. With some few transformations Equation (3) can be rewritten as:

$$\{a_{\text{abs}}(t)\} = \sum_{j} \{\phi_j\} \cdot \Gamma_j \cdot a_{\text{abs},j}(t) + \{r\} - \sum_{j} \{\phi_j\} \cdot \Gamma_j \cdot a_g(t)$$

(4)

By using modal superposition method a simplification of practical relevance can be achieved if time information is neglected and only peak responses are considered, resulting in the response spectrum method. Such peak responses can be taken from readily available design response spectra. Since information on the time of occurrence of peak responses gets lost, superposition of modal peak responses has to be done by statistical methods. The most common way in using response spectrum method is applying the SRSS (square root of sum of squares) combination rule as in Equation (5), where the prefix $S$ reflects that the maximum absolute or spectral value of the response history is meant:

$$\{S_{a_{\text{rel}}}(t)\} = \sqrt{\sum_{j} \{\phi_j\} \cdot \Gamma_j \cdot S_{a_{\text{rel},j}}^2}$$

(5)

A further important simplification is usually done in using not all, but just a few modes, which are able to represent global response sufficiently.

For the calculation of the absolute accelerations in the structure sometimes Equation (6) is used by replacing $S_{a_{\text{rel},j}}$ with $S_{a_{\text{abs},j}}$ and by superpositioning only a few modes.

$$\{S_{a_{\text{abs}}}(t)\} = \sqrt{\sum_{j} \{\phi_j\} \cdot \Gamma_j \cdot S_{a_{\text{abs},j}}^2}$$

(6)

When comparing Equation (6) to Equation (4) it is evident that this is mathematically not consistent, since the second term is fully neglected. As easily can be seen in Equation (6) nodes restrained to the ground reside at an absolute acceleration of zero, instead of being equal the ground acceleration as in Equation (4). A further concern about Equation (6) is that application of commonly used SRSS combination rule is not self-evident when dealing with absolute accelerations, as ground acceleration is incorporated in the modal responses - thus uncorrelated responses are at least questionable.

In order to assess the suitability of Equation (6), Equation (4) was used, thus not neglecting time information of the responses, which can be called modal superposition in its time domain. It was applied to the three buildings in two different forms: a) using the full equation and b) neglecting the 2nd summand. Results are shown in Figure 3 for different floors of the 5-storey building as function of included modes and compared to results by including the first 200 modes, when the results converged.
antly de acceleration formulation rather than the or the 6
1.2
1.2

th
s
30
20
0.4
1.2
r
Eq. (6) mode 1 only
Eq. (6) mode 1 to 3
10
1.8
1.2
15
th
0.8,

d.
nts which are not in
40
0.6
2.5
3.5
0

Proceedings of the 9th International Conference on Structural Dynamics, EURODY 2014

Figure 3. Peak floor accelerations at various floors obtained by modal superposition method in the time domain for the 10-
storey building as mean over all 4 earthquakes.

Including the 2nd summand of Equation (4) enforces the
constrained nodes’ absolute accelerations to be equal the
ground acceleration. For the lowest floor the quality of results
increases significantly when considering only a few modes.
However the term is not able to compensate the errors
introduced by mode truncation at the mid and upper floors,
nevertheless peak floor accelerations are well represented by
using 3 modes for the 10th and 4 modes for the 6th storey. A
high number of modes is needed to represent floor
accelerations correctly at the lowest floors.

To further assess Equation (6) it has been applied to the 3
buildings. Figure 4 shows results taking into account the first
mode only, the first and second mode as well as the first 3
modes, which in these cases would be fully sufficient to
calculate equivalent static forces. The results are compared to
reference time history results, showing a very good agreement
at the mid and upper floors when 3 modes are included.
Nevertheless values at lower floors up to 20% of total height
are drastically underestimated. Instead of converging rightly
to the PGA at the lower floors, the lowest nodes remain at a
spurious peak acceleration of zero.

As can be seen the usually applied response spectrum
method without consideration of higher modes is not directly
applicable to the calculation of floor accelerations. However
at mid and upper floors peak floor accelerations are
approximated well, which is not the case at lower floors where
higher mode contribution is much more significant and its
neglecting not justified. By using classic response
superposition methods in general more modes are necessary to
obtain satisfactory results for absolute responses than for
relative ones. Alternatively appropriate mode superposition
methods should be used (see also [15] [16] [17]), which have
its origin in the mode acceleration formulation rather than the
more commonly known mode displacement formulation and
therefore are suited better.

Figure 4. Mean values of PFA/PGA factors obtained by
Equation (6) compared to time history results.

2.3 Decoupled time history analysis (floor response
spectrum method)
In case of important and/or hazardous components, Eurocode
8 prescribes the use of the floor response spectrum method.
Such a cascaded approach neglects dynamic interaction
effects, i.e. the influence of component response on the
supporting structure’s behaviour. These effects are negligible
if the component mass is very small in relation to the overall
systems mass. Ratios of 1%, at which dynamic interaction
becomes important, can be found in literature (see e.g. [18]).

Some representative floor response spectra are shown in
Figure 5 for different normalized heights z/H, where z is the
height of the components attachment point and H is the
overall building height. All floor response spectra have been
calculated for a component damping of 5%. For ease of
comparison the PCA is normalized to the PGA and the
component periods Tc are normalized to the fundamental
period Ti of each building. The shown normalized floor
response spectra are the mean of all seven accelerograms. As
example of coded simplified formulas the Eurocode 8
approach is shown.

It can be seen that higher mode effects are significant and
their neglecting as done in several coded simplified
approaches is not justified. Further, when the component is in
tune with the fundamental mode of the 5-storey structure
resonance effects are underestimated. On the contrary the
higher buildings are overestimated by the Eurocode 8
approach at resonance with the fundamental period; however
this is due to the fact that PFA’s are grossly overestimated.
The combination of both poorly estimated PFA/PGA and
PCA/PFA amplification factors leads to reasonable results for
the 10-storey structure’s roof floor. The Eurocode 8 approach
is overly conservative for very flexible components and such
components which have a period in between dominant periods
of the supporting structure i.e. components which are not in
tune. As can be seen further the amplification factors
compared between the different buildings are far from
uniform, as it was already observed for the PFA/PGA factors.
This holds also true when the amplification factors
PCA/PFA are considered, which are tabulated in Table 3 as
mean values for all buildings and different normalized
heights. Several components have been evaluated:
Components in tune with the fundamental, the second and the
The third mode of its supporting structure as well as components not in tune with these modes. With “in tune” or “tuned”, such components are meant, which period approaches one of a dominant mode period of the building. A component which period is within ±10% of one of the dominant of the supporting structures was defined in tune. A band of ±10% was used in order to ensure the peak value near the resonance case; it is limited in order not to catch tune effects of other modes, which worked well proved by visual judgment for each case. Contrary a not tuned component was defined as one component with a period which differs by at least ±30% from one of the first 3 modes of the supporting structure.

The highest amplification factor PCA/PFA from all codes compared in Table 2 is 2.5. In a lot of cases this is significantly below the obtained amplification values listed in Table 3. Furthermore, the maximum coded value is questionable because of following simple logic: An amplification factor of 2.5 is generally assumed for SDOF systems which are based on the ground and excited by earthquakes. This value is originally based on studies conducted by Newmark [19] and consolidated in the fact that the maximum value of the elastic response spectrum is obtained by multiplying the PGA by 2.5 e.g. in Eurocode 8. Therefore an amplification factor of 2.5 would be reasonable for components on the ground. If the component in contrast is mounted on a supporting structure, its base excitation is of totally different nature. The supporting structure acts like a filter and intensifies the energy contents of the earthquake at the building’s dominant modes, resulting in a more sine wave like signal. If the component is in tune with one of the building periods, this filtered signal acts much more detrimental for the component, thus a higher amplification factor of 2.5 is obvious. Following these simple thoughts it is self-evident that in most cases analytical solutions will show much higher amplification factors PCA/PFA than 2.5 if component and supporting structure are in tune (see e.g. [20]).

This is also confirmed in [21], where 5% damped floor response spectra of several hundred recorded floor accelerations have been calculated. Maximum PCA/PFA amplification factors of about 8 were obtained, which would be even higher for lower damping ratios. Considering the fact that design code’s intentions are to design for some type of mean and not maximum possible demands, the authors finally suggest a maximum amplification factor PCA/PFA of 3.3 for new buildings.

<table>
<thead>
<tr>
<th>Norm. height</th>
<th>0.2</th>
<th>0.4</th>
<th>0.6</th>
<th>0.8</th>
<th>1.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>In tune</td>
<td>5-st.</td>
<td>1.9</td>
<td>3.0</td>
<td>4.2</td>
<td>5.4</td>
</tr>
<tr>
<td>with mode 1</td>
<td>10-st.</td>
<td>1.1</td>
<td>1.9</td>
<td>3.3</td>
<td>4.9</td>
</tr>
<tr>
<td></td>
<td>15-st.</td>
<td>0.6</td>
<td>1.0</td>
<td>2.0</td>
<td>3.0</td>
</tr>
<tr>
<td>In tune</td>
<td>5-st.</td>
<td>4.0</td>
<td>4.6</td>
<td>3.2</td>
<td>1.7</td>
</tr>
<tr>
<td>with mode 2</td>
<td>10-st.</td>
<td>3.3</td>
<td>3.9</td>
<td>2.9</td>
<td>2.6</td>
</tr>
<tr>
<td></td>
<td>15-st.</td>
<td>3.1</td>
<td>4.7</td>
<td>4.1</td>
<td>2.3</td>
</tr>
<tr>
<td>In tune</td>
<td>5-st.</td>
<td>3.0</td>
<td>2.0</td>
<td>2.0</td>
<td>1.9</td>
</tr>
<tr>
<td>with mode 3</td>
<td>10-st.</td>
<td>3.5</td>
<td>2.5</td>
<td>3.3</td>
<td>2.0</td>
</tr>
<tr>
<td></td>
<td>15-st.</td>
<td>3.3</td>
<td>2.9</td>
<td>3.8</td>
<td>2.7</td>
</tr>
<tr>
<td>Not in tune</td>
<td>5-st.</td>
<td>2.8</td>
<td>2.3</td>
<td>2.0</td>
<td>2.3</td>
</tr>
<tr>
<td>with 1st 3</td>
<td>10-st.</td>
<td>2.5</td>
<td>2.1</td>
<td>1.7</td>
<td>2.3</td>
</tr>
<tr>
<td>modes</td>
<td>15-st.</td>
<td>2.9</td>
<td>2.2</td>
<td>2.4</td>
<td>2.9</td>
</tr>
</tbody>
</table>

Regarding every earthquake input separately, instead of taking the mean values, maximum amplification factors of 6.4, 5.4 and 5.2 for the 5-st., 10-st. and 15-st. building respectively have been found. As comparison the German standard for nuclear facilities KTA 2201.4 [22] employs an amplification factor PCA/PFA of approximately 6.5 for the resonance case, if the supporting structure as well as the component do have damping ratios of each 5%. Amplification values contained in KTA 2201.4 match in general the results obtained by floor response spectrum approach better.

From the explanations before it is clear that floor response spectra will in general yield higher amplification factors when the component is in tune with a dominant mode of its supporting structure, and thus the coded formula is not conservative in these cases. Furthermore it should be noted that amplification factors contained in Table 3 as compared to current code provisions can be especially higher due to the fact that resonance with higher modes is usually fully neglected. Moreover, the use of upper limits suggested in codes (see Table 2) could lead also to big differences between outcomes of coded formulas and numerical simulations. Thus, in [23] it is suggested to consider of dropping the upper limit in ASCE 7. However in many cases where the component is not in tune with one of the building’s dominant mode, coded assumptions seem to be conservative or even uneconomic.

![Figure 5. Normalized floor response spectra for a component damping ratio of 5% compared to coded approaches.](image-url)

![Table 3. PCA/PFA amplification factors as mean over all 7 accelerograms for tuned and not tuned components.](table-url)
2.4 Coupled time history analysis

For heavy components the decoupled approach can yield overly conservative and thus uneconomic results. In such cases a dynamic time history analysis where supporting structure and component are coupled in one model can provide more realistic results. The most crucial drawback of such a coupled analysis is that design of primary and secondary cannot be separated anymore. This is impractical for two reasons: Firstly both design processes are usually made by different design teams and also at different project stages. Such the characteristics of the component are just very roughly known when the supporting structure’s design is taking place or vice versa. Secondly if the component’s dynamic properties or the attachment place change during design process, this would enforce a new analysis of the complete structure, potentially causing a redesign of the supporting structure.

Two types of coupled systems have been investigated: A) Only the mass was added to the building’s floor, any dynamic interaction effect between supporting structure and component has been neglected; B) The mass was connected to the building’s floor by means of a horizontal spring. While holding the mass constant the period of the component was adjusted by the spring’s stiffness. Two different masses have been investigated: i) 1 t, representing light to moderate weight components, e.g. HVAC equipment; ii) 10 t, representing heavy components as for example chillers, vessels or water storage tanks.

As illustrative example PCA/PGA amplification factors are shown in Figure 6 for one earthquake and the 5-storey structure. Compared are the values obtained by floor response spectrum method, with and without consideration of the component mass at the attachment floor of the component, and values obtained by fully coupled analyses – for a component mass of 1 and 10 t respectively at the 2nd and 5th floor. For the coupled analyses each point represents one time history analysis. Values between the explicitly calculated spots are spline interpolated. As can be seen dynamic interaction can significantly decrease acceleration demands of components if they are in tune with a dominant mode of the supporting structure and are not light. However for not tuned components beneficial effects of interaction are much less or even nonexistent. Concluding from Figure 6 already the 1 t component’s dynamic interaction has some decreasing effect on component demands for the 5-storey structure; however for the heavier 10- and 15-storey supporting structures the mass of 1 t was too low to have markedly effect. As the floor masses of all buildings are almost equal, it can be concluded that the mass ratio of component mass to supporting structure’s mass rather than to the mass of the attached floor should be used as measure to assess severance of dynamic interaction effects.

In the same fashion as in Table 3 PCA/PFA amplification factors are shown for a component with a mass of 10 t as mean over all 7 earthquakes in Table 4. Component period increments of at least 0.05 s have been used with a maximum considered period of 3 s whereby the first 5 periods of the uncoupled supporting structure have been also included and values in between have been spline interpolated. As can be seen amplification factors are lower as compared to decoupled results, however amplification factors also reach values well above 2.5.

![Figure 6](image_url)

Figure 6. Amplification factors PCA/PGA obtained by decoupled and coupled analyses for the 5-storey system for accelogram no. 1 at two different floors.

<table>
<thead>
<tr>
<th>Norm. height</th>
<th>0.2</th>
<th>0.4</th>
<th>0.6</th>
<th>0.8</th>
<th>1.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>In tune</td>
<td>1.7</td>
<td>2.3</td>
<td>2.9</td>
<td>3.0</td>
<td>2.3</td>
</tr>
<tr>
<td>with 5-st.</td>
<td>1.0</td>
<td>1.6</td>
<td>2.5</td>
<td>3.4</td>
<td>2.2</td>
</tr>
<tr>
<td>mode 1</td>
<td>0.6</td>
<td>1.0</td>
<td>1.7</td>
<td>2.7</td>
<td>1.7</td>
</tr>
<tr>
<td>In tune</td>
<td>3.5</td>
<td>3.0</td>
<td>2.2</td>
<td>1.5</td>
<td>2.0</td>
</tr>
<tr>
<td>with 10-st.</td>
<td>2.9</td>
<td>3.2</td>
<td>2.6</td>
<td>2.4</td>
<td>2.6</td>
</tr>
<tr>
<td>mode 2</td>
<td>2.9</td>
<td>3.6</td>
<td>3.5</td>
<td>2.2</td>
<td>2.6</td>
</tr>
</tbody>
</table>

For all the analyses the commonly used Rayleigh damping approach was applied, whereby the 1st and 2nd modal period of each building was set to 5% of critical damping. The alteration of building period due to attached mass or stiffness of the component was not taken into account to avoid peculiar cases where 1st and 2nd period of the coupled structure are almost equal and also in order to not have further changing parameters which would make comparison difficult. As with the Rayleigh approach just the 2 chosen modes have the specified damping ratio, all other have either lower or higher damping ratios. This is illustrated for the 15-storey building in
Figure 7 where component demands are shown at the highest floor. The extreme damping of high modes, which usually accompanies Rayleigh damping approach, is deemed to be uncritical for global response assessment. However, as the response of a local mode, here the mode or modes comprising the response of components, is of paramount interest, such a bias could be critical. As can be seen, stiff or very flexible modes, which reproduce the response of the corresponding component, have much higher damping ratios than 5%. Thus responses of these components are biased by unintended high component damping ratios and the evaluation of coupled analyses should be done carefully. This effect is however not critical for extreme stiff or flexible components, since viscous damping does not influence their response.

![Amplification factors PCA/PGA obtained by decoupled and coupled analyses for the 15-storey system for accelerogram no. 1 at roof level along with the applied component damping ratio.](image1)

Figure 7. Amplification factors PCA/PGA obtained by decoupled and coupled analyses for the 15-storey system for accelerogram no. 1 at roof level along with the applied component damping ratio.

To qualitatively assess the amount of possible bias at the 3rd mode resonance the upper part of Figure 7 has been reproduced in the lower part including a component mass of 1 t, which showed to have no dynamic interaction effect at the 1st and 2nd mode resonance for the 1064 t structure. However in the biased region the coupled analyses yield lower demands, which are caused by the higher damping, which usually is not intended by the analyst.

Concluding in general it would be desired to assign the required damping directly to the component mode in a coupled analysis. Thus in the Rayleigh damping approach this component mode could be used to specify Rayleigh damping. It should be verified how global modes are influenced by such a modification. Another way would be to use modal superposition method, where damping can be specified directly for each mode. However it is restricted to linear analysis only. If nonlinear analysis is needed the more general Caughey damping approach (see e.g. [24]), where damping can be specified for various modes, would be suitable. However in the vast majority of commonly used software this approach is not supported. Thus in the end the practical engineer will usually stick to use Rayleigh damping approach, but should bear in mind limitations and background of this type of damping when conducting coupled analyses.

3 CONCLUSIONS

Four methods for the determination of force demands on nonstructural components were studied, with simple to more complex background. The intent of this paper is to show the challenge and possible pitfalls, which could be overseen in applying some of these methods. Also deficiencies in current coded formulas are highlighted, which are not able to describe peak floor accelerations or force demands on components realistically in a lot of cases.

Based on the conducted analyses presented in this paper following conclusions can be drawn:

- Higher mode resonance can be of crucial importance and thus in general should be accounted for when component force demands are evaluated.
- A big scatter among buildings of different height exists in terms of peak floor accelerations (PFA) as well as peak component accelerations (PCA), which is not accounted for in current codes’ simplified formulas.
- Amplification factors of PCA normalized to PFA as well as PFA normalized to peak ground accelerations (PGA) used in current codes are not able to predict adequately their response parameters. Both not well balanced factors can however lead to reasonable results in some cases.
- PFA/PGA amplification factors contained in current codes can be highly overestimated in the case of higher/more flexible buildings.
- On the contrary PCA/PFA amplification factors appear to be too low and theoretically not plausible for tuned components. Analytical results show in general much higher amplification factors when component’s and supporting structure’s dominant mode periods are close.
- Extreme high amplification factors of PCA/PFA for very light, low damped and tuned components which are sometimes obtained by analytics could be doubted to occur in reality. Since no or very limited field data is available, effort should be made to obtain such recorded data in order to judge on this issue.
If floor accelerations are calculated by classical response spectrum method in general more modes will be necessary than usually e.g. when computing base shear forces. This especially holds true for lower floors, since truncated classical superposition methods are not proper in these cases.

A decoupled approach, i.e. floor response spectrum method, can lead to overly conservative results for heavy components if they are in tune with one of their supporting structure’s dominant modes. This can already be true at a low ratio of component mass to supporting structure mass of 1 %. Caution should be paid when Rayleigh damping approach is used in a coupled analysis. Untended high damping can lead to underestimation of components in tune with higher modes of the supporting structure.

In summary, for the calculation of seismic force demands on nonstructural components following recommendations can be given:

- If the component mass is more than 1 % of the supporting structures mass, and its period is in the range of one of the dominant periods of the supporting structure, a coupled analysis should be conducted to obtain more economic results. For light components in general the floor response spectrum method is recommended, since it explicitly takes into account higher mode resonance.
- If simplified formulas are used, resonance should be not only considered at the fundamental but also at higher modes. The modal shapes of the building should be examined in order to estimate the significance of a mode at the considered floor.
- To attenuate conservatism in higher building’s peak floor accelerations, they should be calculated by means of suitable response spectra methods rather than simplified coded estimates. The used response spectrum method should be suitable for this purpose; otherwise sufficient modes should be included to avoid severe mode truncation errors especially at lower floors or a lower bound should be set, which is at least the PGA.
- The use of a peak component acceleration to peak floor acceleration amplification factor PCA/PGA of in general higher than 2.5 is suggested. The limitation of forces acting on nonstructural components contained in coded formulas to an upper limit can be unsafe and thus is not recommended.
- If coupled analyses in conjunction with Rayleigh damping are conducted, the presence of unintended high damping for component modes should be checked to avoid biased results. As consequence Rayleigh damping should be modified, other types of damping should be applied or floor response spectrum approach should be used.

As final remark it can be stated that a new or refined simplified code formula for the determination of seismic force demands on nonstructural components is necessary. Such a formula should be more reasonable than current ones, which lack rationality and can lead to uneconomic as well as unsafe design of nonstructural components. In such a new formula building properties should be taken into account to have more reliable results. Since the lack of detailed component properties during design process of the supporting structure is often a fact that cannot be altered it should account for that, however allow for more economic predictions if such information is available.

REFERENCES